Preliminary Foundation Investigation and Design Report – Westminster Drive Underpass, Site 19-366

Stantec

Highway 401 Structure Replacements and Interchange Improvements

G.W.P. 3070-09-00

Geocres No. 40I14-150

Job No. 165000776

September 2012

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PRELIMINARY FOUNDATION INVESTIGATION REPORT
For
G.W.P 3070-09-00

Westminster Drive Underpass
City of London

1.0 Introduction

Stantec Consulting Ltd. (Stantec) was retained by the Ministry of Transportation, Ontario (MTO) to undertake a preliminary design for the replacement of the existing Westminster Drive Underpass at Highway 401 in the City of London, Ontario.

This Foundation Investigation Report has been prepared specifically and solely for the proposed bridge replacement structure.

Project Number: G.W.P.: 3070-09-00

Project Location: Westminster Drive and Highway 401, London

The work was carried out under Agreement Number 3009-E-0028 with Stantec Consulting Ltd., the Preliminary Design Consultant for this project.

2.0 Site Description and Geology

Site Location

The site location is shown on the Key Plan inset to Drawing No. 1, provided in Appendix A. At the project site, Westminster Drive runs approximately in the northwest-southeast direction while Highway 401 runs approximately in the southwest-northeast direction. For the purpose of this project, Westminster Drive is assumed to run in the north-south direction while Highway 401 is assumed to run in the east-west direction. Chainage increases from south to north on Westminster Drive and west to east on Highway 401.

General Site Description

At the project site, Westminster Drive is carried over Highway 401 by a single-span bridge (Westminster Drive Bridge). Highway 401 is a six lane (three lanes in each direction) divided freeway. The span of the existing bridge across Highway 401 is approximately 25 m. Westminster Drive currently has a single lane in each direction and the traffic is controlled by a “stop condition” located on either side of the bridge structure. Photographs 1 through 4 show the general site features.

The existing drainage at this site consists of catch basins along the paved center median leading to storm sewers, and ditches and culverts along the outside lanes.

Physiographic Description

The site is located within a physiographic region known as the Mount Elgin Ridges (Chapman and Putnam, 1984). The ridges are generally moraines of pale brown calcareous clay or silty clay, whereas the vales generally consist of alluvium deposits of gravel, sand or silt. These regions were formed from clay till similar to that of Wyoming Moraine and the Stratford plain. The surficial deposits in the region generally consist of clay loam, silt loam and sands.

In the vicinity of the project site the terrain is generally undulating with gentle slope and hence good natural drainage in some areas.

3.0 Investigation Procedures

3.1 FIELD INVESTIGATION

The geotechnical investigation for the preliminary design of the bridge foundations for the proposed replacement structure included two boreholes in the vicinity of the existing Westminster Drive alignment. These boreholes are designated BH11-1 and BH11-2, and their locations are shown on the Borehole Location Plan, Drawing No. 1 in Appendix A.

Prior to carrying out the investigation, Stantec contacted the public utility authorities to clear the borehole locations of both private and public utilities.

The field drilling program was carried out from March 1 through March 26, 2011. The boreholes were advanced using a combination of continuous flight hollow stem augers at shallow depths and N and B casings at deeper locations. Drilling was carried out with a truck-mounted CME 75 drill rig equipped for soil and bedrock sampling. The drilling equipment was owned and operated by DBW Drilling Ltd. of Ajax, Ontario.

The subsurface stratigraphy encountered in each borehole was recorded in the field by an experienced Stantec field technician. Split spoon samples were collected at regularly spaced intervals (every 760 mm for up to 6 m below existing ground surface and every 1.5 m for deeper strata). A pocket penetrometer was used to estimate the undrained shear strength where cohesive soil was encountered. All samples recovered were returned to Stantec’s Ottawa laboratory for detailed classification and testing.

It is noted that during drilling, frequent cobbles and occasional boulders were encountered in both boreholes advanced for this project. Drilling difficulties, due to the presence of highly permeable soils were encountered during the course of the investigation.

After completion of drilling, boreholes were backfilled with a mix of bentonite powder and stone dust (crushed gravel) and sealed with cold asphalt patch.

3.2 LOCATION AND ELEVATION SURVEY

The ground surface elevation at each borehole location was surveyed on March 2, 2011, with reference to a Concrete Monument at 17T 480545 4749568 (Ontario Department of Highways #227). The geodetic elevation of this benchmark, provided by Callon Dietz of London, Ontario, is 264.3 m. Offsets were measured with respect to Westminster Drive centerline. Summary information pertaining to the boreholes included in this report is given in Table 3.1.

Table 3.1: Borehole Information Summary

	Boreholes	
	11-1	**11-2**
MTM Zone 10 Coordinates Northing Easting	 4751747 407749	 4751704 407816
Offset, m	2.7 Rt CL	1.8 Lt CL
Ground Surface Elevation, m	268.0	267.5
Total Depth Drilled, m	40.8	36.9
End of Borehole Elevation, m	227.2	230.6
Depth Augered, m	40.8	36.9
Number of Soil Samples	29	19

Notes: (1) CL = centerline, Rt = right, and Lt = left; offsets are given with respect to the centreline of Westminster Drive; (2) No bedrock coring was carried out at this site.

3.3 LABORATORY TESTING

All samples were taken to our Ottawa laboratory where they were subjected to a detailed visual examination by a Geotechnical Engineer. Routine soil testing was carried out on selected soil samples. The tests carried out included plasticity testing (8 samples), grain size analysis (15 samples) and moisture content testing (44 samples). Three samples were submitted to Parcel Laboratories of Ottawa for analysis of pH, soluble sulphate content, chloride content and resistivity.

Samples remaining after testing will be placed in storage for a period of one year after issuance of the final report. After the storage period, the samples will be discarded unless we are directed otherwise by MTO.

4.0 Subsurface Conditions

4.1 GENERAL

The subsurface conditions observed in the two boreholes included in this report are presented in detail on the Borehole Records provided in Appendix B. An explanation of the symbols and terms used to describe the Borehole Records is also provided in Appendix B.

It is noted that geotechnical investigation results for four boreholes in the immediate vicinity of the existing Westminster Bridge structure were made available to Stantec by MTO (Geocres Report No. 40I14-78). These investigation results are also included in Appendix B. Based on this report, the subsurface condition at the four boreholes included approximately 600 mm to 750 mm thick fill over sandy clay till over glacial sands and gravel over sandy silt (till). The maximum depth of exploration was approximately 11.0 m.

In general, the subsurface stratigraphy consisted of asphalt over roadway and embankment fill material over a silty clay deposit over a sandy silty clay till over a poorly graded sand (with gravel) deposit over till. For the purpose of this report, the subsurface materials encountered at the site can be grouped into the following six stratigraphic regions (zones):

Pavement
Roadway granular fill
Embankment fill
Silty clay
Sandy silty clay till
Sand (medium to coarse)
Glacial till

It is noted that the subsurface profile encountered in the current two boreholes is generally consistent with that from the four boreholes summarized above (Geocres Report No. 40I14-78).

Descriptions of these strata are given below. Borehole location plans and stratigraphic sections of the soils encountered within the boreholes are provided on Drawing No. 1 in Appendix A.

4.2 OVERBURDEN

4.2.1 Pavement

Asphalt pavement was encountered in both boreholes. The observed asphalt thicknesses were 120 to 150 mm.

4.2.2 Roadway Granular Fill

A granular fill material was encountered in both boreholes immediately beneath the asphalt pavement. The thickness of the granular fill was approximately 500 mm, extending to elevations of 267.3 to 267.0 m. The fill was predominantly composed of silty sand with gravel.

4.2.3 Embankment Fill

This fill was encountered in both boreholes immediately beneath the roadway granular fill. The thickness of the embankment fill varied from 3.8 to 5.1 m and extended to bottom elevations of 262.2 to 263.1 m.

The embankment fill was predominantly composed of sandy (lean) clay with varying proportions of sand and trace gravel. The Standard Penetration Test results (SPT N-values) for the embankment fill ranged from 6 to 27 blows/0.3 m indicating a firm to very stiff consistency.

Index tests carried out on two representative samples from this fill yielded the following results:

Gravel:	3%
Sand:	30-32%
Silt:	37-49%
Clay:	18-28%
Moisture Content:	7-20%

Atterberg limits tests carried out on two samples from this layer indicated a plasticity index (PI) ranging from 12 to 17%. The embankment fill material is classified as sandy lean clay (CL) according to the Unified Soil Classification System (USCS).

Representative grain size distribution plot and the corresponding plasticity chart for this fill are given in Figures 1 and 6 in Appendix C, respectively.

4.2.4 Silty Clay

This deposit was encountered in both boreholes immediately beneath the embankment fill. The thickness of the deposit varied from approximately 2.9 to 3.7 m and extended to elevations ranging from 260.2 to 258.6 m.

This deposit was predominantly composed of silty (lean) clay with trace amounts of sand and gravel. The SPT N-values for this deposit ranged from 14 to 46 blows/0.3 m indicating a stiff to hard consistency. Pocket penetrometer testing carried out indicated undrained shear strength measurements of 175 to 300 kPa, indicating a very stiff consistency.

Index tests carried out on four representative samples from this deposit yielded the following results:

Gravel:	0-1%
Sand:	13-20%
Silt:	45-50%
Clay:	29-43%
Moisture Content:	12-17%

Atterberg limits tests carried out on four representative samples from this layer indicated a plasticity index range of 11-18%. The material of this deposit is classified as lean silty clay (CL) according to the USCS.

Representative grain size distribution plot and the corresponding plasticity charts for this fill are given in Figures 2 and 6 in Appendix C, respectively.

4.2.5 Sandy Silty Clay Till

This deposit was encountered in both boreholes immediately beneath the silty clay deposit described above. The thickness of this deposit ranged from 3.1 to 4.2 m and extended to elevations of 257.1 to 254.3 m.

This deposit was composed predominantly of sandy silty clay. Frequent cobbles and boulders were also encountered in this deposit. The SPT N-values for this deposit ranged from 25 to greater than 100 blows/0.3 m indicating a very stiff to hard consistency.

Index tests carried out on one representative sample from this deposit yielded the following results:

Gravel:	5%
Sand:	40%
Silt:	40%
Clay:	15
Moisture Content:	9-19%

Atterberg limits tests carried out on one representative sample from this layer indicated a plasticity index of 6%. The material of this till deposit is classified as sandy silty clay (CL-ML) according to the USCS.

The grain size distribution plot and the corresponding plasticity chart for the sample obtained from this deposit are shown in Figure 3 and 6 in Appendix C, respectively.

4.2.6 Medium to Coarse Sand

This deposit was encountered in both boreholes immediately beneath the sandy silty clay till deposit. The thickness of this sand deposit ranged from 21.3 to 23.9 m and extended to elevations of 235.8 to 230.4 m.

This deposit was predominantly composed of medium to coarse sand with trace amounts of silt and fine gravel. The SPT N-values for this deposit ranged from 19 to greater than 100 blows/0.3 m indicating a compact to very dense state.

Index tests carried out on six representative samples from this deposit yielded the following results:

Gravel:	12-23%
Sand:	64-79%
Fines (silt & clay):	6-13%
Moisture Content:	9-17%

According to the USCS the material from this deposit belongs to a group ranging from well-graded sand with silt and gravel (SW-SM) to poorly graded sand with silt and gravel (SP-SM).

The grain size distribution plots of samples obtained from this deposit are shown in Figure 4 in Appendix C.

4.2.7 Silty Sand with Gravel Till (Glacial Till)

A sandy silt with gravel till layer was encountered in both boreholes immediately beneath the sand deposit described above. In both boreholes drilling was terminated within this layer upon split-spoon refusal and hence the thickness of this deposit was not determined.

The SPT N-values for this deposit ranged from 77 to greater than 100 blows/0.3 m indicating a very dense state. It is noted that occasional cobbles were encountered in this deposit.

Index tests carried out on two representative samples from this deposit yielded the following results:

Gravel:	21-33%
Sand:	47-50%
Fines (silt & clay):	20-30%
Moisture Content:	7-11%

It is noted that for BH11-2, a composite sample of SS-17 and SS-18 was formed to carry out index tests.

Atterberg limits tests carried out on two samples of this deposit produced non-plastic results. This material is classified as silty sand with gravel (SM) according to the USCS. The grain size distribution plots and the plasticity chart of samples obtained from this deposit are shown in Figures 5 and 6 in Appendix C, respectively.

4.3 BEDROCK

Borehole advancement was terminated above the bedrock level. Therefore the depth to bedrock at this site is not known.

4.4 GROUNDWATER

Groundwater level measurement was carried out at the time of drilling in both boreholes. The groundwater levels were not stabilized at the time of measurement, and hence will be referred to as “inferred”.

The inferred groundwater levels are summarized in Table 4.1.

Table 4.1: Inferred Groundwater Levels (Time of Drilling)

Borehole No	Ground Surface Elevation (m)	Groundwater	
		Depth (m)	Elevation (m)
BH11-1	268.0	13.4	254.6
BH11-2	267.5	10.7	256.8

Fluctuations in the groundwater level due to seasonal variations or in response to a particular precipitation event should be anticipated.

4.5 CHEMICAL TESTING

Three representative samples retrieved from variable depths at this site were submitted to Paracel Laboratories in Ottawa, Ontario, for analysis of pH, water soluble sulphates and chloride concentrations, and resistivity. The analysis results are provided in Table 4.2.

Table 4.2: Results of Chemical Analysis

Borehole No	Sample No.	Depth (m)	pH	Chloride (µg/g)	Sulphate (µg/g)	Resistivity (Ohm-m)
BH11-1	SS-9	6.1 to 6.7	7.4	209	6	23
BH11-1	SS-26	33.2 to 33.8	7.7	39	369	18
BH11-2	SS-2	3.1 to 3.7	7.7	827	67	9

5.0 Discussion and Engineering Recommendations

5.1 GENERAL

Project Purpose/Justification

Westminster Drive is carried over Highway 401 by a single-span bridge (Westminster Drive Bridge). Immediately to the north and south of Highway 401, Westminster Drive (east to west corridor) and White Oak Road (south to north corridor) meet at T-intersections on Westminster Drive. Highway 401 is a four lane (two lanes in each direction) divided freeway at the bridge location. Westminster Drive Bridge is a single-span reinforced concrete rigid frame box girder structure. The span of the existing bridge across Highway 401 is approximately 28.8 m. Westminster Drive currently has a single lane in each direction and the traffic is controlled by a "stop condition" located beyond both ends of the bridge structure.

Proposed Underpass Structure

Several alternatives were reviewed and evaluated as part of the proposed structure replacement study. The preliminary alternatives reviewed are provided in Appendix F. It is understood that the alternatives being assessed are to address the following:

- Existing bridge span will not accommodate any further widening of Highway 401 (which is anticipated to have an ultimate eight-lane configuration);
- Four alternatives will likely maintain the existing Westminster Drive horizontal alignment;
- Two alternatives involve removing the existing bridge structure and replacing with a new one approximately 130 m east of current alignment; and
- All the alternatives are anticipated to result in a possible raise of the vertical profile of Westminster Drive (and hence grade raise of approach embankments).

Regardless of the alignment alternatives being considered, it is assumed that the proposed replacement bridge will accommodate four lanes of Highway 401 traffic lanes in each direction.

It is anticipated that the proposed replacement bridge structure will have two spans with a centre pier through the middle of Highway 401 and two integral abutments supported on piles.

5.2 GEOTECHNICAL DESIGN PARAMETERS

The soil conditions encountered at this site generally consist of embankment fill over a silty clay deposit over sandy silty clay till deposit over a medium to coarse sand over a silty sand till. The site soils are generally compact to very dense. Both boreholes in the vicinity of the existing bridge structure were advanced to 3 m past split-spoon refusal to elevations corresponding to 227.2 and 230.6 m.

It is understood that the get-in/get-out (GiGo) bridge design and construction concept is being applied to this project. This involves closing the crossing road and using rapid bridge construction techniques to limit the closure of Westminster Drive to less than two months. As part of the GiGo bridge design approach, integrated caisson/pier units are being considered for the foundations at the existing Highway 401 median, as well as more conventional foundation types.

For preliminary design purposes, the soil profile indicated in Table 5.1 below can be used. The geotechnical soil profile was developed based on the synthesis of the measured N values and the laboratory index test results (including moisture contents) of soil samples retrieved from the site. This profile is indicated in Figure 7 in Appendix D and was developed based on the information obtained from both boreholes BH11-1 through BH11-2.

Table 5.1: Representative Soil Profile for Foundation of Bridge Structure

Elevation (m)		Soil Type	Design Parameters			
From	To		γ	ϕ	S_u	E
267.8	267.1	Silty sand granular FILL	20	35		50
267.1	262.5	Sandy clay FILL (firm to very stiff)	20	-	75	20
262.5	259.0	Silty clay (stiff to hard)	20	-	150	30
259.0	256.0	Sandy silty clay TILL (stiff to hard)	21	-	200	35
256.0	233.0	Sand with silt and gravel (compact to very dense)	21	38	-	100
	< 233.0	Silty sand with gravel TILL (very dense)	22	40	-	200

Note: (1) γ = total unit weight (kN/m^3), ϕ = soil friction angle (°), S_u = undrained shear strength (kPa), and E = soil modulus (MPa).

(2) Groundwater will be assumed to be at approximate elevation of 256.0 m for preliminary design purposes. Submerged unit weight (γ') should be used below the groundwater level.

Cobbles and boulders were present in the silty clay till (elevation 259 - 256 m) whereas cobbles were encountered in the sand and till deposits (below elevation of 256 m).

5.3 FROST PENETRATION

In accordance with OPSD 3090.101, the design frost penetration depth for foundations, f, at the site is 1.2 m. Therefore, footings and pile caps should be provided with a minimum of 1.2 m of soil cover or equivalent insulation for protection against frost heaving.

5.4 SEISMIC DESIGN CONSIDERATIONS

It is recommended that a Soil Profile I as defined in Canadian Highway Bridge Design Code (CHBDC, 2006) Section 4.4.6 be used in the seismic design of this site.

Table A3.1.1 of the CHBDC indicates that the Zonal Acceleration Ratio (ZAR) for London, Ontario, which is approximately 10 km north of the site, is 0.00. A seismic hazard calculation for the site was obtained from Natural Resources Canada (copy attached in Appendix E). It indicates that for this site, the peak ground acceleration (PGA) value corresponding to 10% exceedance in 50 years is 0.043, which is greater than the ZAR for London. Hence, a ZAR of 0.043 should be used for this site.

The potential liquefaction of the site soil under seismic loading conditions was evaluated. The evaluation indicated that liquefaction of the foundation soils is not a concern for this site due to:

(a) very low ZAR,
(b) dense to very dense (stiff to hard) nature of the site soil,
(c) relatively deep groundwater, and
(d) relatively high fraction of fines content within the shallow soils.

Even though it is not likely very significant, seismically induced lateral earth pressures should be considered for this project with a Zonal Acceleration Ratio of 0.043.

5.5 FOUNDATION OPTIONS

For the bridge foundation both shallow and deep foundations options can be considered. Shallow foundations would be placed within the sandy silty clay till and deep foundations extended into the deeper very dense glacial till.

At the bridge abutments, driven piles are being considered as the deep foundation alternative reflecting the preference, from a structural perspective, of constructing integral abutment bridges.

At the center pier location, cast-in-place caissons extending to a concrete beam supporting the girders are being considered as the deep foundation alternative reflecting the proposed GiGo design and construction concept. Based on the General Arrangement drawings the caissons would be laterally unsupported for a length of about 4.0 m.

For the centre pier, driven piles are also considered feasible options.

The Table 5.2 compares the foundation options from a foundations design and constructability perspective:

Table 5.2: Comparison of Foundation Options for Bridge Structure

Option	Advantages	Disadvantages	Relative Cost	Risk/Consequences
Shallow foundation within sandy silty clay till	▪ Excavation and drilling through difficult deposit not needed ▪ Generally suitable to support bridge piers	▪ May necessitate large footing area ▪ Not suitable for integral abutment bridge construction	Low to medium	• Differential settlement
Piles End bearing on Till	▪ Reduced differential settlement ▪ Suitable for integral abutment and the centre pier	▪ Difficulty driving piles through boulders and cobbles, may require pre-augering	Medium	▪ Pile damage during installation ▪ Negative pile interaction if closely spaced
Frictional	▪ Reduced pile length	▪ Pile capacity may not be fully utilized ▪ Difficulty driving piles through cobbles and boulders, may require pre-augering	Medium	▪ Larger settlement ▪ Pile damage during installation
Drilled Caissons	▪ Can transmit very large axial and lateral loads	▪ Difficult to drill though boulders and cobbles ▪ Not suitable for integral bridge abutment	High	▪ Risk of cave-in, especially below groundwater table during drilling ▪ Contractor would need to balance the water pressures and possibly use a drilling mud ▪ Contractor would use a liner to protect the augered caisson walls ▪ Concrete placement within the caisson would be carried out using a tremie operation while the liner is extracted

Note: All options presented in Table 5.2 are suitable for a semi-integral abutment bridge configuration.

Based on the comparison presented above in Table 5.2, a combination of driven piles end bearing on very dense sand or till for the abutment foundation and caissons, driven piles or a shallow foundation for the centre pier will provide a suitable solution for the conditions presented herein. This foundation combination will meet the requirements of the anticipated integral abutment bridge configuration.

5.6 FOUNDATION RECOMMENDATIONS

5.6.1 General

The design recommendations presented in the following sections have been developed in accordance with the requirements and methods described in the Canadian Highway Bridge Design Code (CHBDC, 2006).

5.6.2 Abutment Foundations – Driven Piles

5.6.2.1 Geotechnical Axial Resistance

Anticipated pile loads have not been established yet. However, it is anticipated that a pile foundation consisting of HP310x110 piles will be used to support the proposed integral abutments (north and south of Highway 401). It is anticipated that the underside of the pile caps will be at approximate elevation of 263.0 to 264.0 m. This elevation was based on the assumption that the pile caps will form part of an integral abutment structure.

