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REPORT ON

**FOUNDATION INVESTIGATION AND DESIGN
OCR OVERPASS REPLACEMENT
STRUCTURE SITE 3-257 (EBL)
HIGHWAY 417
G.W.P. 4254-05-01**

Submitted to:

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November 2006

021-1155-4



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PART A

**FOUNDATION INVESTIGATION REPORT
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Marshall Macklin Monaghan (MMM) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations associated with the twinning of Highway 7 from two to four lanes in West Carleton and Goulbourn Townships in the City of Ottawa, and in Beckwith Township in Lanark County. Foundation investigation services are also required as part of this assignment for selected structures of Highway 417 from the Highway 417-7 interchange easterly (W.P. 458-98-00) to Moodie Drive. This report addresses the proposed replacement of the Ottawa Central Railway (OCR) overpass, eastbound lane (EBL) structure of Highway 417.

The terms of reference for the original scope of work and Addenda 1 through 7 issued during the proposal period are outlined in the MTO's Request for Proposal (RFP) and in Golder Associates' Proposal No. P21-1301, dated July 2002. Scope changes related to additional borehole investigation work at the abutments of several structures and the high fill embankment on the Highway 417E-7W ramp are outlined in Golder Associates' letters dated November 12, 2002 and November 18, 2002, respectively. Scope changes related to the additional geotechnical investigation for the bridge replacement and grade raise are outlined in the Golder Associates' letter dated April 17, 2006.

2.0 SITE DESCRIPTION

The existing OCR overpass structures are located on Highway 417 between Moodie Drive and March Road in Ottawa, Ontario. Through this section, Highway 417 consists of two eastbound lanes (EBL) and two westbound lanes (WBL) plus a bus lane each way and divided by a 30 m to 40 m wide median. The eastbound and westbound lanes are carried over the OCR line on separate structures. These two existing structures are designated as MTO's Structure Site 3-257. The existing approaches to the EBL OCR overpass bridge have net embankment heights of about 7.5 m each with side slope profiles of approximately 2H:1V to the south and moderate swales into the median to the north. Erosion of the south slope of the west embankment is presently taking place from two culverts at about Station 12+600 and from a seepage source at about Station 12+620. In addition, under the existing fill at the west and east abutments there is evidence of rotation of the approach slabs and settlements beyond the slabs.

The existing bridges for both the eastbound and westbound lanes consist of a three-span concrete deck on precast concrete girders, supported on concrete abutments and piers. The bridges span a distance of approximately 45 m between abutments. The foundation investigations for the design of the existing bridges were carried out in 1966 and the results of those investigations are summarised in MTO's GEOCREs No. 31G5-109, *Preliminary Foundation Investigation Report, W.P. 108-65, Proposed Overhead at the Ottawa Queensway and C.N.R. Crossing, District #9 (Ottawa) Hwy: Queensway W.J. 66-F-17*. The Department of Highways' Ontario Bridge Division Drawing No. D6138-P-2, "*C.N.R. Overhead*," dated May 1967, indicates the EBL structure to be supported on steel H-piles and the elevation of the bridge deck to vary from about Elev. 93.5 m to 94.6 m.

It is currently proposed to increase the capacity of Highway 417 in this area to four lanes, plus a bus lane in both the eastbound and westbound directions. It is our understanding that the existing WBL structure is wide enough to accommodate the increased number of lanes without requiring any structural modification. However, the existing EBL structure is too narrow and it is proposed to be replaced with a new structure that is approximately 11 m wider than the existing to accommodate the additional traffic lanes. Widening of the existing approach embankments by about 10 m (EBL) and 4 m (WBL) primarily into the median swale is required along with a grade raise for the approach embankments of up to about 2.5 m (in the median area) and up to about 1.5 m (on the existing roadway).

The results of the 1966 foundation investigations indicate that the overburden at the OCR overpass structures consists of surficial sands, overlying a thick deposit of sensitive clayey silt to silty clay, overlying a thin layer of sandy till underlain by bedrock at a depth of about 18 m.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out for the initially proposed widening of the eastbound OCR overpass structure between November 25 and December 6, 2002. During this time, a total of three (3) sampled boreholes (02-402 to 02-404) and two cone penetration tests (CPT 02-401 and CPT 02-405) were advanced within the area of the proposed structure and approach embankment widening (i.e. on the north side of the existing EBL structure and approaches). Subsequent to this an additional field investigation was carried out between August 15 and 21, 2006 for the currently proposed grade raise and bridge replacement option. During this time, two (2) additional sampled boreholes (06-1 and 06-2) were put down within the area of the south side of the proposed replacement structure. Three (3) additional boreholes were put down beyond the approach embankments and are reported in a separate report.