For this project it is recommended that HP 310x110 piles be designed using a geotechnical resistance at ULS of 1600 kN and at SLS of 1400 kN, with a minimum target pile tip elevation of 235 m geodetic. These recommended geotechnical resistances consider the following:

- Pile drivability analysis, discussed further below, suggest that driving conditions will be excessively difficult if the objective is to drive the HP 310x110 piles to the deeper glacial till where N-values of greater than 100 were observed.
- Static analysis review confirms that the above capacity can be achieved with partial penetration in the coarse to medium sand layer which is over 20 m thick.

The geotechnical resistance at ULS includes a resistance factor of 0.4.

The supply and installation of the piles should be in accordance with the OPSS 903 Construction Specification for Deep Foundations.

Downdrag

It is anticipated that the proposed bridge replacement will require approximately 7.5 m high embankment fill (grade raise in the order of 1 to 1.5 m). The grade raise is anticipated to be completed prior to the installation of piles. The grade raise will cause some settlement of the subsurface soil (see below under the Embankment Settlement Section). At the abutment locations some unloading will occur due to the removal of the portion of the embankment fill in front of the proposed abutment face due to the longer bridge span. The anticipated settlement is expected to be completed within a few days of the completion of the grade raise. The resulting negligible downdrag is not anticipated to have a significant impact on the capacity of the piles.

Relaxation of driven piles

For H-piles driven to the competent (very dense) sand or till layer, relaxation and reduction of pile capacity will not be of a concern.

Drivability

The site soil generally consists of dense embankment fill over very stiff to hard silty clay deposit over a very dense coarse to medium sand deposit over hard glacial till. In addition, occasional cobbles and boulder should be anticipated. As such, the site is expected to pose some resistance to pile driving.

Figure 10 provides the anticipated geotechnical resistance (static analysis) versus depth for an HP 310x110 pile driven at this site.

A geotechnical resistance at ULS of 1600 kN would be achieved with the pile tip at or below elevation 235 m (see Figure 10). Similarly, the resistance at ULS of 1800 kN would be achieved with the pile tip at or below elevation 232 m. It is noted that penetration through the 21 to 24 m thick very dense sand layer is expected to be very difficult.

Figure 11 shows the results of a pile drivability analysis carried out using the GLRWEAP computer program for an HP 310x110 pile driven to elevation 242 m. Figure 11 shows the following:

- DELMAG D30-32 driver was assumed.
- Transferred energy of 50 kJ.
- Pile penetration 20 m, from el. 262 m correspond to a pile tip elevation of 242 m.
- A driving resistance of over 50 blows/25 mm corresponding to an unfactored resistance of 2500 kN or a corresponding ULS value of 1000 kN.
- A driving resistance of over 125 blow/25 mm corresponding to an unfactored resistance of 3500 kN or a corresponding ULS value of 1400 kN.
- A driving resistance of 300 blows/25 mm corresponding to an unfactored resistance of 4000 kN or a corresponding ULS value of 1600 kN.

Based on the above static and drivability analyses the piles should be designed driven to an elevation of approximately 235 m to achieve a design capacity at ULS of 1600 kN. The GLRWEAP analysis suggests that damage would occur to achieve a resistance of 1600 kN; however, MTO experience using the Hiley Formula (MTO SS103-11) during driving control suggest that this capacity would be achievable. Due to the difficult driving conditions at this site, a pile driving analyser would be recommended as part of the pile driving control in conjunction with the use of MTO SS103-11.

<u>Axial resistance in tension</u>

For design against uplift, the tensile resistance provided in Table 5.3 is recommended. This value is based on a minimum pile length of 28 m (elevation of approximately 235 m or deeper).

Table 5.3: Recommended Tensile Pile Resistance

Pile Type	Minimum Pile Length(m)	Factored Geotechnical Resistance (Tension) at ULS (kN)
310 x 110	28	800

A resistance factor, Φ, of 0.3 has been applied to ULS resistance. The factored geotechnical resistance (tension) at ULS provided above does not include the own weight of the pile.

5.6.2.2 Geotechnical Lateral Resistance

The geotechnical resistance of the pile against lateral loads is mobilized due to the passive resistance of the surrounding soil. Assessed values for horizontal passive resistance and geotechnical resistances at SLS for the proposed pile can be generated from information provided in Table C6.4 of the Commentaries to the Canadian Highway Bridge Design Code (CHBDC, 2006). A value at ULS of 120 kN and a value at SLS of 50 kN may be used for an HP 310x110 pile. It should be noted that a horizontal displacement at the ground surface of 10 mm was assumed for the SLS condition.

The passive lateral resistance for vertical piles can be calculated according to Sections C6.8.7.1 and C6.8.7.2 of the Canadian Highway Bridge Design Code (CHBDC, 2006). The resistance can be calculated with the unfactored soil parameters presented in Table 5.4 below. The till layer was conservatively assumed to have zero cohesion.

The lateral capacity of piles was evaluated using the program called LPile Plus v6.0 developed by Ensoft, Inc. (Ensoft, 2010). The input parameters are given in Table 5.4. A moment of inertia of 237×10^6 mm^4 was used for a 310x110 pile section. A modulus of elasticity of 200 GPa was used for the pile material (steel). The pile was modelled with a total length of 28 m and embedment length of 1.5 m into the competent till. The p-y modulus values were based on values suggested by Ensoft, Inc. (Ensoft, 2010).

Table 5.4: Recommended Parameters for Lateral Pile Capacity Evaluation

Soil Layer	Depth Range (m)		Unit weight, γ	Friction angle, ϕ	Undrained shear strength, S_u	p-y Modulus, k
	From	To	kN/m^3	Degrees	kPa	kN/m^3
Loose sand[(1)] in CSP	0.0	3.0	20	33	-	5,400
Silty clay	3.0	4.5	20	-	150	ε_{50}=0.005[(3)]
Sandy silty clay TILL	4.5	7.5	21[(2)]	-	200	ε_{50}=0.005[(3)]
Sand to silty sand	7.5	30.5	21[(2)]	38	-	34,000
Glacial TILL	-	>30.5	22[(2)]	40	-	34,000

Notes:

(1) This layer represents the loose uniform sand filled around the pile in the CSP.
(2) Submerged unit weight should be used below groundwater level.
(3) For clay, the strain corresponding to one-half the principal stress difference, ε_{50} = 0.005, was specified in the model.

Two plots from LPILE are presented in Figures 8 and 9 in Appendix D. Figure 8 shows the deformed shape of the pile for lateral (shear) force ranging between 50 and 120 kPa. This plot indicates that the pile head undergoes negligible lateral deflection for the conditions modeled herein.

Figure 9 presents the p-y plot that gives the non-linear response of the pile-soil interaction. It provides a series of curves obtained from program LPILE generated for selected depths below the pile head. These plots can be used in the structural evaluation of the proposed bridge founded on H-piles.

Group action of piles (pile interaction) for lateral loading should be considered if centerline spacing of piles is less than 8 pile diameters (or least lateral dimension of pile) parallel to the direction of lateral load, or less than 4 pile diameters, perpendicular to the load. The effect of interaction between piles can be considered by applying a reduction factor to the coefficient of lateral subgrade reaction (p-y modulus). The following reduction factors may be used to account for pile group action:

Table 5.5: Recommended Reduction Factors for Pile Groups

Pile spacing / pile diameter	Reduction Factor	Pile spacing / pile diameter	Reduction Factor
Load Parallel to Pile Spacing		Load Perpendicular to Pile Spacing	
7	1.0	4	1.0
4	0.8	3	0.9
3	0.7	2	0.75
2	0.6	-	-

5.6.3 Centre Pier Foundation

5.6.3.1 Caissons

5.6.3.1.1 Axial Resistance

It is understood that concrete caisson foundations are being considered as the preferred option to support the centre pier of the proposed Underpass structure. The caissons will tie into the pier columns and as such would act as partially embedded piles. No pile caps would be required at the ground surface for the centre pier. The caissons are anticipated to be 1000 to 1500 mm in diameter. The unsupported length of pier columns are assumed to be approximately 4.0 m.

Figure 12 provides the anticipated geotechnical resistance at ULS (static analysis) versus depth for concrete caissons with diameters of 1.0, 1.2 and 1.5 m.

It is recommended that the caisson be drilled to a minimum of 3 m into the compact to very dense sand (approximate elevation of 253 m). Figure 12 indicates that a 1.2 m diameter caisson drilled into the sand at elevation 253 m will have a factored geotechnical resistance at ULS of 3900 kN. A 1.0 m diameter caisson will have a geotechnical resistance at ULS of 2800 kN at elevation 253 m. A 1.5 m diameter caisson would have a factored geotechnical resistance at ULS of 8400 kN at elevation 253 m. It is noted that this caisson tip elevation reflects the minimum embedment depth within the dense sand required to provide the specific geotechnical resistances given above.

The supply and installation of the caissons should be according to the OPSS 903 Construction Specification for Deep Foundations.

5.6.3.1.2 Bending and Buckling of Partially Embedded Caissons

As noted above, the caissons and centre pier columns are assumed to act together as partially embedded piles, with approximately 4.0 m of unsupported section above the ground surface. It is anticipated that the structural bending and buckling analysis of the partially embedded piles will be carried out using the equivalent depth-to-fixity design method (Davisson and Robinson, 1965). Based on the soil conditions within the upper 9 m, an approximate horizontal modulus of subgrade reaction, kh, of 15 to 30 MPa/m was considered for design purposes. Based on this soil property, the following depths-to-fixity may be used for design purposes.

Table 5.6: Calculated Depth-to-Fixity

Caisson Diameter (m)	Assumed Stiffness (EI) ($N.m^2$)	Minimum Required Embedment Depth (m)	Depth-to Fixity (m)
1.0	1325	10.5	4.5
1.2	2748	12.5	5.0
1.5	6710	14.0	5.0

Note that the stiffness (EI) values are based on uncracked concrete section and a modulus of elasticity (E) of 27,000 MPa for concrete.

5.6.3.1.3 Geotechnical Lateral Resistance

The preliminary geotechnical lateral resistance at ULS and SLS of a caisson with an unsupported length of 4.0 m was evaluated using the p-y modeling approach. The results suggest the following.

Table 5.7: Estimated Caisson Lateral Resistance at SLS

Caisson Diameter (m)	Unsupported Length (m)	Caisson Supported Length (m)	Lateral Resistance at SLS
1.0	4.0	9.0	170 kN
1.2	4.0	9.0	250 kN
1.5	4.0	9.0	400 kN

Note: The SLS values are based on 10 mm of deflections at the top of the 4.0 m unsupported length of caisson and assume a constant caisson diameter above and below ground surface.

The SLS values assumed that the caisson heads are free to rotate.

Table 5.8: Estimated Caisson Lateral Resistance at ULS

Caisson Diameter (m)	Unsupported Length (m)	Caisson Supported Length (m)	Lateral Resistance at ULS
1.0	4.0	9.0	355 kN
1.2	4.0	9.0	525 kN
1.5	4.0	9.0	675 kN

Note: The ULS values include a resistance factor of ULS of 0.5.

The following caisson properties were assumed in the p-y modeling analyses.

1.0 m caisson

- 32 MPa concrete, 19 mm stone, and 75 mm concrete cover on rebars
- 16 three-bar bundles of #10 US Std reinforcing steel

1.2 m caisson

- 32 MPa concrete, 19 mm stone, and 75 mm concrete cover on rebars
- 19 three-bar bundles of #10 US Std reinforcing steel

1.5 m caisson

- 32 MPa concrete, 19 mm stone, and 75 mm concrete cover on rebars
- 24 three-bar bundles of #10 US Std reinforcing steel

These values are preliminary and will need to be re-evaluated using a p-y modeling approach once the unsupported pile length is confirmed, the pile stiffness properties are defined, and boreholes are drilled at the pier location.

Soil Spring Modelling

Linear-elastic springs can be developed to model the geotechnical soils such that the springs stiffness provided would produce a 10 mm deflection under a lateral load corresponding to the SLS values listed in Table 5.7. Typically, a table of spring stiffness values can be provided with 1 ft or 1 m spring spacing, depending on the structural engineering software being used. If this type of analysis is proposed, a table of stiffness values will be generated using the p-y model at the spacing frequency and format required by the structural engineer for the specific caisson diameter and stiffness proposed.

5.6.3.2 Driven Piles

Driven piles involving a single row of HP 310x110 piles are also suitable options for the proposed centre pier foundation. Driven piles, if selected, will avoid drilling within the Highway 401 median and hence will likely reduce the construction time compared to that of caissons.

The piles would be similar to that for the abutments as described above for abutment foundations. At the location of the centre pier, an HP310x110 pile driven to elevation 235 m (approximate pile length of 25 m) will have a factored geotechnical resistance at ULS of 1600 kN and a corresponding SLS value of 1400 kN.

It is understood that a short section of the piles for the centre pier will extend above the frost elevation to provide a suitable connection with precast concrete elements of the centre pier; however, the bottom of the precast concrete element will need to be set below the frost line.

5.6.3.3 Shallow Foundations

Shallow foundation is also being considered as a possible option for supporting the centre pier. This section provides relevant foundation recommendations for the proposed centre pier footing.

For this option, it is anticipated that the centre pier will be founded on a shallow foundation on native soil at approximate elevation of 260.5 m (frost penetration depth at the center of Highway 401). This footing elevation would be at least 1.2 m below the existing ground elevation.

5.6.3.4 Geotechnical Vertical Resistance

The geotechnical resistances provided in Table 5.9 below may be used in the design provided the footings are placed on undisturbed native soil as described above.

Table 5.9: Geotechnical Resistance for Shallow Foundation (Spread Footing)

Founding Element	Founding Elev. (m)	Footing Width (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Resistance at SLS (kPa)
Bridge pier footing	260.5 m (Silty Clay)	1.5 to 6.0	375	300
	259.0 (Sandy Silty Clay Till)	1.5 to 6.0	500	425

In accordance with Section 6.6.2 of the CHBDC, a resistance factor of 0.5 has been applied in calculating the factored geotechnical resistance at Ultimate Limit State (ULS).

It is noted that for evaluation of the geotechnical resistance at ULS given in Table 5.6, the representative soil properties provided in Table 5.1 were used for the analysis. The groundwater was conservatively assumed to be immediately beneath the proposed founding elevation.

The geotechnical resistance at Serviceability Limit State (SLS) corresponds to a maximum settlement of 25 mm.

5.6.3.5 Geotechnical Horizontal Resistance (Sliding)

The unfactored horizontal resistance of spread footings made of cast-in-place concrete placed on native soil may be calculated using an unfactored coefficient of friction of 0.55. Since this is an unfactored value, a factor of 0.8 should be used in accordance with Table 6.1 of the CHBDC.

The unfactored horizontal resistance of spread footings may be calculated using the following unfactored coefficients of friction:

- 0.55 between OPSS Granular A and cast-in-place concrete
- 0.4 between silty clay and cast-in-place concrete
- 0.5 between sandy silty clay till and cast-in-place concrete
- 0.3 between a precast concrete footings and a thin layer of uncompacted leveling sand

In accordance with Table 6.1 of the CHBDC, a resistance factor against sliding of 0.8 should be applied to obtain the resistance at ULS.

5.7 LATERAL EARTH PRESSURES

5.7.1 Backfill

It is recommended that the backfill within and behind structures for the proposed bridge replacement consist of Approved earth material placed and compacted using methods and equipment appropriate to the type of structure. For the purpose of this preliminary report, it is assumed that a backfill material meeting the requirements of OPSS Gran B Type I or Gran A and Gran B Type II material will be used. The surface of the backfill will be assumed to be horizontal.

5.7.2 Static Lateral Earth Pressures

Static lateral earth pressures will need to be considered in the design of abutments, retaining walls (wingwalls) and retained soil systems (if any).

The bridge abutments should be backfilled with granular material in accordance with OPSD 3101.150.

Computation of earth pressures should be in accordance with Section 6.9 of the CHBDC. For retaining walls that are designed to allow rotation, active earth pressure may be used for design. For rigidly tied and unyielding structures, the at-rest earth pressure should be used for design. The unfactored soil parameters provided in Table 5.10 may be used for design of walls with a horizontal backfill. The effects of compaction should be accounted for by applying a compaction surcharge as shown in Figure 6.6 of the CHBDC.

The total active (P_A) and passive (P_P) thrusts can be calculated using the following equations:

$$P_A = \frac{1}{2} K_a \gamma H^2$$

$$P_P = \frac{1}{2} K_p \gamma H^2$$

where H is the height of the wall and γ is the unit weight of the backfill soil. Values for K_a, K_p, K_o and γ are provided below. The thrust acts at a point one third up the height of the wall.

Table 5.10: Recommended Non-Seismic Earth Pressure Parameters (Horizontal Backfill)

Parameter	OPSS Gran B Type I	OPSS Gran A and Gran B Type II
Bulk Unit Weight, γ (kN/m^3)	21.2	22.0
Effective Friction Angle	32º	35º
Coefficient of Earth Pressure at Rest (K_o)	0.47	0.43
Coefficient of Active Earth Pressure (K_a)	0.31	0.27
Coefficient of Passive Earth Pressure (K_p)	3.2	3.7

5.7.3 Seismic Lateral Earth Pressures

The low zonal acceleration ratio for this site suggests that the lateral earth pressures on the bridge due to seismic loads will likely be negligible. The following design parameters are provided should the bridge abutment and wingwalls also be designed to resist the earth pressures induced under seismic loading conditions. The seismic earth pressures may be calculated using the parameters detailed in Table 5.11 below.

The total active and passive thrusts under seismic loading conditions can be calculated using the following equations:

- $P_{AE} = \frac{1}{2} K_{AE} \gamma H^2 (1 - k_V)$
- $P_{PE} = \frac{1}{2} K_{PE} \gamma H^2 (1 - k_V)$

where:

- K_{AE} = active earth pressure coefficient (combined static and seismic)
- K_{PE} = passive earth pressure coefficient (combined static and seismic)
- H = height of wall
- k_h = horizontal acceleration coefficient
- k_v = vertical acceleration coefficient
- γ = total unit weight

For this site, the following design parameters were used to develop the recommended K_{AE} and K_{PE} values. A site specific Seismic Hazard Calculation sheet prepared by Natural Resources

Canada is provided in Appendix E. For transportation structures the PGA value corresponding to a 10% probability of exceedance in 50 years is typically selected.

- Zonal Acceleration Ratio, A or PGA 0.05
- Horizontal Acceleration Coefficient, k_h 0.025 yielding 0.075 non-yielding
- Vertical Acceleration Coefficient, k_v 0.017 yielding 0.05 non-yielding
- Horizontal Backslope to Wall 0°
- Vertical Back of Wall 0°

The above k_h value corresponds to ½ of the A value for yielding walls and 1.5 times for non-yielding walls. The k_v value corresponds to 0.67 of the k_h value. The angle of friction between the soil and the wall has been set at 0° to provide a conservative estimate.

Table 5.11: Recommended Seismic Earth Pressure Parameters (Horizontal Backfill)

Parameter	OPSS Gran B Type I		OPSS Gran A and Gran B Type II	
Bulk Unit Weight, γ (kN/m^3)	21.2		22.0	
Effective Friction Angle	32º		35º	
	Yielding wall	Non-yielding	Yielding wall	Non-yielding
Active Earth Pressure (K_{AE})	0.32	0.35	0.28	0.31
Height of Application of P_{AE} from base as a ratio of wall height, (H)	0.341	0.356	0.340	0.358
Passive Earth Pressure, (K_{PE})	3.25	3.11	3.69	3.54
Height of Application of P_{PE} from base as a ratio of wall height, (H)	0.325	0.306	0.325	0.307

5.8 EMBANKMENTS

5.8.1 Embankment Construction

The proposed new configuration requires embankments to be built north and south of the bridge structure. It is anticipated that the fill material for the new embankment will be identical to the embankment fill encountered during the geotechnical investigation. This material consisted of compact sand with silt and gravel.

It is noted that any embankment widening associated with the grade raise should be carried out in accordance with OPSD 208.010 Benching of Earth Slopes.

5.8.2 Stability of Slopes

A slope stability evaluation was carried out for a typical cross-section through the embankment. The evaluation was carried out using a commercial program Slope/W (Geo-Slope, 2010).

A typical cross-section through the embankment along with the stability evaluation results for static and seismic loading conditions is provided in Figures 13a and 13b in Appendix D. The results of both the static and seismic slope stability evaluation indicate that for the anticipated configuration, a 2H:1V embankment slope will be required.

A design groundwater level at elevation 256.0 m was selected for analysis.

5.8.3 Embankment Settlement

Settlement of the underlying soil due to the embankment has been assessed based on a simple geometry including 2H:1V side slopes, a height of 7.5 m, and a 15 m wide platform. The maximum fill height of 7.5 m reflects a maximum 1.5 m grade raise over the existing embankment fill. The new 15 m wide embankment platform has been assumed to be centered over the existing 9 m wide embankment platform. The following assumptions were made in evaluating the settlement of the site soil under the proposed embankment:

- Typical soil profile given Table 5.1 (profile south of Highway 401);
- The load from the bridge abutments will be transferred to deeper and more competent strata by the piles (other than that by the centre pier) and hence will not contribute to the settlement of the site soil;
- Both immediate (elastic) settlement (for non-cohesive soils) and consolidation settlement (for cohesive soils) were considered;
- A Poisson's ratio of 0.35 will be used for all soil types;
- The maximum embankment height of 7.5 m (in the immediate vicinity of the bridge abutment);
- The embankment extends approximately 200 m north and south from the abutment;

Evaluation of soil settlement due to the above was assessed using simple elastic theory and stress distribution under embankment loading.

The analysis result indicates that for the conditions presented herein, the maximum total vertical settlement of the existing soil in the vicinity of the bridge abutment is approximately 13 mm under an SSM or Earth Borrow embankment constructed with 2H:1V side slopes. The maximum settlement will take place approximately 20 m back from each bridge abutments; at the proposed abutment, little settlement is anticipated due to the anticipated unloading to remove the portion of the existing embankment in front of the proposed abutment face. This settlement is anticipated to take place relatively rapidly and is expected to be completed during construction of the embankment.