The boreholes were advanced by hollow stem augers using truck-mounted and track mounted drill rigs, supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario. The five sampled boreholes were advanced (including rock coring) to depths ranging from 19.5 m to 26.1 m below the existing ground surface. Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth increasing to 3 m intervals below a depth of about 12 m, using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. In situ vane shear strength testing was carried out using an MTO 'N'-sized vane at regular intervals of depth, where appropriate, in clayey strata. Samples of bedrock were obtained using an 'NQ' size rock core barrel.

The water level in the open boreholes was observed throughout the drilling operations and two standpipe piezometers were installed in selected boreholes to monitor the groundwater level(s) at the site. The screened portion of each standpipe was installed within the overburden soils at a depth of about 20 m. Both of the standpipes consist of 1 inch diameter rigid PVC pipe with a 1.5 m long slotted screen section, installed within a silica sand filter pack, and sealed below minimum 1 m long sections of bentonite pellet backfill.

Two additional boreholes were advanced through the embankment fill in the median swale in order to facilitate the start of the CPTs. The CPT is a in situ testing technique for site characterisation studies. The CPT consists of a special cone tip equipped with electronic sensing elements to continuously measure tip resistance, local side friction on a sleeve and porewater pressure. It is pushed at a constant rate into the ground using a drill rig (ASTM D5778-95). A continuous stratigraphic profile together with engineering properties, such as strength, stress history and density, can be interpreted from the results of the CPT.

The CPT equipment was advanced using the hydraulic ram system on the track-mounted drill rig. The two CPTs were advanced to refusal, which was encountered at depths ranging from about

18.6 m to 20.8 m below ground surface. Record of Cone Penetration Test sheets are included with the Record of Borehole sheets following the text of this report. Profiles of tip resistance, porewater pressure during pushing and sleeve-friction are presented together with an interpreted profile of undrained shear strength (s_u) and classification index (I_c) that is used to infer soil type (stratigraphy).

The field work was supervised on a full-time basis by members of Golder's staff who located the boreholes and CPTs, directed the drilling, sampling, and in situ testing operations, and logged the boreholes. The soil and bedrock samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Ottawa for further examination, and subsequently to Golder's laboratory in Mississauga for testing. Laboratory testing, including water content determinations, Atterberg limits testing and grain size distribution analyses, was carried out on selected soil samples. For scheduling reasons, laboratory consolidation tests were not carried out on samples obtained from the new boreholes advanced within the OCR structure limits. Instead, consolidation tests were performed on Shelby tube samples of the silty clay obtained from the investigation for the Tall Rock Fill embankment widening (Golder Report 021-1155-12, titled, "Tall Rock Fill Embankment Widening, Highway 417") located immediately adjacent to the OCR structure area. All boreholes were abandoned in accordance with O. Reg. 128 (amendment to O. Reg. 903).

The borehole/CPT locations and ground surface elevations were determined by Golder relative to points staked in the field by MMM. The borehole/CPT locations from the current investigation, including MTM NAD83 northing and easting coordinates and ground surface elevations referenced to geodetic datum, are summarised in the following table and shown on Drawing 1.

Borehole / CPT Number	Borehole/CPT Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
CPT02-401	STA 12+636, 3.2 m north of centreline stake	5021560.1	355216.3	93.0
02-402	STA 12+650, 5 m north of centerline stake	5021574.1	355243.2	94.0
02-403	STA 12+694 at centreline stake	5021592.4	355264.0	87.0
02-404	STA 12+721, 5.3 m north of centerline stake	5021601.8	355290.1	92.5
CPT02-405	STA 12+738, 1.9 m north of centerline stake	5021613.4	355303.0	90.7
06-1	STA 12+731, 1.7 m north of edge of barrier wall	5021585.6	355310.7	92.7
06-2	STA 12+708, 2.4 m east of existing pier column centreline	5021571.9	355292.4	86.5

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geological Conditions

The study area for this assignment lies within two minor physiographic regions that lie within the major physiographic region of the Ottawa-St. Lawrence Lowland (Chapman and Putnam, 1984). The Highway 7 area between the Highway 417-7 interchange and Carleton Place is part of the Smiths Falls Limestone Plain, while the area along Highway 417 east of the Highway 417-7 interchange is part of the Ottawa Valley Clay Plain. Most of both physiographic regions are underlain by a series of sedimentary rocks, consisting of sandstones, dolostones, limestones and shales that are, in turn, underlain by igneous and metamorphic bedrock of the Precambrian Shield. The Shield rock generally outcrops to the north of the Ottawa River, and it is also present immediately below the overburden in a localised area between the Hazeldean Fault (approximately the location of the Carp River) and the Ottawa River.