Self-weight settlement due to compression of the embankment fill during the construction process is expected to be less than 5 mm given that the anticipated maximum grade raise is only 1.5 m. This settlement is expected to be completed almost immediately after the fill has achieved its full height.

5.9 PRELIMINARY CONSTRUCTION CONSIDERATIONS

5.9.1 Construction Staging

The existing Westminster Drive Bridge will be closed during construction of the proposed replacement bridge. No construction staging is anticipated during the proposed bridge replacement.

5.9.2 Excavation and Backfilling

Excavation backfill for the new bridge structure should be carried out in accordance with OPSS 902 Construction Specification for excavation and Backfilling – Structures.

For embankment widening, benching of earth slopes should be carried out in accordance with OPSD 208.010 Benching of Earth Slopes.

Native site soil encountered during geotechnical investigation predominantly included very stiff to hard silty clay over sand to sandy silt over hard till deposit. The soils encountered at the site may be classified in accordance with the OHSA as follows:

Existing Embankment Fills	Type 3 Soil
Silty Clay and Sandy Silty Clay Till	Type 2 Soil

Any vegetation, fill, organic soils and other deleterious materials must be removed from beneath proposed pier footing, the pile cap and embankment. Where deleterious materials are encountered, the materials should be excavated, removed and replaced. The lateral extent of such excavation should include all deleterious material within the influence zone of the embankments.

Grading work should be carried out in accordance with OPSS 206 Construction Specification for Grading and SP 206S03.

Any side slopes for open cut excavations should conform to Occupational Health and Safety Act regulations for Construction Projects (OHSA).

5.9.3 Unwatering

Groundwater was encountered at elevation of approximately 256.0 m, which is higher than the anticipated founding elevation of the proposed caisson foundation for the centre pier. The sand deposit at this location is expected to be highly permeable and hence unwatering of the caisson excavation using conventional sump and pump techniques is not considered appropriate.

Construction of the caisson will require a lined caisson hole. This requires that concrete will be tremied under water. An NSSP will be required to alert the contractor that a tremied approach to concrete placement is anticipated.

5.9.4 Reuse of Excavated Material

The material near the ground surface in the vicinity of the project site consists of clayey material. This material will not be suitable as backfill within and behind the structures for the proposed replacement bridge. However, it may be used as embankment fill if proper placement and compaction procedure is followed.

5.10 CEMENT TYPE AND CORROSION POTENTIAL

Two samples of the native soil were submitted to Paracel Laboratories in Ottawa, Ontario for analysis of pH, water soluble sulphate and chloride concentrations, and resistivity. The testing was completed to determine the potential for degradation of the concrete in the presence of soluble sulphates and the potential for corrosion of exposed steel used in foundations and buried infrastructure. The analysis results together with the results for the sample from the bottom part of the fill are summarized in the Table 4.1.

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. The soluble sulphate concentrations for the two samples were 6 and 369 µg/g. Soluble sulphate concentrations less than 1000 µg/g generally indicate that a low degree of sulphate attack is expected for concrete in contact with soil and groundwater. Type GU (General Use) Portland Cement should therefore be suitable for use in concrete at this site.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The soil pH values were 7.4 and 7.7 which are within what is considered the normal range for soil pH of 5.5 to 9.0. The pH levels of the tested soil do not indicate a highly corrosive environment. The test results provided in the Table 4.1 may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects.

5.11 FUTURE INVESTIGATIONS

The recommendations provided herein were based on geotechnical investigation carried out within the general area of the existing bridge structure for preliminary design purposes. Once the final locations of the proposed structure foundations are identified, additional geotechnical investigations should be carried out at these locations to enable detailed recommendations for the proposed structure foundation.

6.0 Specifications

The following specifications are referenced in this report:

Table 6.1: Specifications Referenced in Report

Document	Title
OPSD 208.010	Benching of Earth Slopes
OPSD 3090.101	Foundation Frost Depths for Southern Ontario
OPSD 3101.150	Walls – Abutment, Backfill – Minimum Granular Requirement
OPSS 206	Construction Specification for Grading
OPSS 902	Construction Specification for Excavation and Backfilling - Structures
OPSS 903	Construction Specification for Deep Foundations
SP 206S03	Earth Excavation, Grading

7.0 References

CGS. 2006. Canadian Foundation Engineering Manual, 4th Edition. Canadian Geotechnical Society.

Chapman, L.J., and Putnam, D.F. 1984. The physiography of Southern Ontario, Ontario Geological Survey Special Volume 2. Ontario Research Foundation, Toronto, Ontario.

CHBDC, 2006. Canadian Highway Bridge Design Code. Canadian Standards Association, Mississauga, Ontario.

Davisson, M.T., and Robinson, K.E. 1965. Bending and Buckling of Partially Embedded Piles, Proc. 6th International Conference on Soil Mechanics and Foundation Engineering, Montreal, Canada, pp. 243-246.

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GEO-SLOPE International Ltd. 2010. Stability Modeling with SLOPE/W 2010©. Calgary, AB.

Rocscience, 2009. Settle3D Settlement and Consolidation Analysis: Theory Manual, Rocscience, Inc.

8.0 Miscellaneous

The field work was carried out under the supervision of Jeff Forrester, CET, Senior Technologist, under the direction of Simon Gudina, Ph.D., P.Eng., Geotechnical Engineer.

MultiVIEW Locates Inc. of Mississauga, Ontario, carried out the private and public utility locates for the boreholes.

The drilling equipment was supplied and operated by DBW of Ajax, Ontario. Traffic control was provided by On Track Safety of Thornhill, Ontario.

Geotechnical laboratory testing was carried out at the Stantec Ottawa laboratory. Chemical testing on soil samples was carried out by Paracel Laboratories in Ottawa.

This report was prepared by Simon Gudina, Ph.D., P.Eng. and reviewed by Raymond Haché, M.Sc., P.Eng., MTO Designated Principal Contact.

9.0 Closure

The recommendations made in this report are in accordance with our present understanding of the project. We request that we be permitted to review our recommendations when the drawings and specifications are complete.

A soil investigation is a limited sampling of a site. The conclusions given herein are based on information gathered at the specific borehole locations. Should any conditions at the site be encountered which differ from those at the borehole locations, we request that we be notified immediately in order to assess the additional information and its effects on the above recommendations.

We trust the information presented herein meets your present requirements. Should you have any questions or require additional information, please do not hesitate to contact us.

This report has been prepared by Simon Gudina. Technical review was carried out by Raymond Haché.

Respectfully submitted,

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APPENDIX A

Drawings No. 1 – Borehole Location Plan and Soil Strata Plot

Site Photographs

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

PLATE No
CONT
WP 3070–09–00
SHEET

HIGHWAY 401
WESTMINSTER DRIVE, LONDON, ONTARIO
BOREHOLE LOCATIONS & SOIL STRATA

Stantec

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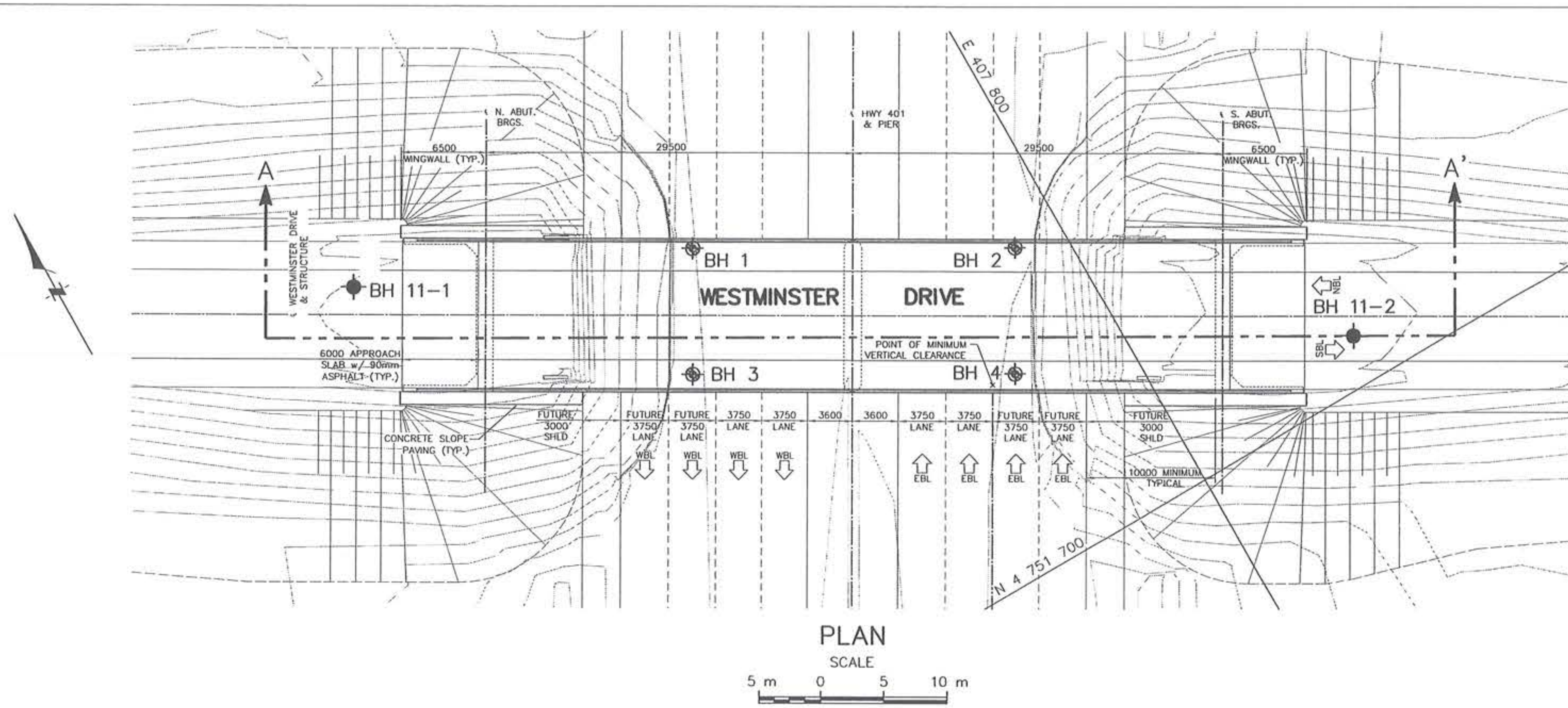
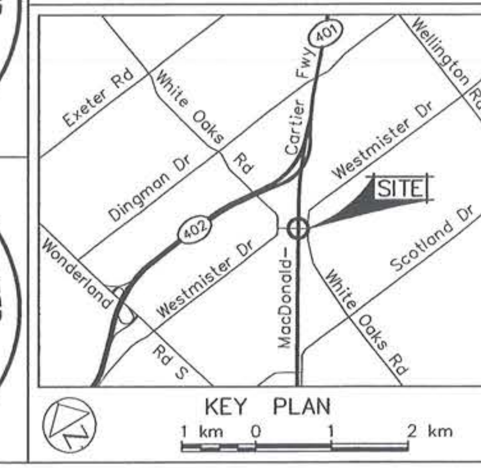
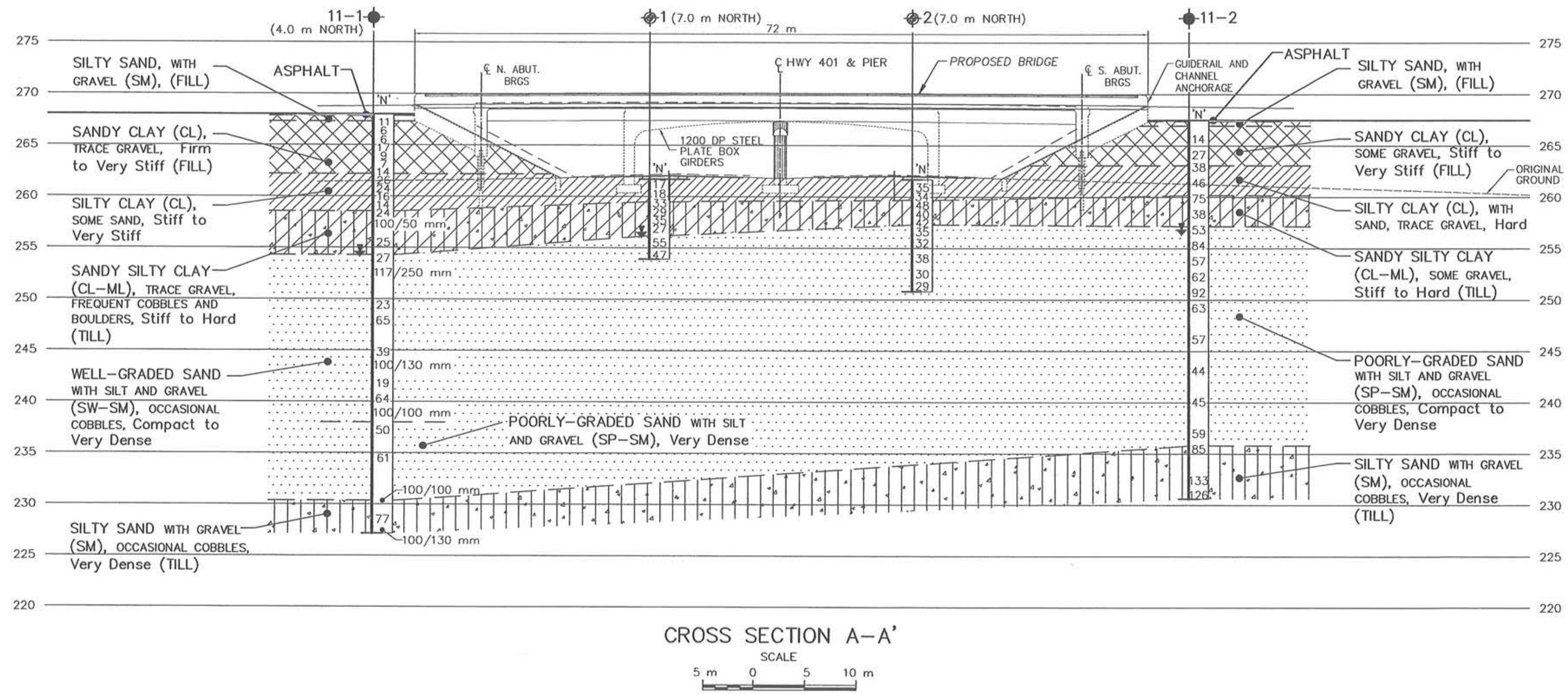


PLAN

SCALE 5 m 0 5 10 m





CROSS SECTION A–A'

SCALE 5 m 0 5 10 m

LEGEND

⊕	Borehole (by Stantec)
⊕	Borehole (by others)
N	Blows/0.3m (Std Pen Test, 475 J/blow)
↓	WL at time of investigation March 2011
(m NORTH)	Offset from Cross Section Line in metres

No	ELEVATION	MTM ZONE 11 COORDINATES NORTH	EAST
11–1	268.0	4 751 747.2	407 748.7
11–2	267.5	4 751 703.8	407 816.0
1	261.7	4 751 736.3	407 773.6
2	261.7	4 751 723.4	407 796.0
3	261.6	4 751 727.8	407 768.6
4	261.7	4 751 714.8	407 791.0

NOTES

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

This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 100.

REVISIONS DATE	BY	DESCRIPTION
12/09/20	GBB	FOR REVIEW

GEOCRES No 40I14–150

HWY No 401			DIST
SUBM'D SG	CHECKED	DATE 2011–05–26	SITE 19–366
DRAWN GBB	CHECKED	APPROVED SG	DWG 1

DRAWING NAME: 165000776–1_MAY2012.dwg
CREATED BY: GBB MODIFIED: 2012/06/08
T:\Autocad\Drawings\Project Drawings\2012\165000776\Geotech\White Oak, London\Plan & XSection\White Oak Road, London\September\165000776–1_May2012.dwg (BW) Printed: Sep 20, 2012

Photo No. 1: Westminster Drive looking north at Highway 401 (Google Earth Pro® Image).

Photo No. 2: Westminster Drive looking south at Highway 401 (Google Earth Pro® Image).

Photo No. 3: Looking east from BH11-1 on Westminster Drive along Hwy 401

Photo No. 4: Looking west from BH11-2 on Westminster Drive along Hwy 401

APPENDIX B

Symbols and Terms Used on Borehole Records

Borehole Records

Borehole Records from a Previous Investigation (Geocres Report No. 40I14-78)

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

Topsoil	- mixture of soil and humus capable of supporting vegetative growth
Peat	- mixture of visible and invisible fragments of decayed organic matter
Till	- unstratified glacial deposit which may range from clay to boulders
Fill	- material below the surface identified as placed by humans (excluding buried services)

Terminology describing soil structure:

Desiccated	- having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
Fissured	- having cracks, and hence a blocky structure
Varved	- composed of regular alternating layers of silt and clay
Stratified	- composed of alternating successions of different soil types, e.g. silt and sand
Layer	- > 75 mm in thickness
Seam	- 2 mm to 75 mm in thickness
Parting	- < 2 mm in thickness

Terminology describing soil types:
The classification of soil types are made on the basis of grain size and plasticity in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 or D 2488). The classification excludes particles larger than 76 mm (3 inches). The USCS provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

Terminology describing cobbles, boulders, and non-matrix materials (organic matter or debris):
Terminology describing materials outside the USCS, (e.g. particles larger than 76 mm, visible organic matter, construction debris) is based upon the proportion of these materials present:

Trace, or occasional	Less than 10%
Some	10-20%
Frequent	> 20%

Terminology describing compactness of cohesionless soils:
The standard terminology to describe cohesionless soils includes compactness (formerly "relative density"), as determined by the Standard Penetration Test N-Value (also known as N-Index). A relationship between compactness condition and N-Value is shown in the following table.

Compactness Condition	SPT N-Value
Very Loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very Dense	>50

Terminology describing consistency of cohesive soils:
The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by *in situ* vane tests, penetrometer tests, or unconfined compression tests.

Consistency	Undrained Shear Strength	
	kips/sq.ft.	kPa
Very Soft	<0.25	<12.5
Soft	0.25 - 0.5	12.5 - 25
Firm	0.5 - 1.0	25 - 50
Stiff	1.0 - 2.0	50 – 100
Very Stiff	2.0 - 4.0	100 - 200
Hard	>4.0	>200

ROCK DESCRIPTION

Terminology describing rock quality:

RQD	Rock Mass Quality
0-25	*Very Poor*
25-50	*Poor*
50-75	*Fair*
75-90	*Good*
90-100	*Excellent*

Rock quality classification is based on a modified core recovery percentage (RQD) in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be due to close shearing, jointing, faulting, or weathering in the rock mass and are not counted. RQD was originally intended to be done on NW core; however, it can be used on different core sizes if the bulk of the fractures caused by drilling stresses are easily distinguishable from *in situ* fractures. The terminology describing rock mass quality based on RQD is subjective and is underlain by the presumption that sound strong rock is of higher engineering value than fractured weak rock.

Terminology describing rock mass:

Spacing (mm)	Joint Classification	Bedding, Laminations, Bands
> 6000	*Extremely Wide*	-
2000-6000	*Very Wide*	*Very Thick*
600-2000	*Wide*	*Thick*
200-600	*Moderate*	*Medium*
60-200	*Close*	*Thin*
20-60	*Very Close*	*Very Thin*
<20	*Extremely Close*	*Laminated*
<6	-	*Thinly Laminated*

Terminology describing rock strength:

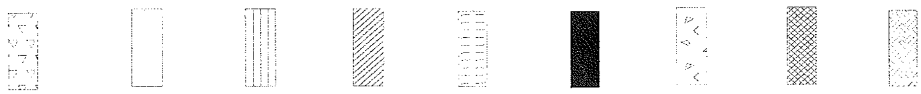
Strength Classification	Unconfined Compressive Strength (MPa)
Extremely Weak	< 1
Very Weak	1 – 5
Weak	5 – 25
Medium Strong	25 – 50
Strong	50 – 100
Very Strong	100 – 250
Extremely Strong	> 250

Terminology describing rock weathering:

Term	Description
Fresh	No visible signs of rock weathering. Slight discolouration along major discontinuities
Slightly Weathered	Discolouration indicates weathering of rock on discontinuity surfaces. All the rock material may be discoloured.
Moderately Weathered	Less than half the rock is decomposed and/or disintegrated into soil.
Highly Weathered	More than half the rock is decomposed and/or disintegrated into soil.
Completely Weathered	All the rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact.

STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



Boulders Cobbles Gravel	Sand	Silt	Clay	Organics	Asphalt	Concrete	Fill	Bedrock

SAMPLE TYPE

SS	Split spoon sample (obtained by performing the Standard Penetration Test)
ST	Shelby tube or thin wall tube
DP	Direct-Push sample (small diameter tube sampler hydraulically advanced)
PS	Piston sample
BS	Bulk sample
WS	Wash sample
HQ, NQ, BQ, etc.	Rock core samples obtained with the use of standard size diamond coring bits.

WATER LEVEL MEASUREMENT

▼ measured in standpipe, piezometer, or well

▽ inferred

RECOVERY

For soil samples, the recovery is recorded as the length of the soil sample recovered. For rock core, recovery is defined as the total cumulative length of all core recovered in the core barrel divided by the length drilled and is recorded as a percentage on a per run basis.

N-VALUE

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 140 pound (64 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (305 mm) into the soil. For split spoon samples where insufficient penetration was achieved and N-values cannot be presented, the number of blows are reported over sampler penetration in millimetres (e.g. 50/75). Some design methods make use of N value corrected for various factors such as overburden pressure, energy ratio, borehole diameter, etc. No corrections have been applied to the N-values presented on the log.

DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to A size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone one foot (305 mm) into the soil. The DCPT is used as a probe to assess soil variability.

OTHER TESTS

S	Sieve analysis
H	Hydrometer analysis
k	Laboratory permeability
γ	Unit weight
G_s	Specific gravity of soil particles
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct Shear
C	Consolidation
Q_u	Unconfined compression
I_p	Point Load Index (I_p on Borehole Record equals $I_p(50)$ in which the index is corrected to a reference diameter of 50 mm)

	Single packer permeability test; test interval from depth shown to bottom of borehole
	Double packer permeability test; test interval as indicated
	Falling head permeability test using casing
	Falling head permeability test using well point or piezometer

Stantec

RECORD OF BOREHOLE No BH 11-1

METRIC

W.P. 3070-09-00 | LOCATION Westminster Drive Underpass N: 4 751 747 E: 407 749 | ORIGINATED BY JF

DIST ___ HWY 401 | BOREHOLE TYPE Hollow stem augers, N casing, B casing, Splitspoons | COMPILED BY JF

DATUM Geodetic | DATE 2011 03 01 - 2011 03 16 | CHECKED BY SG

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) | SHEAR STRENGTH kPa: O UNCONFINED, ✖ FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100) | PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30 | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS GR SA SI CL
268.0	Asphalt				
267.9 / 0.0	150 mm ASPHALT				
0.2	Silty sand with gravel (SM), FILL Brown	1	BS		
267.3 / 0.7	Sandy clay (CL), trace gravel, FILL Firm to very stiff Greyish brown	2	SS	11	
		3	SS	6	3 32 37 28
		4	SS	6	
	@ ~3.1 m b.g.s: some gravel	5	SS	17	
		6	SS	9	
		7	SS	7	3 30 49 18
		8	SS	14	
262.2 / 5.8	SILTY CLAY (CL), some sand Stiff to very stiff Reddish brown to brownish grey	9	SS	26	PP = 280 kPa
		10	SS	24	0 13 48 39 PP = 300 kPa
		11	SS	16	
		12	SS	14	0 13 45 43 PP = 175 kPa
258.6 / 9.5	Sandy silty clay (CL-ML), trace gravel, TILL Stiff to hard Reddish brown	13	SS	24	

Elevation scale: 267, 266, 265, 264, 263, 262, 261, 260, 259

Continued Next Page

✖ 3, × 3: Numbers refer to Sensitivity   O 3% STRAIN AT FAILURE

Stantec

RECORD OF BOREHOLE No BH 11-1

METRIC

W.P. 3070-09-00 | LOCATION Westminster Drive Underpass N: 4 751 747 E: 407 749 | ORIGINATED BY JF

DIST ___ HWY 401 | BOREHOLE TYPE Hollow stem augers, N casing, B casing, Splitspoons | COMPILED BY JF

DATUM Geodetic | DATE 2011 03 01 - 2011 03 16 | CHECKED BY SG

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Sandy silty clay (CL-ML), trace gravel, TILL Stiff to hard Reddish brown *(continued)* -frequent cobbles and boulders	14	SS	100/ 50mm		257		
		15	SS	25		256		5 40 40 15
					∇	255		
254.3 / 13.7	Well-graded SAND WITH SILT AND GRAVEL (SW-SM) Compact to very dense Brownish grey to grey - occasional cobbles	16	SS	27		254		
		17	SS	117/ 250mm		253		
						252, 251, 250		
		18	SS	23		249		

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, ✖ FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³

Continued Next Page

✖ 3, × 3 Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

Stantec

RECORD OF BOREHOLE No BH 11-1

METRIC

W.P. 3070-09-00 LOCATION Westminster Drive Underpass N: 4 751 747 E: 407 749 ORIGINATED BY JF

DIST HWY 401 BOREHOLE TYPE Hollow stem augers, N casing, B casing, Splitspoons COMPILED BY JF

DATUM Geodetic DATE 2011 03 01 - 2011 03 16 CHECKED BY SG

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Well-graded SAND WITH SILT AND GRAVEL (SW-SM) Compact to very dense Brownish grey to grey - occasional cobbles *(continued)*	19	SS	65		15 79 (6)
					247	
					246	
		20	SS	39	245	
		21	SS	100/ 130mm	244	
					243	
		22	SS	19	242	12 78 (10)
		23	SS	64	241	
					240	
		24	SS	100/ 100mm	239	
238.0	@ ~30 m b.g.s: Poorly graded sand with silt and gravel (SP-SM), very dense					

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, ✖ FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³.

Continued Next Page

✖³, ×³: Numbers refer to Sensitivity O³% STRAIN AT FAILURE

Stantec

RECORD OF BOREHOLE No BH 11-1

METRIC

W.P. 3070-09-00 | LOCATION Westminster Drive Underpass N: 4 751 747 E: 407 749 | ORIGINATED BY JF

DIST ____ HWY 401 | BOREHOLE TYPE Hollow stem augers, N casing, B casing, Splitspoons | COMPILED BY JF

DATUM Geodetic | DATE 2011 03 01 - 2011 03 16 | CHECKED BY SG

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
30.0	Well-graded SAND WITH SILT AND GRAVEL (SW-SM) Compact to very dense Brownish grey to grey - occasional cobbles *(continued)*									
			25	SS	50			o		15 73 (12)
							237			
							236			
							235			
			26	SS	61					
							234			
							233			
							232			
							231			
230.4 37.6	Silty sand with gravel (SM), TILL Very dense Grey - occasional cobbles		27	SS	100/ 100mm					
							230	o		
							229			
			28	SS	77			o		33 47 (20)

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED, ✖ FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

Continued Next Page

✖ 3, × 3: Numbers refer to Sensitivity   O 3% STRAIN AT FAILURE

Stantec

RECORD OF BOREHOLE No BH 11-1

METRIC

W.P. 3070-09-00 LOCATION Westminster Drive Underpass N: 4 751 747 E: 407 749 ORIGINATED BY JF

DIST HWY 401 BOREHOLE TYPE Hollow stem augers, N casing, B casing, Splitspoons COMPILED BY JF

DATUM Geodetic DATE 2011 03 01 - 2011 03 16 CHECKED BY SG

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES – NUMBER	SAMPLES – TYPE	SAMPLES – "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, ✖ FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L – WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty sand with gravel (SM), TILL Very dense Grey - occasional cobbles *(continued)*		29	SS	100/ 130mm				o		
227.2 40.8	End of Borehole										

ONTARIO MTO STANTEC 165000776-A - HIGHWAY 401 LONDON.GPJ ONTARIO MOT.GDT 12/9/20

✖³, ×³: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

Stantec

RECORD OF BOREHOLE No BH 11-2

METRIC

W.P. 3070-09-00	LOCATION Westminster Drive Underpass	N: 4 751 704 E: 407 816	ORIGINATED BY JF
DIST ___ HWY 401	BOREHOLE TYPE Hollow stem augers, N casing, B casing, Splitspoons		COMPILED BY JF
DATUM Geodetic	DATE 2011 03 16 - 2011 03 25		CHECKED BY SG

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	NATURAL MOISTURE CONTENT w (%)	PLASTIC LIMIT W_P / LIQUID LIMIT W_L	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
267.5	Asphalt													
267.4 / 0.1	120 mm ASPHALT Silty sand with gravel (SM), FILL Brown						267							
267.0 / 0.6	Sandy clay (CL), some gravel, FILL Stiff to very stiff Brown to grey						266							
			1	SS	14		265	○						
			2	SS	27		264							
263.1 / 4.4	SILTY CLAY (CL), with sand, trace gravel Hard Greyish brown		3	SS	38		263	○	●—●		1	18	45	36
			4	SS	46		262, 261	○	●—●		1	20	50	29
260.2 / 7.3	Sandy silty clay (CL-ML), some gravel, TILL Stiff to hard Grey to brown		5	SS	75		260, 259	○						
			6	SS	38		258	○						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED ✖ FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 20 40 60 80 100

WATER CONTENT (%): 10 20 30

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

✖ 3, × 3: Numbers refer to Sensitivity   ○ 3% STRAIN AT FAILURE

Stantec

RECORD OF BOREHOLE No BH 11-2

METRIC

W.P. 3070-09-00 LOCATION Westminster Drive Underpass N: 4 751 704 E: 407 816 ORIGINATED BY JF

DIST HWY 401 BOREHOLE TYPE Hollow stem augers, N casing, B casing, Splitspoons COMPILED BY JF

DATUM Geodetic DATE 2011 03 16 - 2011 03 25 CHECKED BY SG

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
257.1 / 10.4	Poorly graded SAND WITH SILT AND GRAVEL (SP-SM), Compact to very dense, Brownish grey to grey, - occasional cobbles	7	SS	53	∇	257	
						256	
		8	SS	84		255	
		9	SS	57		254	21 68 (11)
						253	
		10	SS	62		252	
		11	SS	92		251	
						250	
		12	SS	63		249	
						248	

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, ✖ FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10 20 30); UNIT WEIGHT γ kN/m³

Continued Next Page

✖³, ×³: Numbers refer to Sensitivity O³% STRAIN AT FAILURE

ONTARIO MTO STANTEC 165000776-A - HIGHWAY 401 LONDON.GPJ ONTARIO MOT.GDT 12/9/20

Stantec

W.P. 3070-09-00		LOCATION	Westminster Drive Underpass	N: 4 751 704 E: 407 816	ORIGINATED BY JF
DIST	HWY 401	BOREHOLE TYPE	Hollow stem augers, N casing, B casing, Splitspoons		COMPILED BY JF
DATUM Geodetic		DATE	2011 03 16 - 2011 03 25		CHECKED BY SG

ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, ✖ FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Poorly graded SAND WITH SILT AND GRAVEL (SP-SM), Compact to very dense, Brownish grey to grey, - occasional cobbles *(continued)*						247				
			13	SS	57		246		o (~7)		
							245				
							244				
			14	SS	44		243		o (~11)		23 64 (13)
							242				
							241				
			15	SS	45		240		o (~12)		13 74 (13)
							239				
							238				

Continued Next Page

✖³, ×³: Numbers refer to Sensitivity   O³%: STRAIN AT FAILURE

ONTARIO MTO STANTEC 165000776-A - HIGHWAY 401 LONDON.GPJ ONTARIO MOT.GDT 12/9/20

Stantec

RECORD OF BOREHOLE No BH 11-2

METRIC

W.P. 3070-09-00 | LOCATION Westminster Drive Underpass N: 4 751 704 E: 407 816 | ORIGINATED BY JF

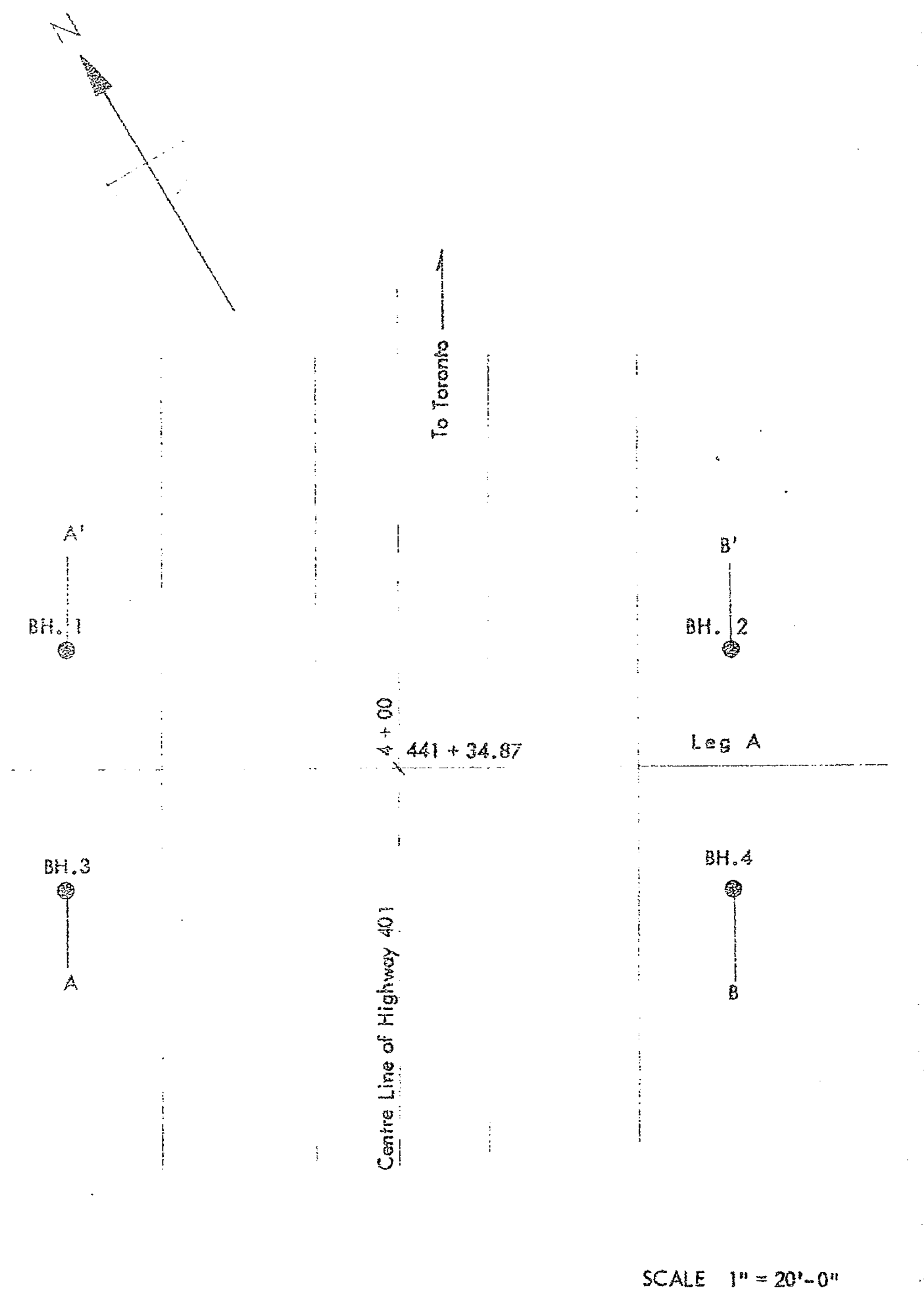
DIST | HWY 401 | BOREHOLE TYPE Hollow stem augers, N casing, B casing, Splitspoons | COMPILED BY JF

DATUM Geodetic | DATE 2011 03 16 - 2011 03 25 | CHECKED BY SG

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	NATURAL MOISTURE CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Poorly graded SAND WITH SILT AND GRAVEL (SP-SM), Compact to very dense, Brownish grey to grey, - occasional cobbles *(continued)*	16	SS	59	237	~12	
235.8 / 31.7	Silty sand with gravel (SM), TILL, Very dense, Grey, - occasional cobbles	17	SS	85	236–235	~10	21 50 24 6* Non-Plastic
		18	SS	133	233–232	~10	21 50 24 6* Non-Plastic
		19	SS	126	231	~12	
230.6 / 36.9	End of Borehole						*A composit sample of SS-17 and SS-18 was tested.

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, ✖ FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; GROUND WATER CONDITIONS; STRAT PLOT.

✖3, ×3: Numbers refer to Sensitivity   O 3% STRAIN AT FAILURE



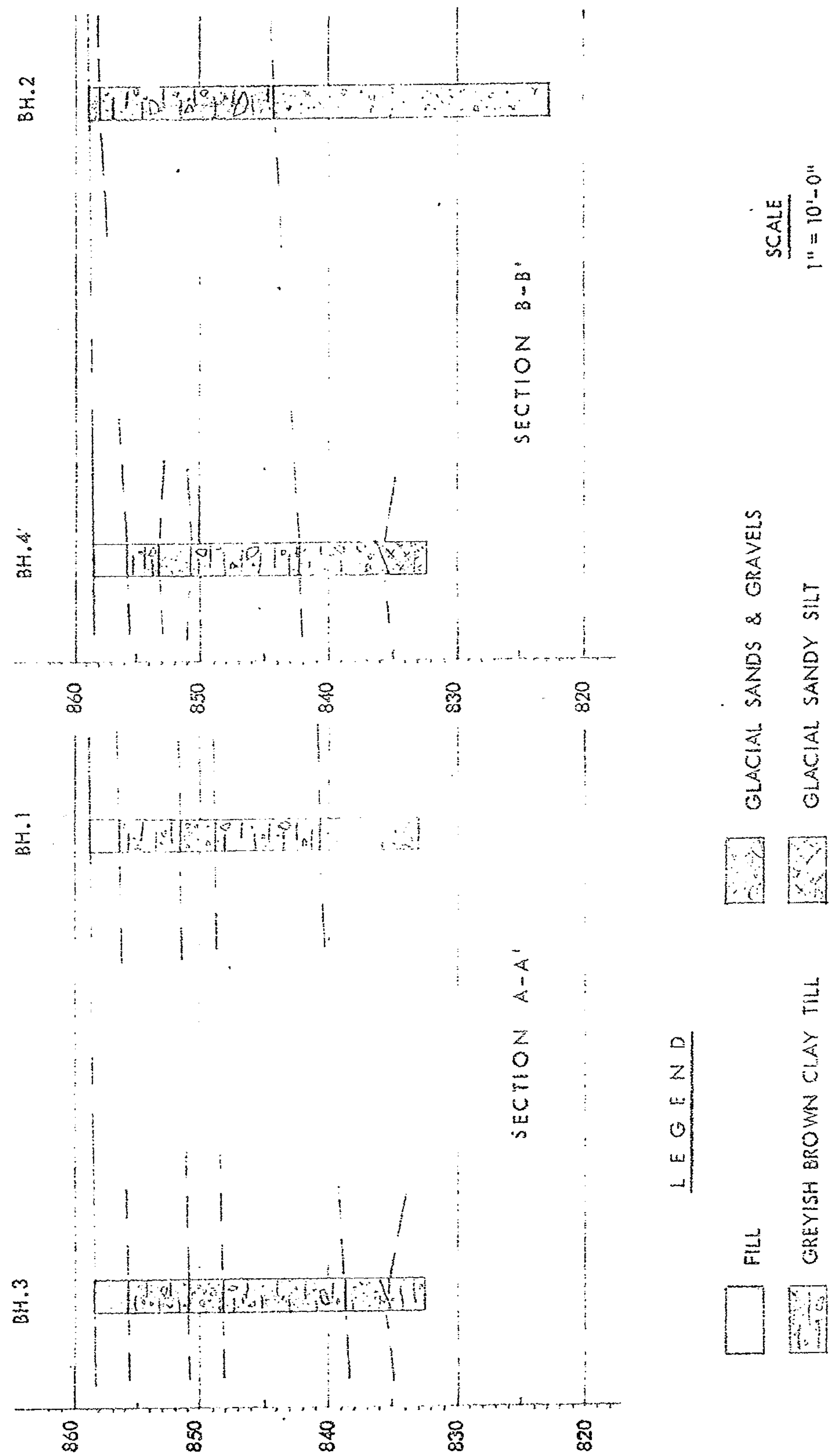
This sketch is a copy of section of plan N° E-3287-1, W.P. 99-57, supplied by D.H.O.

PROJECT Westminster Township, Bridge N° 13, County of Middlesex, Ont.

TITLE Borehole Location Plan

DRG NO. 1 ORDER NO. T.272/57

UNIVERSAL GEOTECHNIQUE LIMITED



PROJECT Westminster Township, Bridge N° 13, County of Middlesex, Ont.

TITLE Borehole Sections

DRG. NO. 2 ORDER NO. T.272/57

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GEOTECHNIQUE
LIMITED

UNIVERSAL **GEOTECHNIQUE** LIMITED

SOIL MECHANICS LABORATORY

BOREHOLE LOG

PROJECT Westminster Township, Bridge N° 13, County of Middlesex, Ontario. ORDER NO. T.272/57

CLIENT Department of Highways, Ontario.

BOREHOLE NO. BH. 1 DIAMETER 2-1/2" CASING 2-1/2"

BOREHOLE LOCATION See Plan INCLINATION Vertical BEARING ———

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
	858.67			Zero			
Dark brown sandy loam with traces of organic matter. FILL.				1'-0"			
Clay, sand and gravel. Probably FILL.				2'-6"			
Very stiff light brown sandy CLAY with generally fine subangular gravel.			● 1			17	Damp. High dry strength.
do			● 2			18	do
				7'-3"			
Dense brown fine to coarse SAND with fine gravel, generally subangular.			● 3			33	Wet No dry strength.
				10'-0"			
Very stiff brownish grey very sandy CLAY with fine to medium subangular gravel.			● 4			29	Moist. High dry strength.
Hard do			● 5			35	Damp High dry strength.
Very stiff do			● 6	Free Water		27	Moist High dry strength.
				18'-0"			
Dense brown fine to coarse SAND.			● 7			55	Wet No dry strength.
				22'-6"			
Dense brown fine to coarse SAND and fine to medium GRAVEL, subangular to subrounded.			● 8	25'-9"		47 (9")	do
			End of Borehole				

SCALE: 1" = 5'-0" ● DISTURBED SAMPLE ▮ UNDISTURBED SAMPLE

UNIVERSAL **GEOTECHNIQUE** LIMITED

SOIL MECHANICS LABORATORY

BOREHOLE LOG

PROJECT: Westminster Township, Bridge Nº 13, County of Middlesex, Ontario. ORDER NO. T.272/57

CLIENT: Department of Highways, Ontario.

BOREHOLE NO.: BH.2 DIAMETER: 2-1/2" CASING: 2-1/2"

BOREHOLE LOCATION: See Plan INCLINATION: Vertical BEARING: ———

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
Brown sand and gravel. FILL.	858.74			Zero 0'-8"			
Hard brown somewhat dessicated sandy CLAY with fine to medium subangular gravel.			⚬ 1			35	Damp High dry strength.
do			⚬ 2	5'-6"		34 (9")	Wet
With pockets of sand and gravel.				6'-9"			
Hard brownish grey very sandy CLAY with fine to large gravel.			⚬ 3			48 (6")	Moist. High dry strength High N due to large gravel.
Hard brownish grey sandy CLAY with fine to medium subangular gravel, some iron stained fissures.			⚬ 4			40	Damp. High dry strength.
do			⚬ 5			42	do
Dense brown fine to coarse SAND with pockets of grey iron stained very sandy clay containing fine gravel.			⚬ 6	14'-6" Free Water		35	Damp Low dry strength.
Dense greyish brown fine silty SAND.			⚬ 7	18'-9"		32	Wet. Low to medium dry strength.
Dense brown fine to coarse SAND with fine gravel, generally subangular.			⚬ 8	21'-6"		38	Wet No dry strength.
do Gravel fine to medium.			⚬ 9			30	do
Firm to dense greyish brown fine silty SAND.			⚬ 10	32'-0" 36'-0" End of Borehole		29	Wet. Low dry strength.