The Ottawa Valley Clay Plain region, present along Highway 417 from the Highway 417-7 interchange site eastward, is characterised by relatively thick deposits of sensitive marine clay, silt and silty clay that were deposited within the Champlain Sea basin. These deposits, known as the Champlain Sea clay or Leda clay, overlie relatively thin, commonly reworked glacial till and glaciofluvial deposits, that in turn overlie bedrock (Chapman and Putnam, 1984). West of the Carp River valley along Highway 417, the upper bedrock consists of limestone of the Ottawa Formation, as described above. Along the east flank of the Carp River valley, the upper bedrock consists of sandstones and dolostones that have been cut by igneous and metamorphic rocks, controlled by faulting in the vicinity of the Hazeldean Fault and associated higher ground of the Carp Ridge (Belanger, 1998). That fault is considered to be seismically inactive, however the bedrock in the vicinity of that fault is known to be rather fractured and to have a highly irregular surface topography.

4.2 Site Stratigraphy

As part of the current subsurface investigation at this site, five boreholes and two CPTs were advanced within or immediately adjacent to the limits of the foundation elements and immediate approach embankments for the proposed structure replacement and embankment widening. The borehole locations and ground surface elevations are shown on Drawing 1.

The detailed subsurface soil, bedrock and groundwater conditions encountered in the boreholes and inferred from the CPT's, together with the results of laboratory testing carried out on selected soil samples, are given on the attached Record of Borehole, Drillhole and Cone Penetration Test Sheets following the text of this report. The results of the laboratory testing are provided in Appendix A. An additional Record of Borehole (No. 1) from the 1966 investigation at this site

(GEOCRETS No. 31G5-109) is provided in Appendix C. The information from this borehole (denoted as borehole 66-1 on Drawings 1 and 2) has been utilized in conjunction with the information from the current investigation to develop the subsurface stratigraphy at the site.

A summary of the in situ field test results along with the laboratory test results is presented on Figure 1. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests (SPTs). These boreholes therefore, represent transition between soil types rather than exact planes of geological change. Furthermore, subsurface conditions will vary between and beyond the borehole locations.

In summary, the subsurface conditions in the area of the OCR structure consist of up to about 0.3 m of topsoil (where present) underlain by about 1.5 m to 12.4 m of fill, varying in composition from earth fill to rock fill. Beneath the fill material is a deposit of native silty clay to clay with a total thickness ranging from about 10.0 m to 17.2 m. The upper approximately 2 m to 5 m of the silty clay stratum has a weathered, stiff to very stiff crust, while the underlying portion has a stiff to firm consistency. The silty clay is underlain at most locations by a 0.2 m to 1.1 m layer of silty sand and gravel till overlying the bedrock surface. The surface of the bedrock appears to slope slightly across the site from west to east with elevations ranging from about 70.8 m to 70.0 m at the borehole locations.

A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections. Stratigraphic profiles and sections of this site are shown on Drawings 1 and 2.

4.2.1 Topsoil/Organics

Topsoil was encountered at the ground surface in the boreholes advanced near the north edge of the abutment areas of the proposed EBL structure replacement. About 0.2 m and 0.3 m of topsoil was encountered in Boreholes 02-402 and 02-404, respectively. A layer of organic silt was encountered in borehole 06-1 (near the south edge of the east abutment) below the existing embankment fill between depths of about 12.4 m and 12.7 m.

4.2.2 Fill

Fill, associated with the construction of the existing approach embankments and associated with the construction of the existing access road located adjacent to the OCR line, was encountered at all of the borehole and CPT locations advanced as part of the current subsurface investigation. Fill was not identified in borehole 66-1 which was drilled prior to the existing roadway

construction, however, a 0.8 m thick layer of sand was encountered at the ground surface in this borehole.

In borehole 02-403 drilled on the NCC access pathway located at the toe of the east approach embankments, approximately 1.5 m of sand and gravel fill was encountered immediately below the ground surface. A single Standard Penetration Test (SPT) 'N' value equal to 19 blows per 0.3 m of penetration was measured in this layer, indicating that the fill in this area has a compact relative density.

In boreholes 02-402 and 02-404 drilled near the north side of the west and east abutment areas of the proposed EBL structure replacement, a total of about 5.6 m and 9.3 m, respectively, of earth fill was encountered immediately below the topsoil. The upper approximately 0.6 m to 2.9 m of the fill in these areas is composed of grey-brown silty clay containing trace sand and gravel, to sandy silt containing trace clay. The lower 5.0 m to 6.4 m of the fill is composed of a brown sand and gravel to sand, trace to some silt and gravel, containing cobbles.

A single Standard Penetration Test (SPT) carried out in the upper portions of the sandy silt earth fill measured an 'N' value of 2 blows per 0.3 m of penetration, suggesting a very loose state of packing. In the lower portion of the earth fill, the measured SPT 'N' values range from 6 to 17 blows per 0.3 m of penetration, indicating a loose to compact relative density. Natural water contents measured on selected samples of the earth fill ranged between 4 and 22 percent. Grain size distribution curves for selected samples of the fill are shown on Figures A2 and A3.