SCALE: 1" = 5'-0" ⚬ DISTURBED SAMPLE ▮ UNDISTURBED SAMPLE

UNIVERSAL **GEOTECHNIQUE** LIMITED

SOIL MECHANICS LABORATORY

BOREHOLE LOG

PROJECT Westminster Township, Bridge N° 13, County of Middlesex, Ontario. ORDER NO. T.272/57

CLIENT Department of Highways, Ontario.

BOREHOLE NO. BH.3 DIAMETER 2-1/2" CASING 2-1/2"

BOREHOLE LOCATION See Plan INCLINATION Vertical BEARING ———

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
	858.44			Zero			
Sand and gravel. FILL.				0'-8"			
Brown very sandy clay with gravel. Probably FILL.				2'-6"			
Very stiff brown sandy slightly dessicated CLAY with fine to medium gravel.			• 1			20	Damp. High dry strength.
				4'-9"			
Very stiff brown sandy silty CLAY with fine to medium gravel, iron stained fissures.			• 2			22	do
				7'-3"			
Loose brown fine to coarse SAND and fine GRAVEL, little clay.			• 3			13	Wet Medium dry strength.
				10'-0"			do
Dense SAND and fine GRAVEL.			• 4	11'-0"		35	Damp
Very stiff brown sandy CLAY with fine to medium subangular gravel.							High dry strength.
Hard brownish grey do			• 5			40	Damp High dry strength.
do			• 6			42	do
					Free Water		
				19'-6"			
Hard brown clayey SAND with pockets of fine to medium sand and silty clay.			• 7			40 (9")	Moist Medium dry strength.
Hard grey very sandy CLAY with occasional gravel.			• 8	25'-9"		41 (9")	Damp. Medium to high dry strength.
			End of Borehole				

SCALE: 1" = 5'-0" • DISTURBED SAMPLE ▮ UNDISTURBED SAMPLE

UNIVERSAL **GEOTECHNIQUE** LIMITED

SOIL MECHANICS LABORATORY

BOREHOLE LOG

PROJECT Westminster Township, Bridge N° 13, County of Middlesex, Ontario. ORDER NO. T.272/57

CLIENT Department of Highways, Ontario.

BOREHOLE NO. BH.4 DIAMETER 2-1/2" CASING 2-1/2"

BOREHOLE LOCATION See Plan INCLINATION Vertical BEARING ——

DESCRIPTION OF STRATA	ELEVATION	LEGEND	SAMPLE	DEPTH	THICKNESS	N	REMARKS
Clay, sand and gravel. FILL.	858.54			Zero 0'-10"			
Brown sandy clay with gravel. Probably FILL.				2'-6"			
Hard brown sandy CLAY with fine to medium subangular gravel.			⊙ 1			34	Damp High dry strength.
				5'-0"			
Dense fine to coarse SAND.			⊙ 2			33 (9")	Damp No dry strength.
				7'-6"			
Hard greyish brown sandy CLAY with fine to medium subangular gravel.			⊙ 3			36	Damp High dry strength.
do			⊙ 4			32	do
do			⊙ 5			29	do
do			⊙ 6	16'-0"		27	do Damp. Medium to high dry strength.
Firm to dense grey fine silty somewhat clayey SAND with occasional fine to medium gravel.				19'-0"	Free Water		
Dense brown fine to coarse SAND and fine to medium GRAVEL, generally subangular.			⊙ 7			40	Moist No dry strength.
Dense grey sandy SILT with layers of grey sandy gravelly clay. Gravel generally fine.			⊙ 8	26'-0"		48	Moist. Low dry strength.
			End of Borehole				

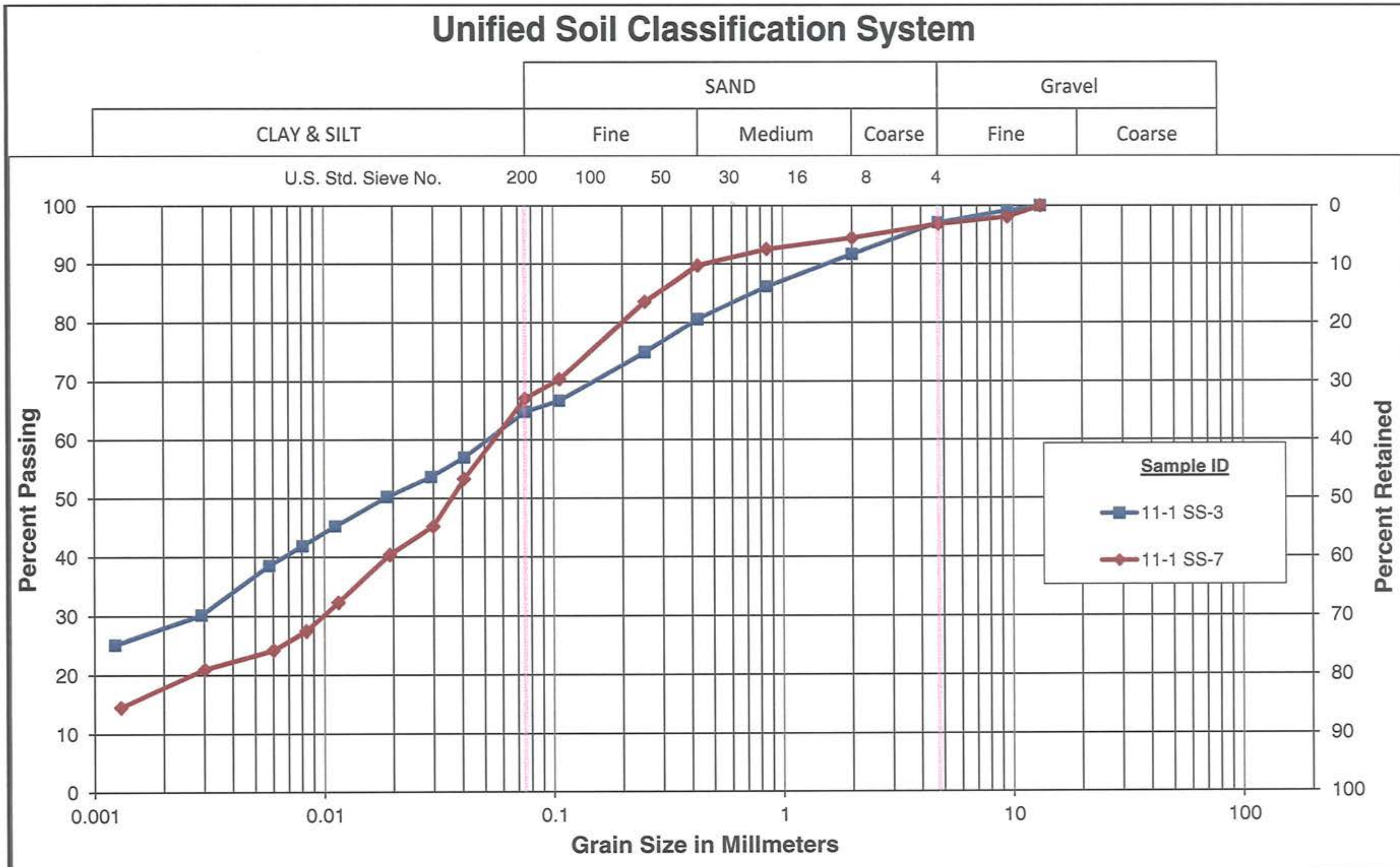
SCALE: 1" = 5'-0" ⊙ DISTURBED SAMPLE ▮ UNDISTURBED SAMPLE

APPENDIX C

Laboratory Test Results

Figures 1 – 5: Grain Size Distribution Plots

Figure 6: Plasticity Chart



GRAIN SIZE DISTRIBUTION

Embankment Fill: Sandy Clay (CL)

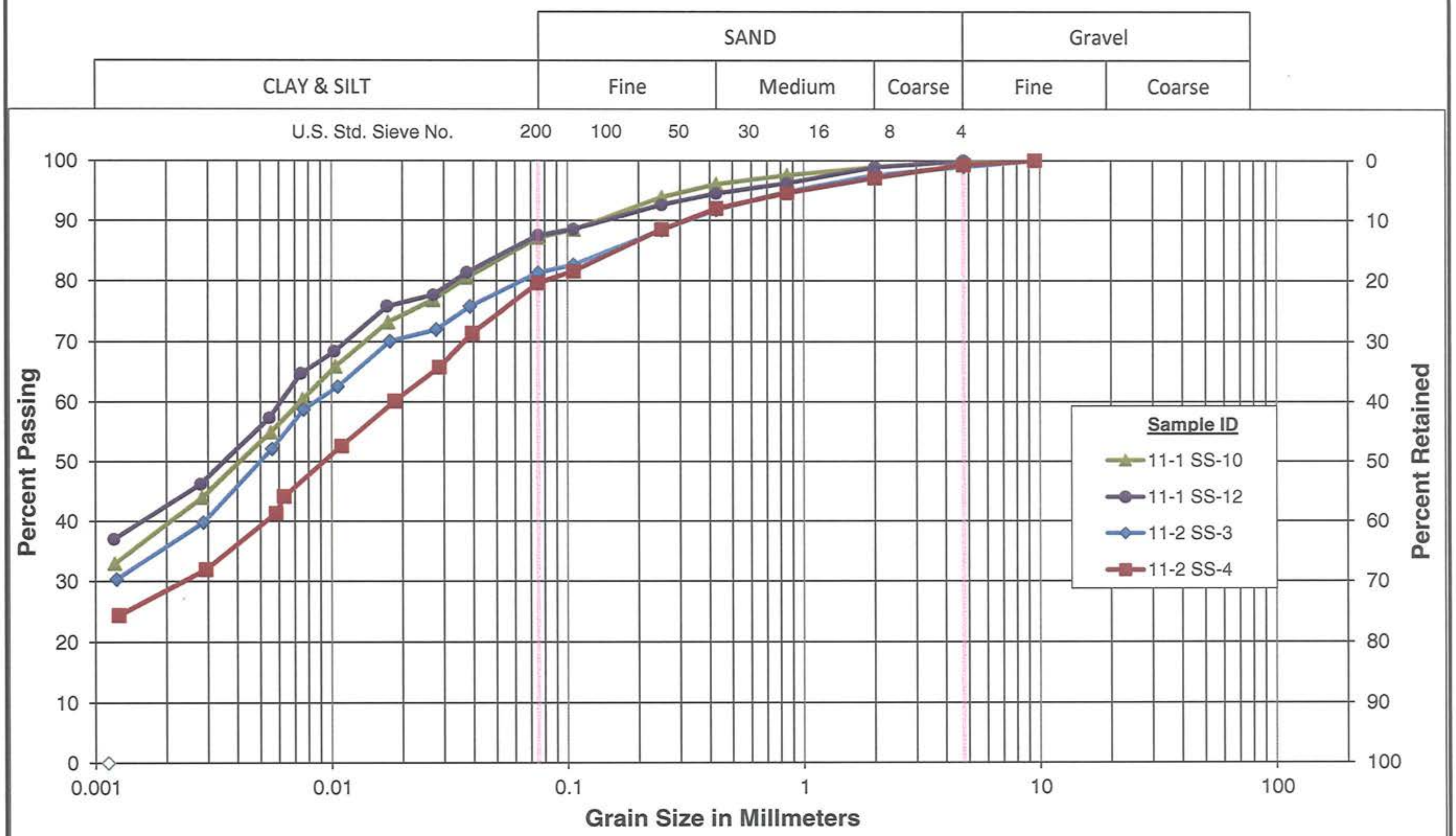
Figure No. 1

Project No. 165000776

GWP No. 3070-09-00

Stantec

Unified Soil Classification System



Stantec

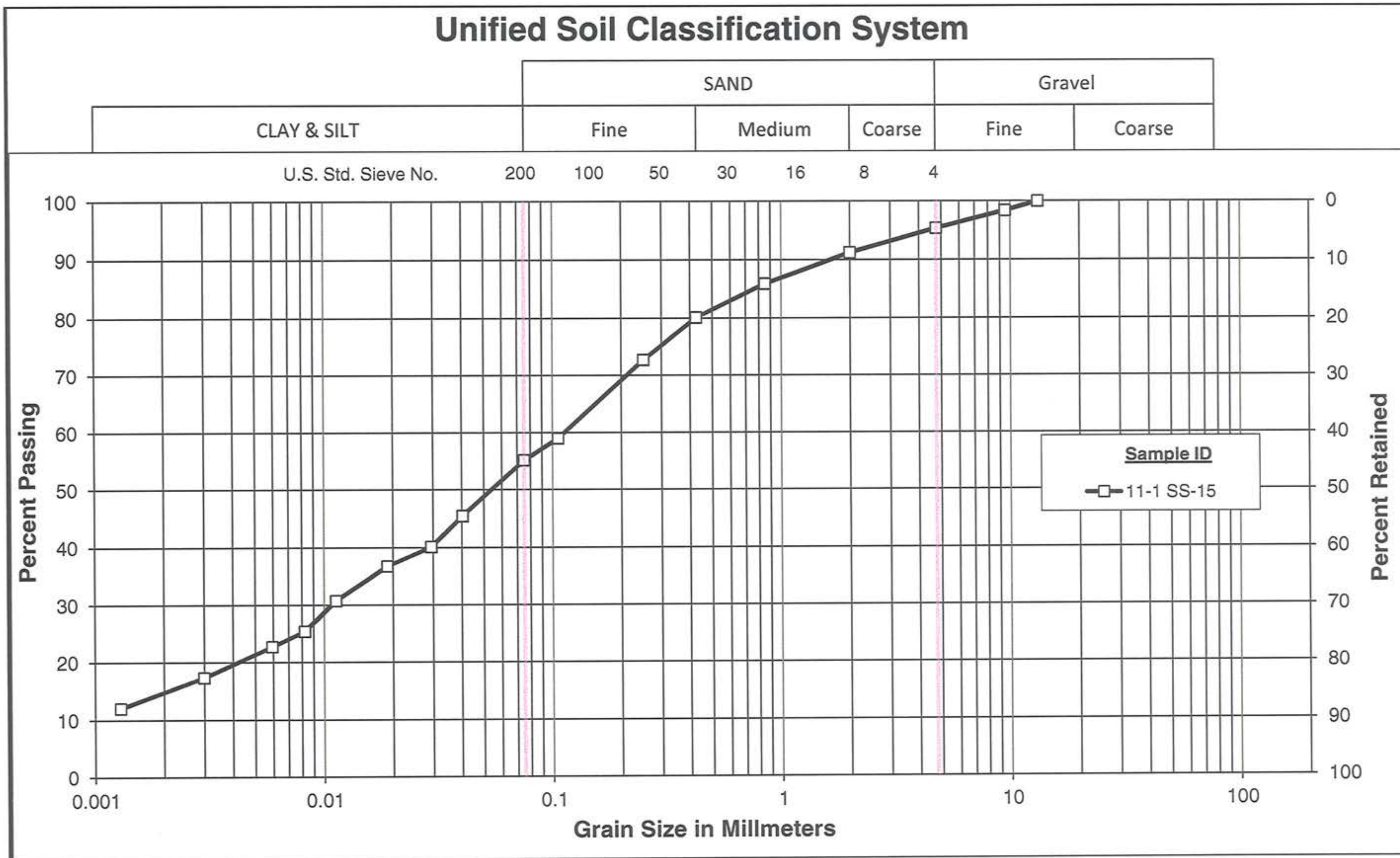
GRAIN SIZE DISTRIBUTION

Silty Clay (CL)

Figure No. 2

Project No. 165000776

GWP No. 3070-09-00



GRAIN SIZE DISTRIBUTION

Sandy Silty Clay (CL-ML) (TILL)

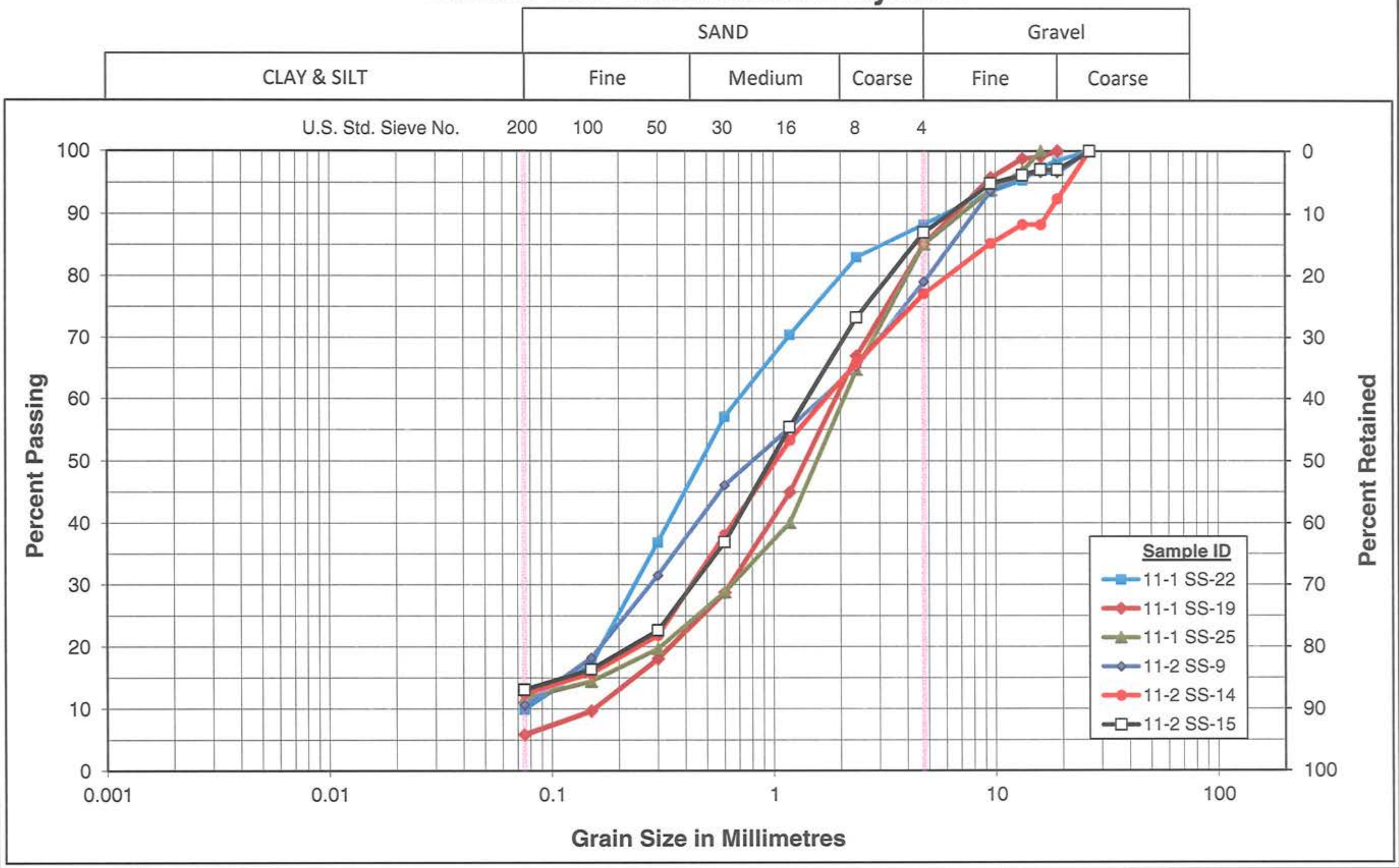
Figure No. 3

Project No. 165000776

GWP No. 3070-09-00

Stantec

Unified Soil Classification System

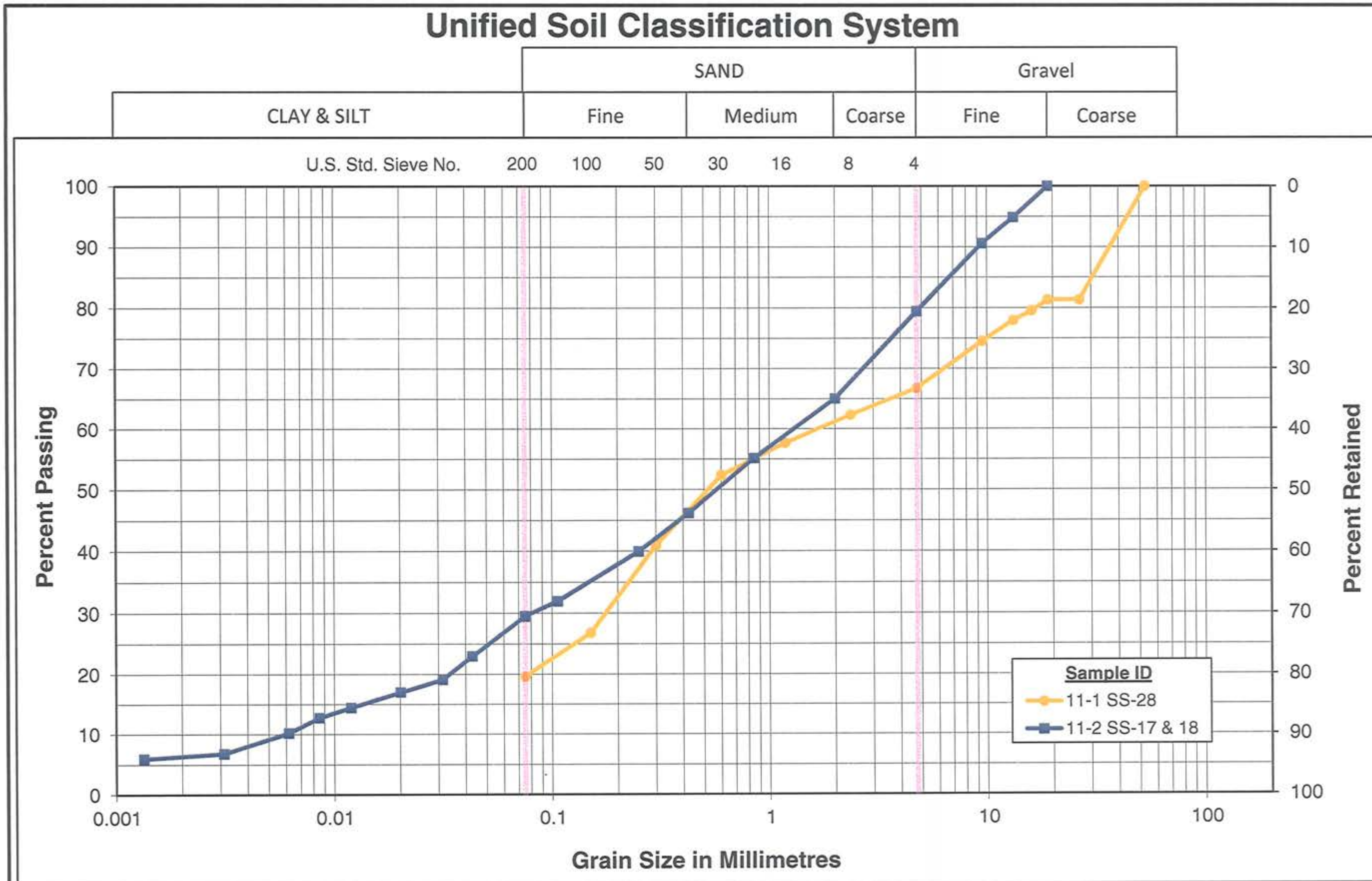


GRAIN SIZE DISTRIBUTION

Sand (SW-SM, SP-SM)

Figure No. 4

Project No. 165000776
GWP No. 3070-09-00



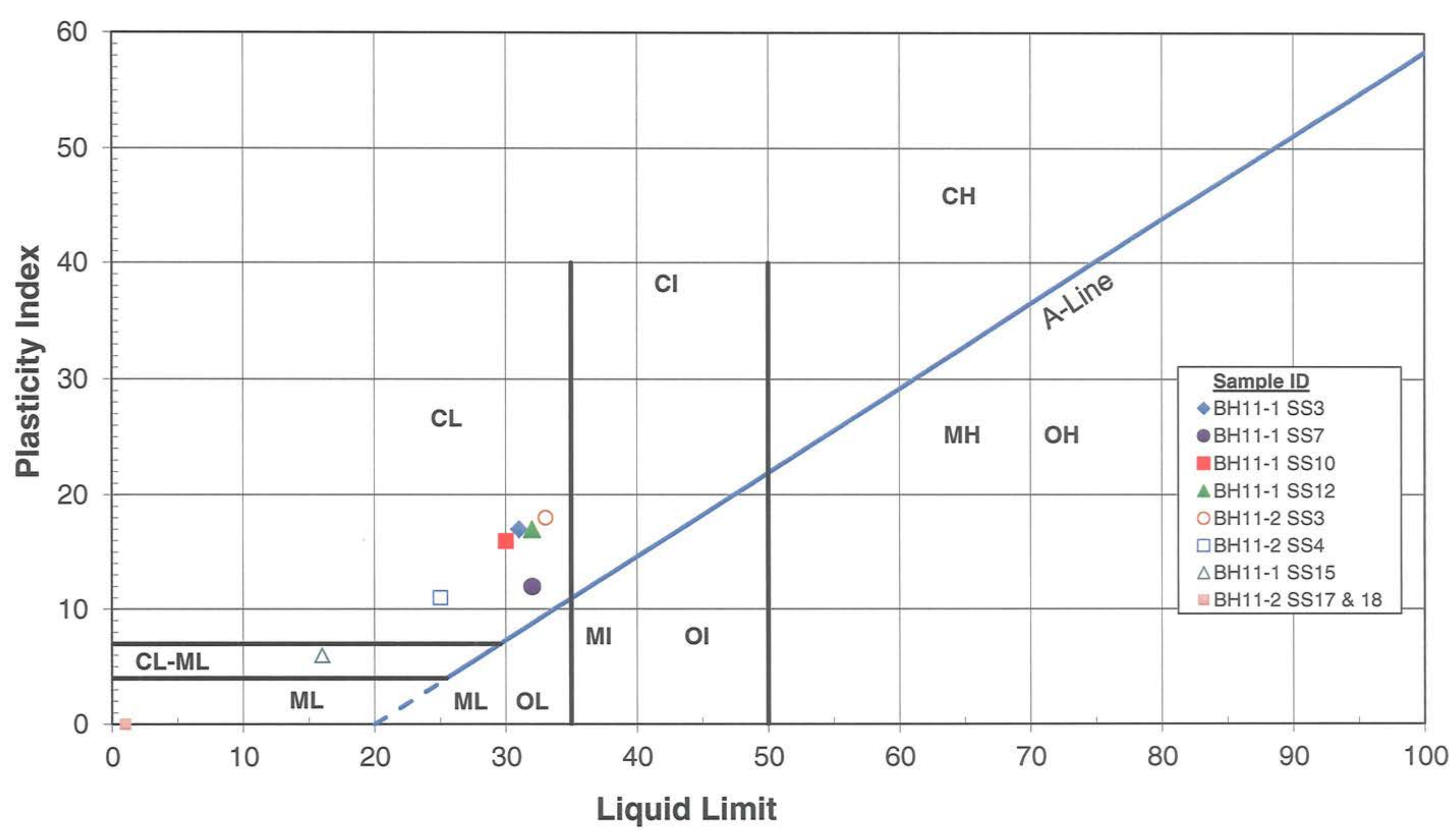
GRAIN SIZE DISTRIBUTION

Silty Sand with Gravel (SM) (Till)

Figure No. 5

Project No. 165000776
GWP No. 3070-09-00

Stantec



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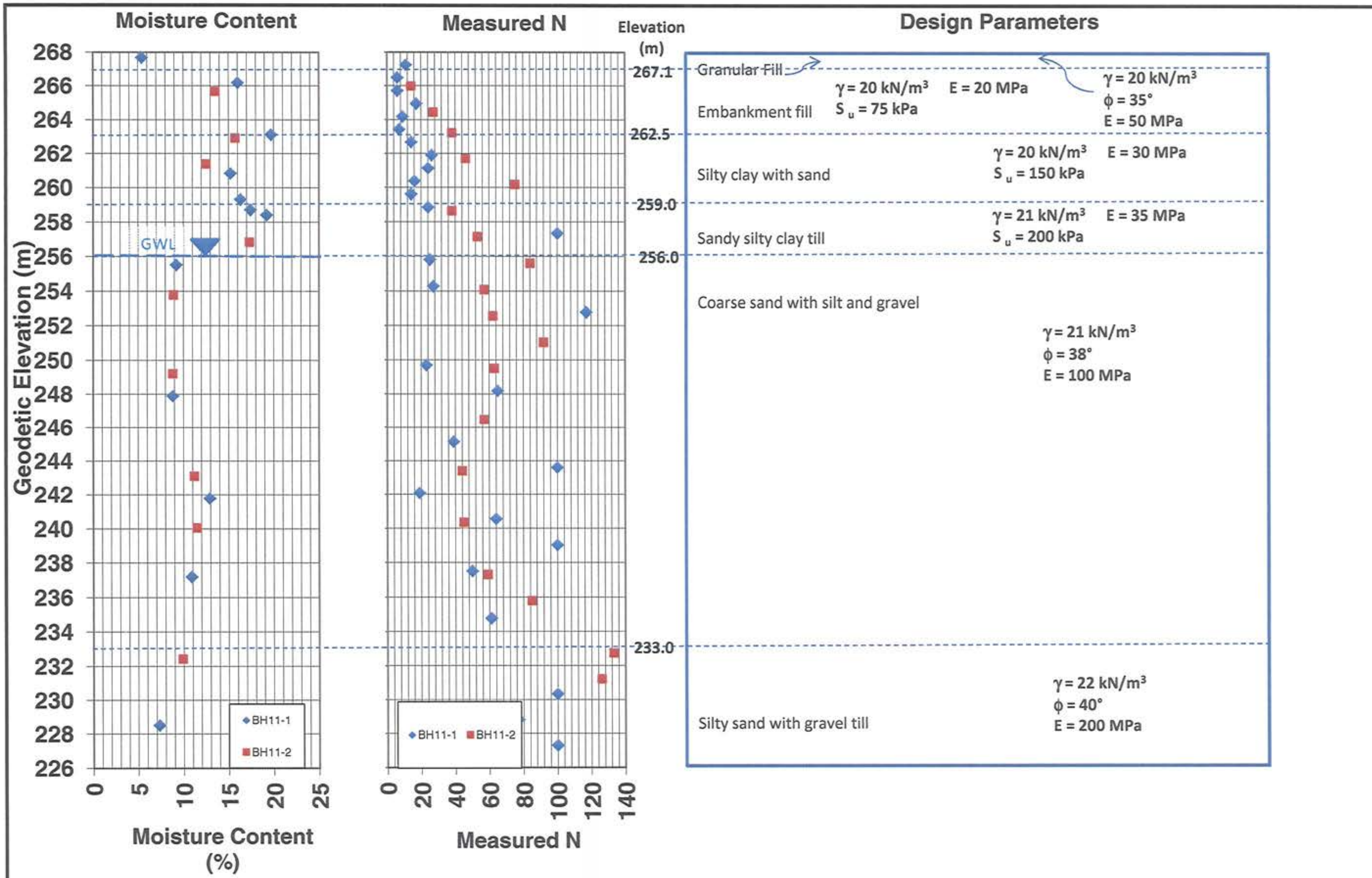
PLASTICITY CHART

Figure No. 6

Project No. 165000776
GWP No. 3070-09-00

APPENDIX D



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Stantec Consulting Ltd.

Project No. 165007776 Westminster Drive Underpass
GWP No. 3070-09-00

Figure 7

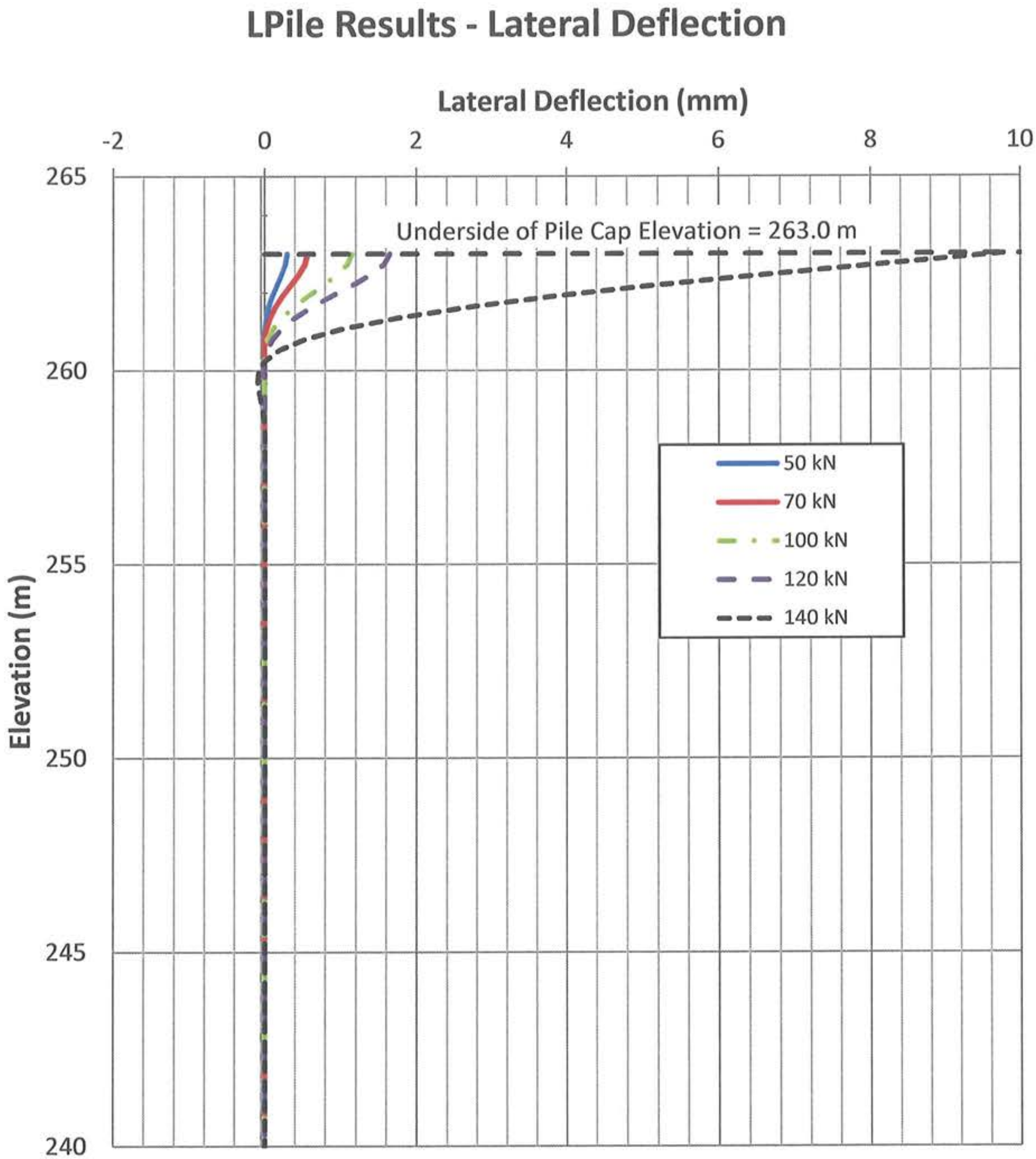


Figure 8
Lateral Deflection of HP 310x110 Piles

Stantec

Project No. 165000776
Highway 401 Improvements - Westminster Drive Underpass (London)
GWP No. 3070-09-00

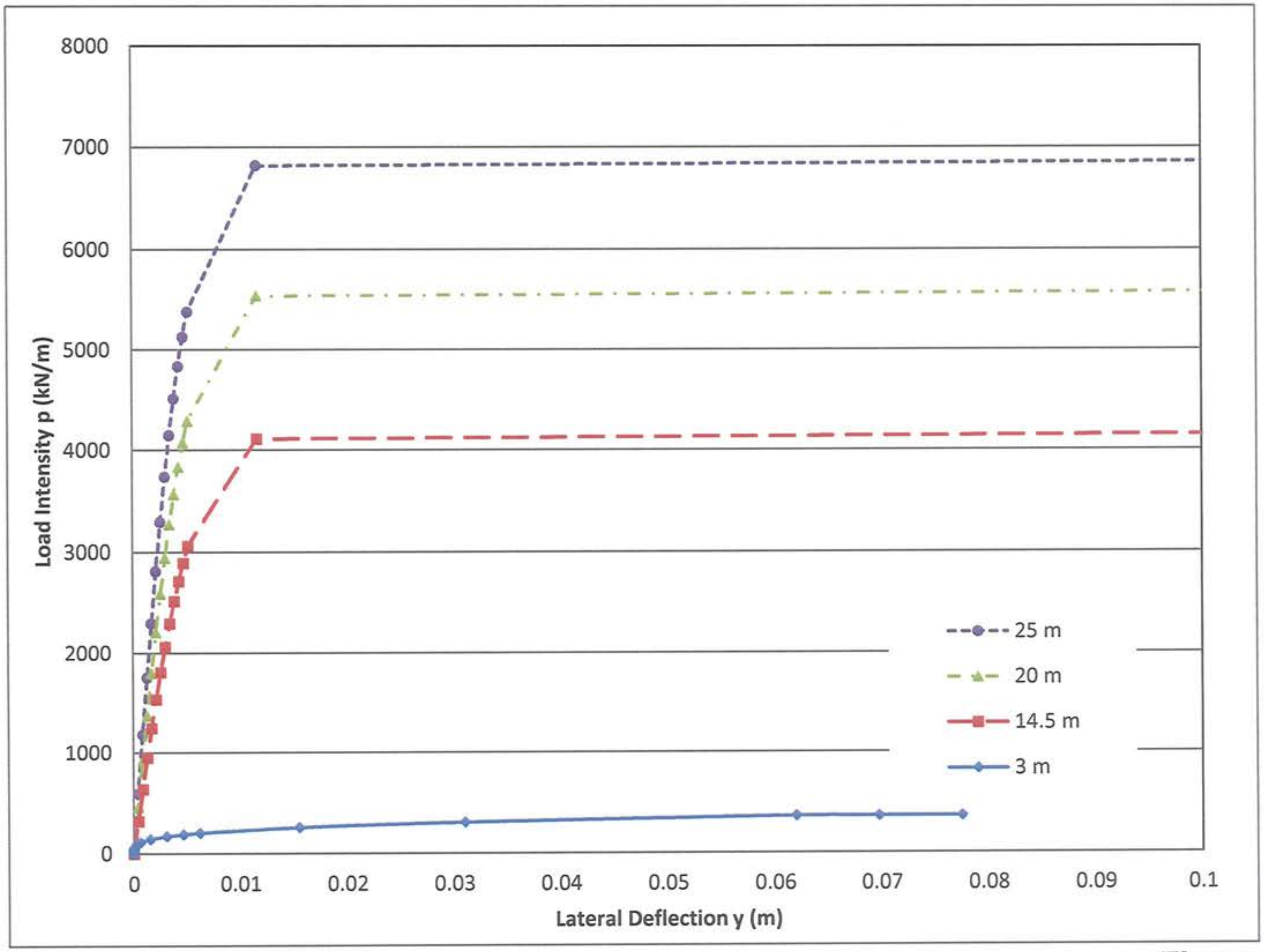


Figure 9
p-y Curves for Proposed HP 310x110 Piles

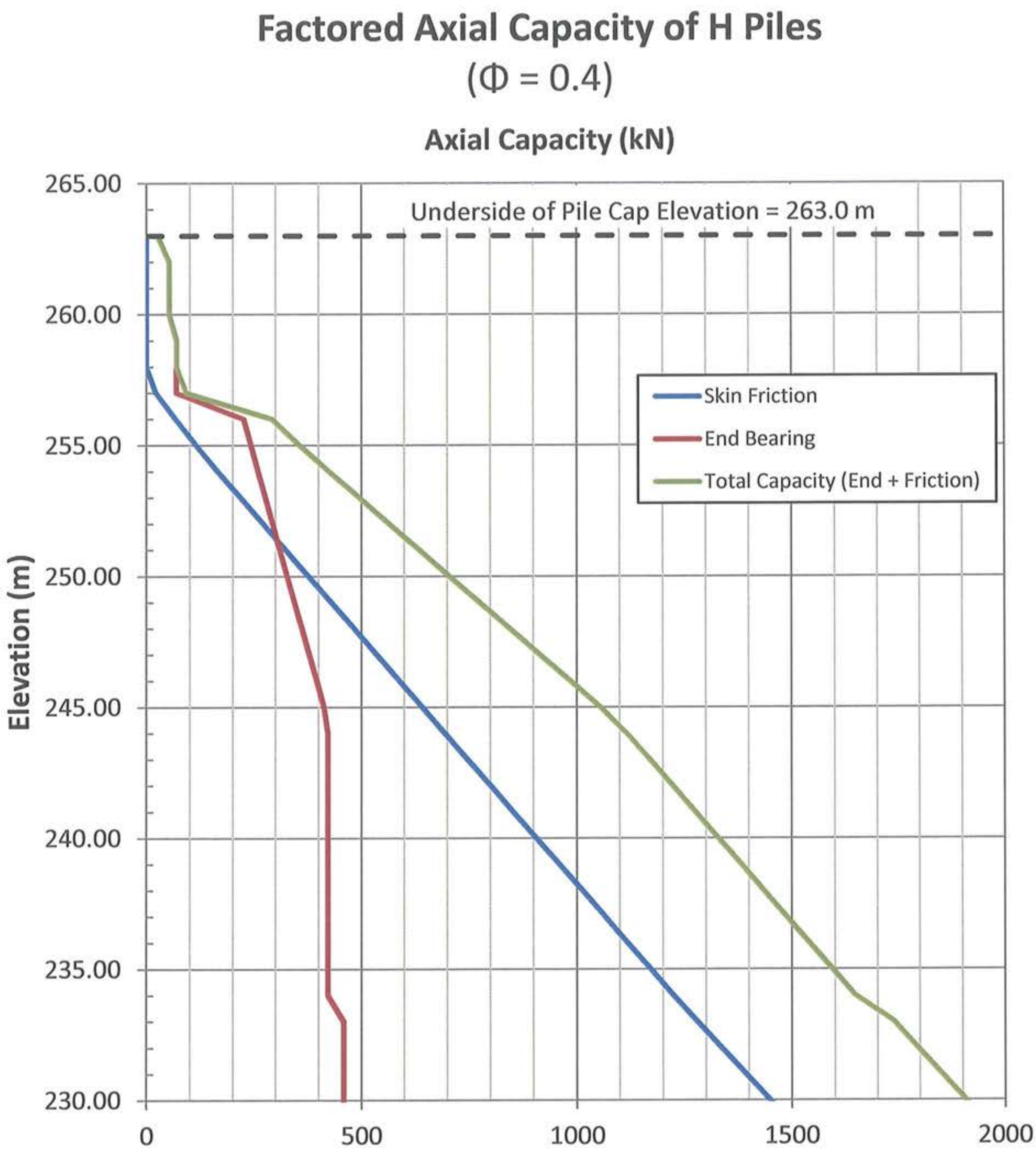
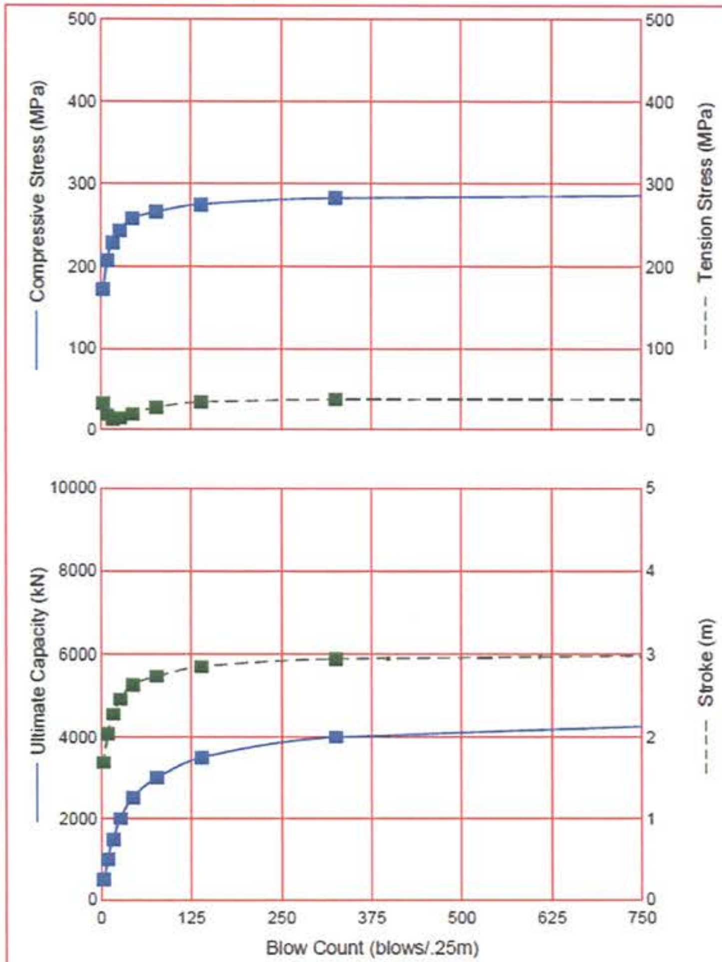


Figure 10
Factored Axial Capacity of HP 310x110 Piles

Project No. 165000776
Westminster Drive Underpass
GWP No. 3070-09-00

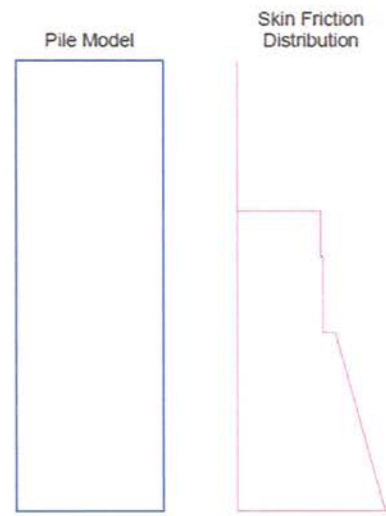
Stantec Inc
Westminster Drive Underpass

08-Jun-2012
GRLWEAP Version 2010



DELMAG D 30-32

Ram Weight	29.37 kN
Efficiency	0.800
Pressure	9652 (100%) kPa
Helmet Weight	8.45 kN
Hammer Cushion	10535 kN/mm
COR of H.C.	0.800
Skin Quake	2.540 mm
Toe Quake	5.178 mm
Skin Damping	0.164 sec/m
Toe Damping	0.490 sec/m
Pile Length	30.00 m
Pile Penetration	20.00 m
Pile Top Area	140.00 cm2



Res. Shaft = 84 %
(Proportional)

Figure 11
Driveability Analysis

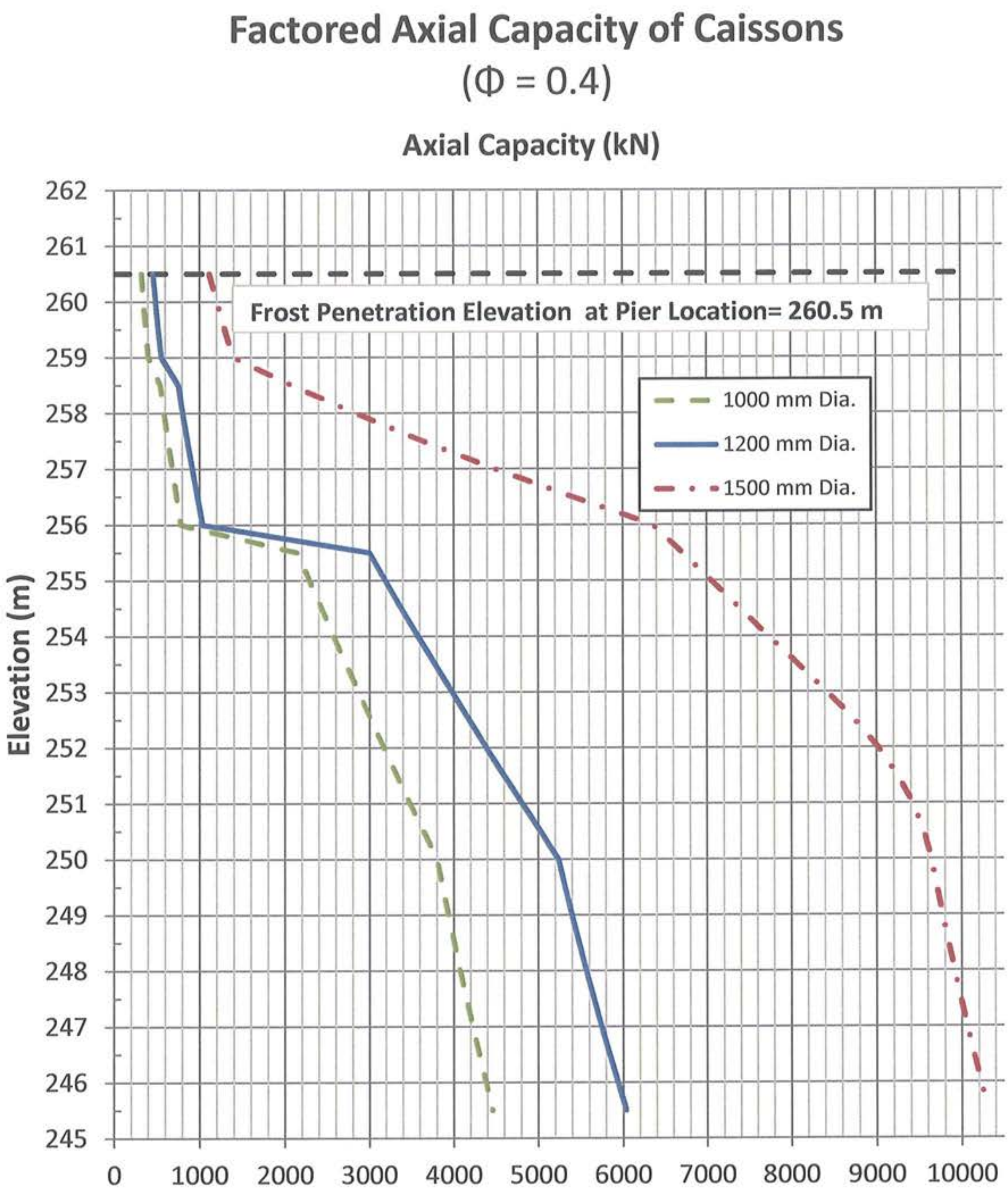
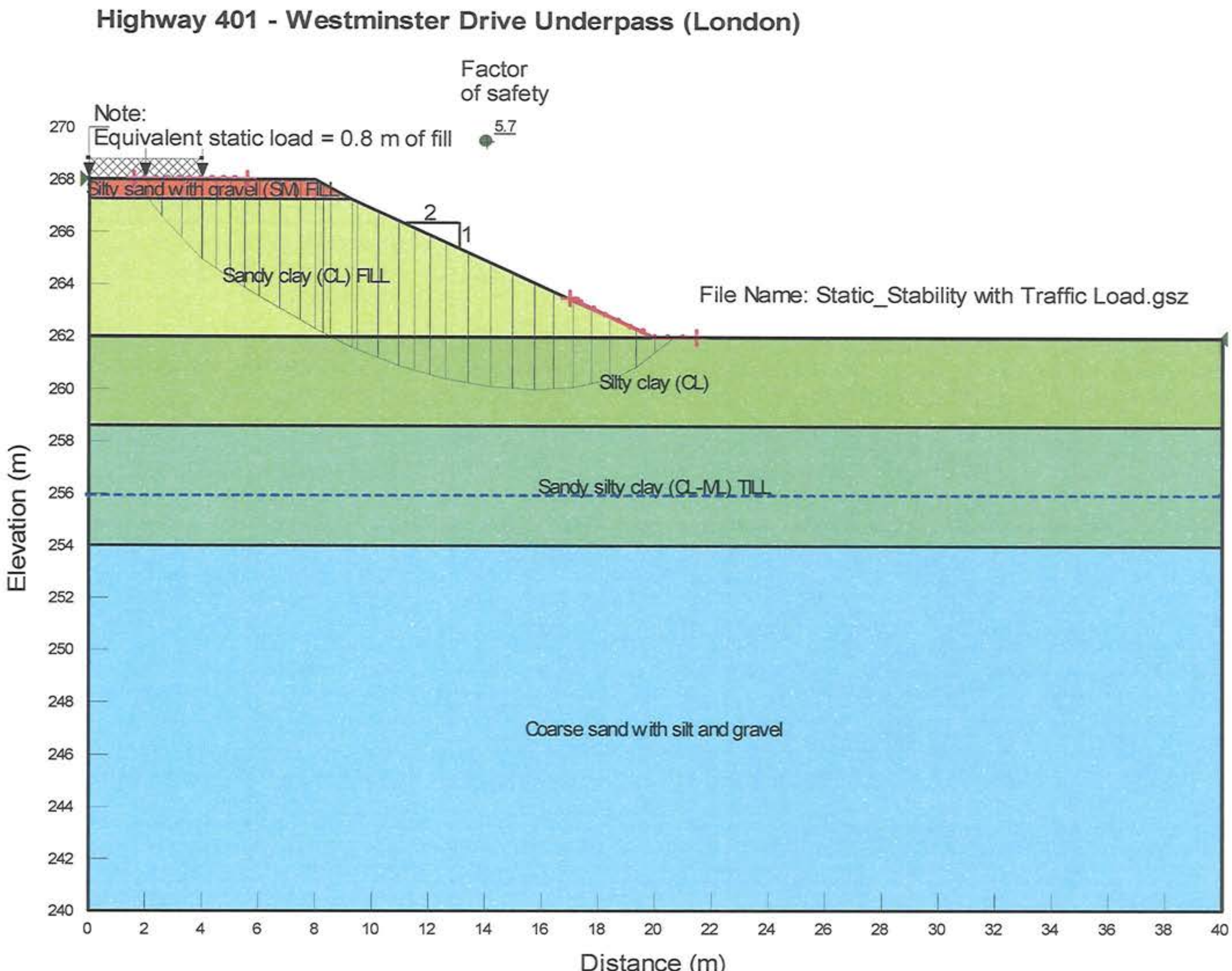


Figure 12
Factored Axial Capacity of Concrete Caissons



Stantec

Static Slope Stability Analysis

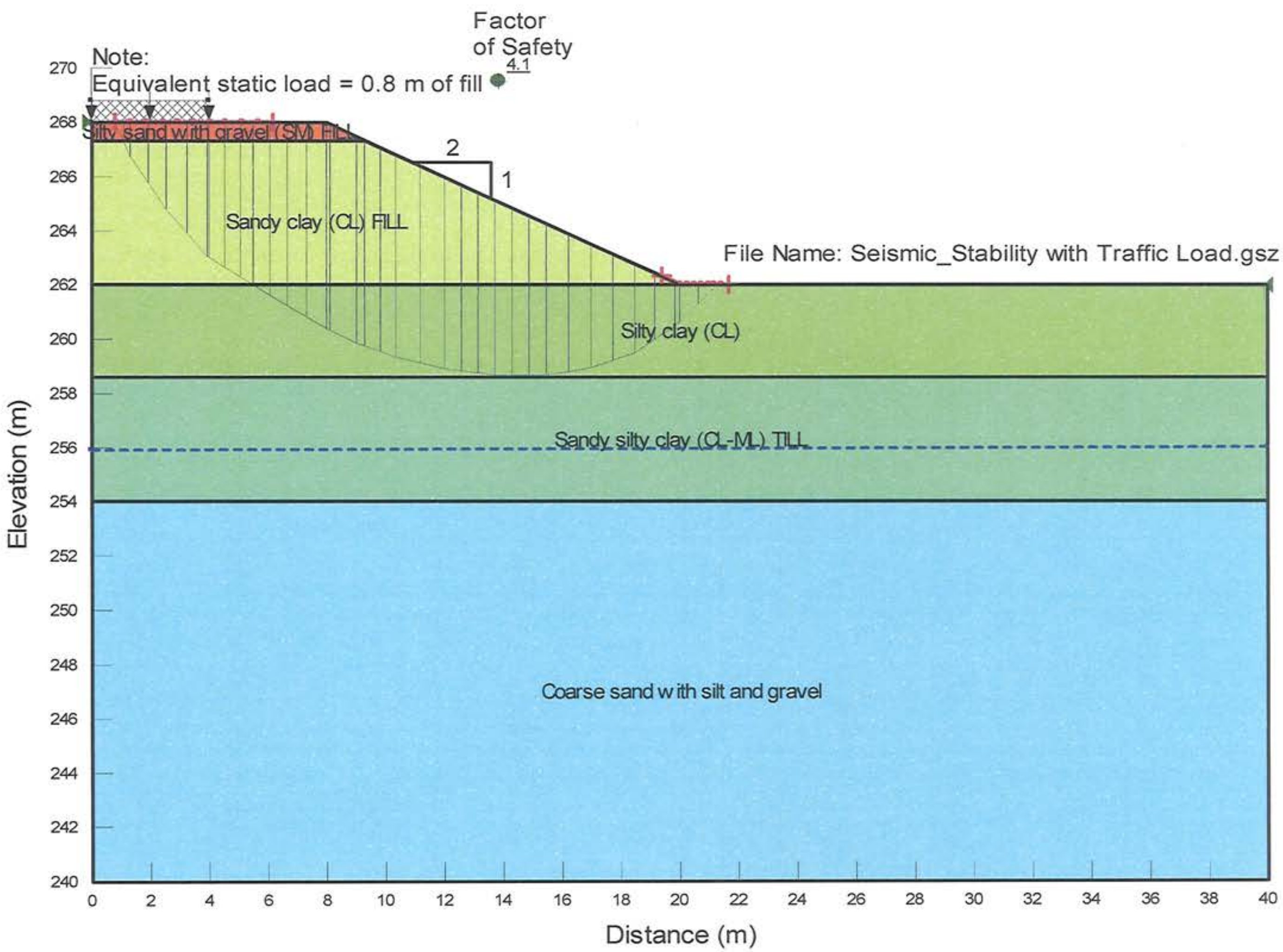
Highway 401 - Westminster Drive Underpass (London)
Side slope at 2H:1V

Figure 13a

Project No. 165000776

GWP No. 3070-09-00

Highway 401 - Westminster Drive Underpass (London)



Stantec	**Seismic Slope Stability Analysis** Highway 401 - Westminster Drive Underpass (London) Side slope at 2H:1V	Figure 13b Project No. 165000776 GWP No. 3070-09-00

APPENDIX E

Geological Survey of Canada Seismic Hazard Calculation

2005 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

Requested by: , Stantec Consulting Ltd.

March 22, 2011

Site Coordinates: 42.8983 North 81.2389 West

User File Reference: Westminster Rd & Hwy 401, London, ON

National Building Code ground motions:
2% probability of exceedance in 50 years (0.000404 per annum)

Sa(0.2)	Sa(0.5)	Sa(1.0)	Sa(2.0)	PGA (g)
0.189	0.095	0.047	0.013	0.123

Notes. Spectral and peak hazard values are determined for firm ground (NBCC 2005 soil class C - average shear wave velocity 360-750 m/s). Median (50th percentile) values are given in units of g. 5% damped spectral acceleration (Sa(T), where T is the period in seconds) and peak ground acceleration (PGA) values are tabulated. Only 2 significant figures are to be used. ***These values have been interpolated from a 10 km spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the calculated values.***

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.2)	0.027	0.075	0.121
Sa(0.5)	0.014	0.040	0.060
Sa(1.0)	0.006	0.018	0.029
Sa(2.0)	0.002	0.005	0.008
PGA	0.015	0.043	0.073

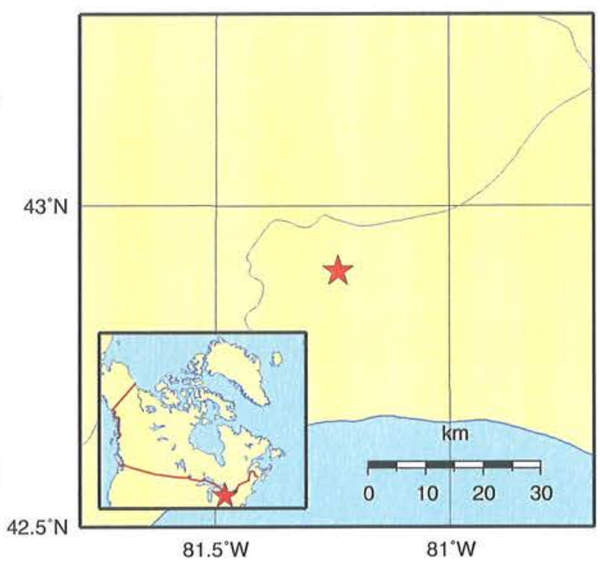
References

National Building Code of Canada 2005 NRCC no. 47666; sections 4.1.8, 9.20.1.2, 9.23.10.2, 9.31.6.2, and 6.2.1.3
Appendix C: Climatic Information for Building Design in Canada - table in Appendix C starting on page C-11 of Division B, volume 2

User's Guide - NBC 2005, Structural Commentaries NRCC no. 48192
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File xxxx
Fourth generation seismic hazard maps of Canada: Grid values to be used with the 2005 National Building Code of Canada (in preparation)

See the websites *www.EarthquakesCanada.ca* and *www.nationalcodes.ca* for more information



Aussi disponible en français

APPENDIX F

Preliminary Alternatives for Proposed Bridge Replacement

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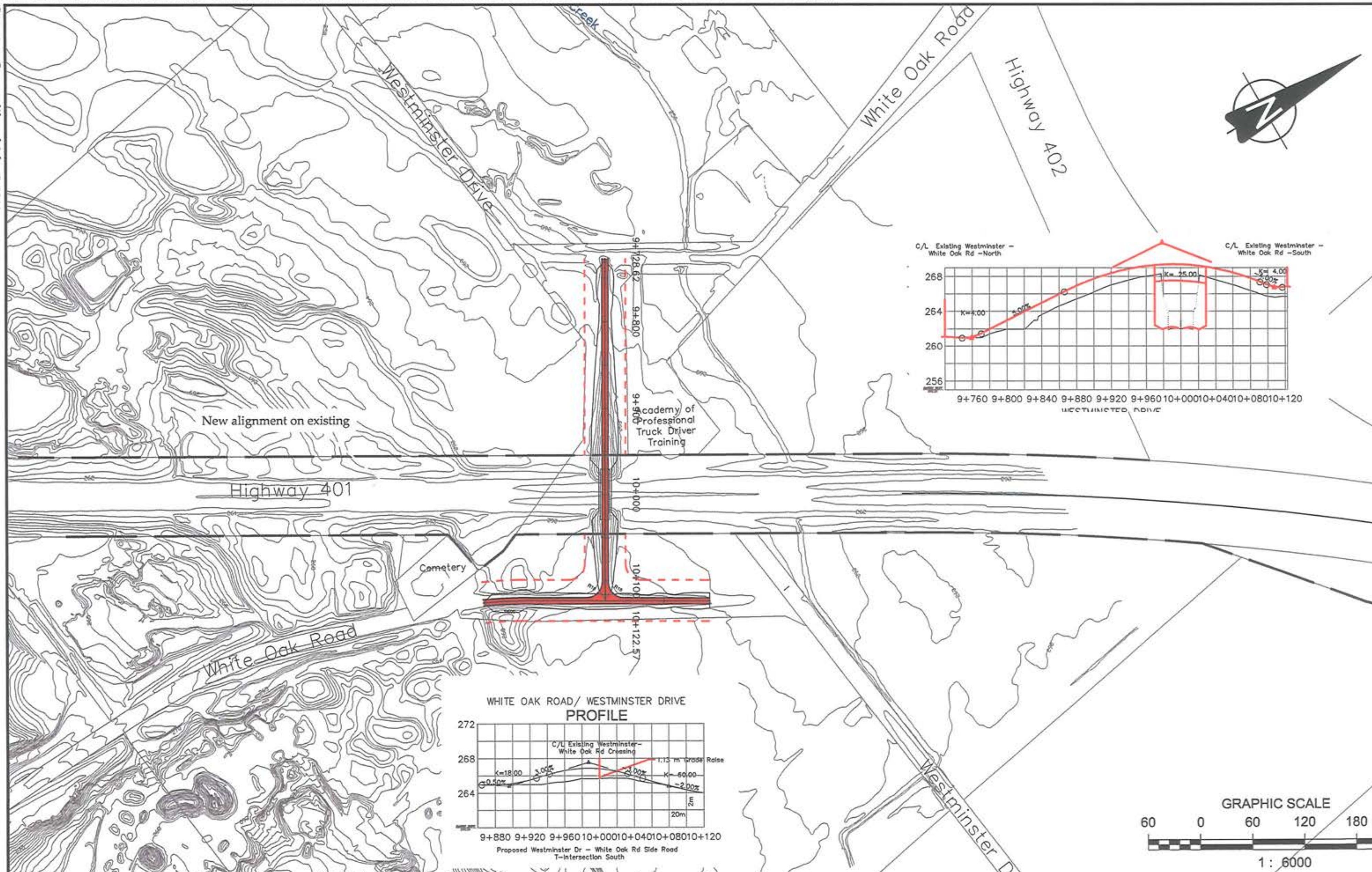

NOTE: THIS DRAWING ILLUSTRATES SUPPORTING INFORMATION SPECIFIC TO A STANTEC CONSULTING LTD. REPORT AND MUST NOT BE USED FOR OTHER PURPOSES.

ALTERNATIVE 1
UNCHANGED BRIDGE ALIGNMENT

GWP 3070-09-00, HIGHWAY 401, WESTMINSTER DRIVE, LONDON, ONTARIO

Job No.:	165000776
Scale:	1:6000 H, 1:600 V
Date:	11/06/24
Dwn. By:	GBB
App'd By:	

Dwg. No.: 1

Stantec

Client: MTO

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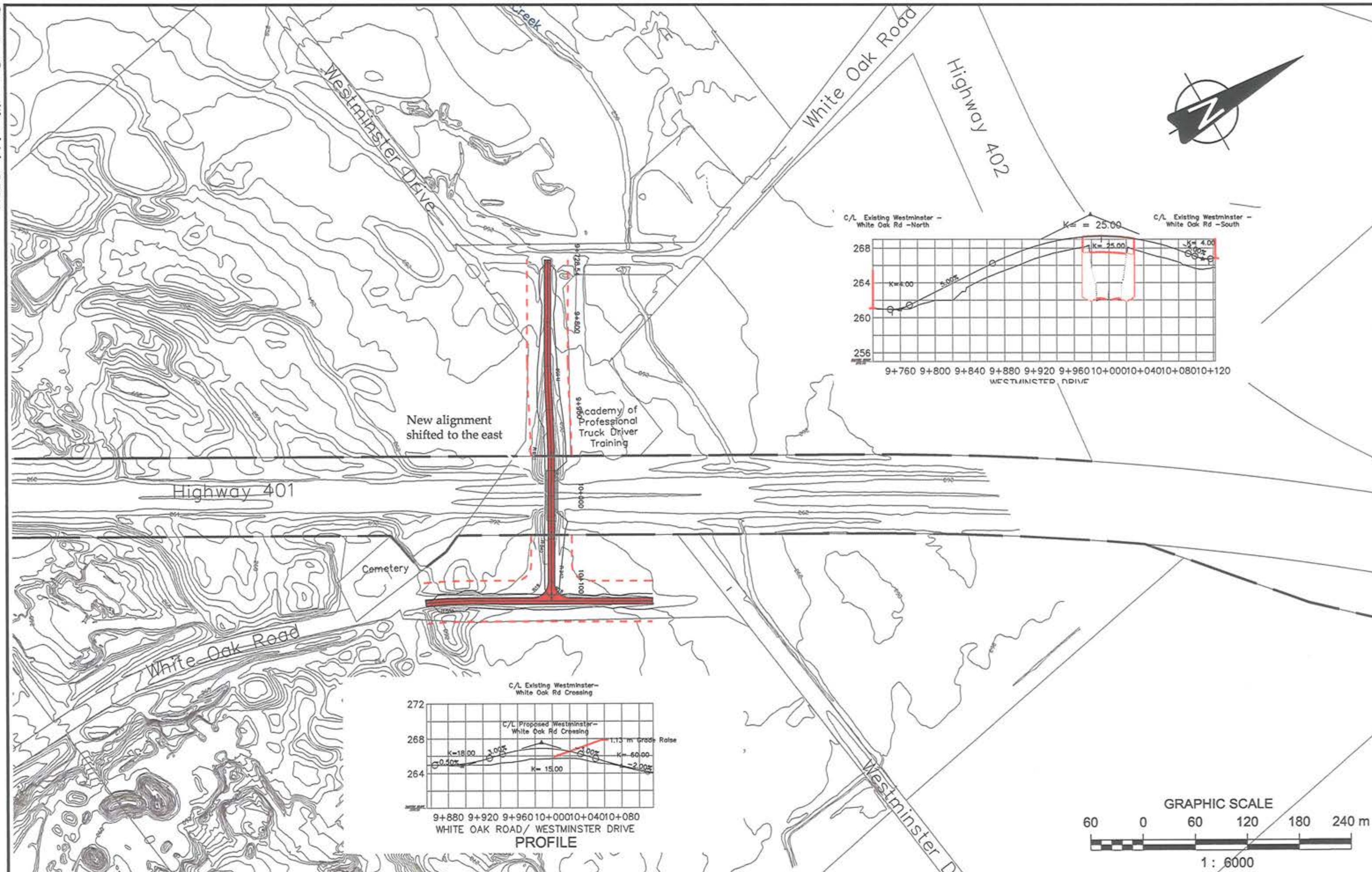

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ALTERNATIVE 2
BRIDGE SLIGHTLY SHIFTED TO THE EAST
GWP 3070-09-00, HIGHWAY 401, WESTMINSTER DRIVE, LONDON, ONTARIO

Job No.:	165000776	Dwg. No.: 2
Scale:	1:6000 H, 1:600 V	
Date:	11/06/24	
Dwn. By:	GBB	
App'd By:		
Client:	MTO	

Stantec

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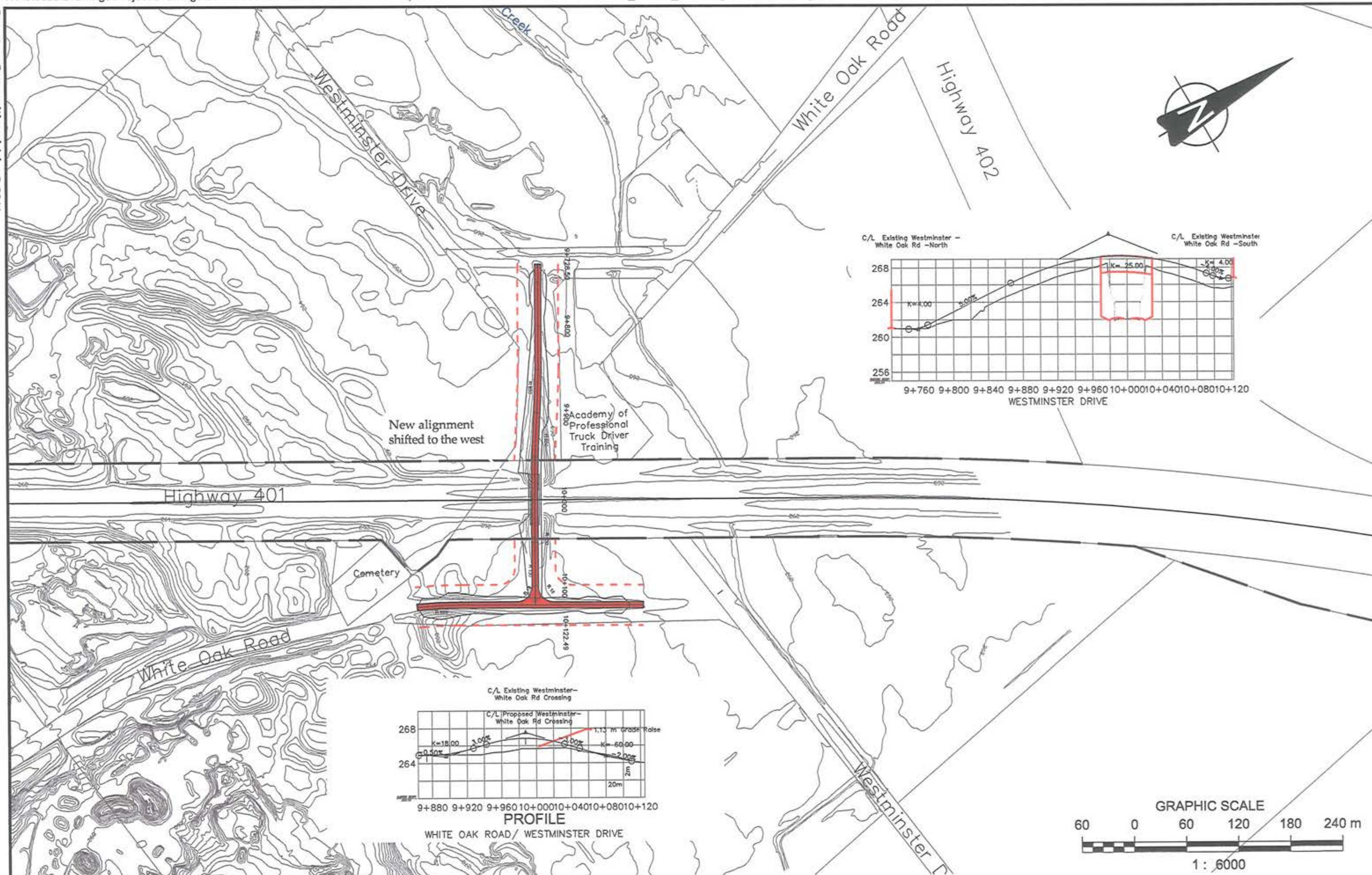

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ALTERNATIVE 3
BRIDGE SLIGHTLY SHIFTED TO THE WEST

GWP 3070-09-00, HIGHWAY 401, WESTMINSTER DRIVE, LONDON, ONTARIO

Job No.:	165000776
Scale:	1:6000 H, 1:600 V
Date:	11/06/24
Dwn. By:	GBB
App'd By:	

Dwg. No.: 3

Stantec

Client: MTO

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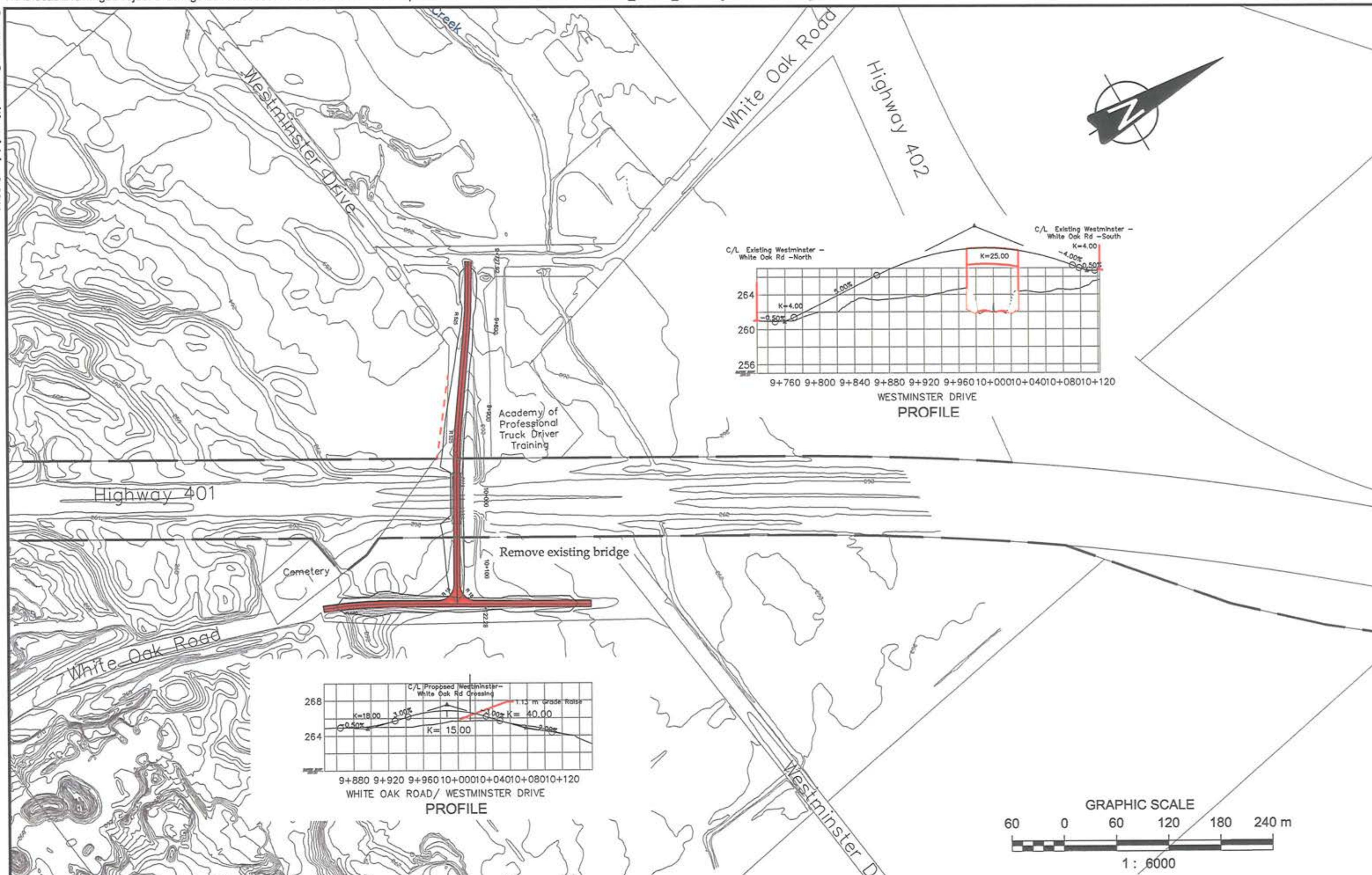

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ALTERNATIVE 4
BRIDGE SHIFTED APPROXIMATELY 15 m WEST
GWP 3070-09-00, HIGHWAY 401, WESTMINSTER DRIVE, LONDON, ONTARIO

Job No.:	165000776
Scale:	1:6000 H, 1:600 V
Date:	11/06/24
Dwn. By:	GBB
App'd By:	
Dwg. No.:	4

Client: MTO

Stantec

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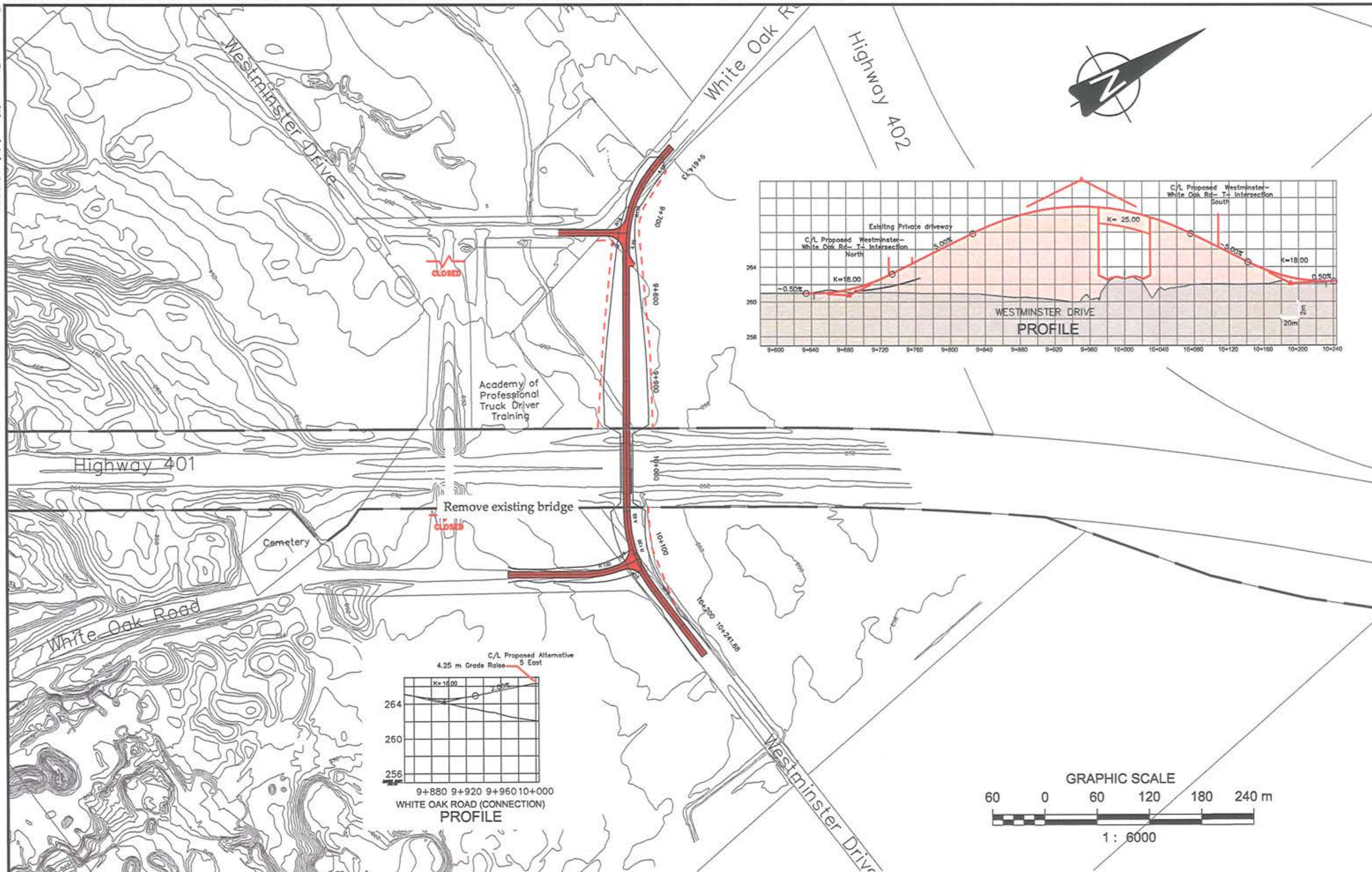

NOTE: THIS DRAWING ILLUSTRATES SUPPORTING INFORMATION SPECIFIC TO A STANTEC CONSULTING LTD. REPORT AND MUST NOT BE USED FOR OTHER PURPOSES.

ALTERNATIVE 5
BRIDGE SHIFTED APPROXIMATELY 200 m EAST
GWP 3070-09-00, HIGHWAY 401, WESTMINSTER DRIVE, LONDON, ONTARIO

Job No.:	165000776
Scale:	1:6000 H, 1:600 V
Date:	11/06/24
Dwn. By:	GBB
App'd By:	
Dwg. No.:	5
Client:	MTO

Stantec

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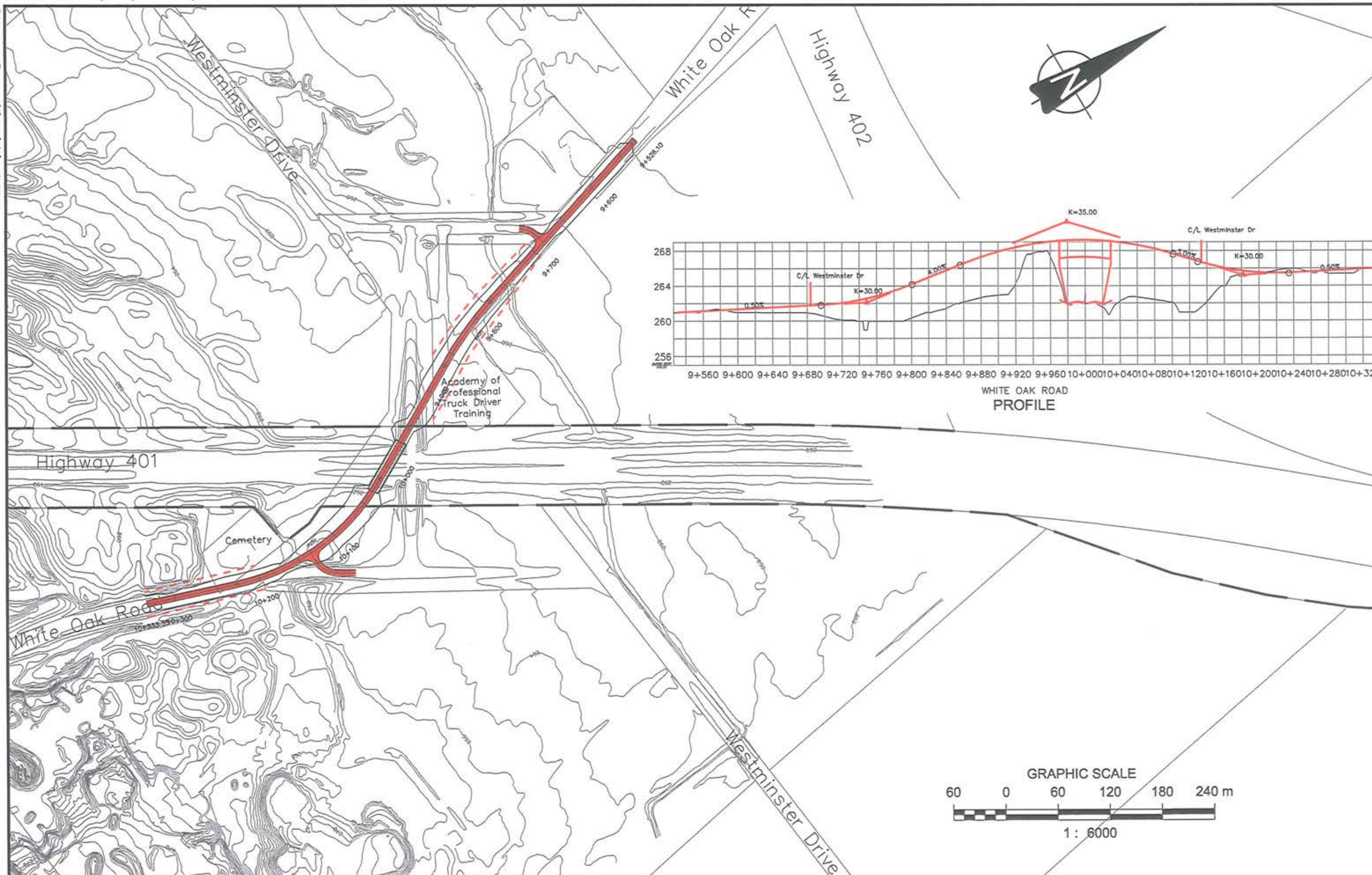

NOTE: THIS DRAWING ILLUSTRATES SUPPORTING INFORMATION SPECIFIC TO A STANTEC CONSULTING LTD. REPORT AND MUST NOT BE USED FOR OTHER PURPOSES.

ALTERNATIVE 5B
BRIDGE SHIFTED APPROXIMATELY 130 m EAST
GWP 3070-09-00, HIGHWAY 401, WESTMINSTER DRIVE, LONDON, ONTARIO

Job No.:	165000776
Scale:	1:6000 H, 1:600 V
Date:	11/06/24
Dwn. By:	GBB
App'd By:	
Dwg. No.:	6

Client: MTO

Stantec