In boreholes CPT02-401 and CPT02-405 drilled in the area of the new widened approach embankments for the EBL structure, about 4.1 m to 10.1 m, respectively, of fill was encountered immediately below the ground surface. The fill in these areas is generally composed of sand and gravel, containing cobbles and boulders.

In boreholes 06-1 drilled through the existing east approach embankment near the south wing wall, about 12.2 m of fill was encountered below a 0.2 m thick layer of asphaltic concrete. The upper 2.1 m of fill is composed of grey sand and gravel and the lower 10.1 m of fill is composed primarily of sand with trace to some gravel and silt with cobbles and boulders. A grain size distribution curve for a selected sample of the fill from this borehole is also shown on Figure A3. Measured SPT 'N' values range from 12 to 49 blows per 0.3 m of penetration, indicating a compact to dense relative density. Natural water contents measured on two selected samples of the fill are 4 and 6 percent.

In borehole 06-2 drilled on the NCC access pathway near the south side of the existing pier column on the east side of the railway track, 0.2 m of sand and gravel fill overlying 4.1 m of sand fill with trace of silt, gravel and silty clay was encountered. Measured SPT 'N' values for the

earth fill range from 3 to 9 blows per 0.3 m of penetration, indicating a very loose to loose relative density.

4.2.3 Clayey Silt and Sandy Silt

Beneath the fill material in borehole 02-403, a 2.9 m thick layer of clayey silt and sandy silt trace to some clay was encountered. Standard Penetration Test (SPT) measured 'N' values ranged between 4 and 19 blows per 0.3 m of penetration (decreasing with depth) suggesting a firm to very stiff consistency. Natural water contents of 21 and 25 percent were measured on selected samples from this deposit.

4.2.4 Silty Clay to Clay

A deep deposit of silty clay to clay was encountered below the fill and/or the clayey silt to sandy silt deposit in all of the boreholes and CPTs in this area. The thickness of the silty clay to clay ranges from about 10.0 m to 17.2 m at the investigated locations. In the median area and at the location of borehole 66-1, the upper approximately 4 m of the deposit is composed of a weathered brown to grey brown silty clay crust containing occasional silty sand and clayey silt seams. The thickness of the weathered crust reduces to about 2 m to 3 m in the vicinity of boreholes 06-1 and 06-2. The lower approximately 8 m to 14 m of the deposit is composed of a grey-brown to grey silty clay to clay. Standard Penetration Testing (SPT) carried out in the upper weathered silty clay measured 'N' values varying from 2 to 23 blows per 0.3 m of penetration indicating a soft to very stiff consistency while SPTs carried out in the lower silty clay measured 'N' values of 0 (weight of rod or static weight of hammer) to 2 blows per 0.3 m of penetration indicating a very soft to soft consistency.

The results of in situ shear vane tests carried out in boreholes 02-402 to 02-404, 06-1 and 06-2 are shown on the Record of Borehole sheets and summarised on Figure 1. The interpreted profiles of undrained shear strength from the CPTs carried out at CPT02-401 and CPT02-405 are shown on the Cone Penetration Test sheets and included in the summary on Figure 1. Based on the in situ vane tests from the current investigation, the undrained shear strength of the upper weathered silty clay varies from about 50 kPa to 85 kPa and the undrained shear strength of the lower silty clay varies from about 28 kPa to 60 kPa. These results are generally consistent with the results from the CPTs that estimate the undrained shear strength to vary from 50 kPa to greater than 100 kPa in the upper silty clay and vary from 45 kPa to 70 kPa in the lower silty clay. The results of the in situ vane tests carried out in borehole 66-1 from the 1966 investigation at the site measured undrained shear strength values within the silty clay to clay stratum ranging from about 45 kPa to 95 kPa, which are consistent with the results from the current investigation.

The sensitivity of the silty clay to clay stratum, as estimated from the in situ vane tests from the current investigation, generally ranges from about 3.5 to 15 (increasing with depth) with a few values of 25 or greater, as shown on Figure 1. This range implies that the silty clay to clay is typically of medium to extra sensitivity (CFEM, 1992).

Natural water contents measured on selected samples of the upper silty clay to clay from the current investigation ranged from about 22 to 51 percent with an average of about 35 percent. Atterberg limits testing conducted on selected samples of the upper silty clay to clay from the current investigation gave liquid limits ranging from about 38 to 71 percent and plasticity indices ranging from about 20 to 48 percent, indicating clay of intermediate to high plasticity. Natural water contents measured on samples of lower silty clay ranged from about 34 to 65 percent with an average of about 50 percent. Atterberg limits testing conducted on selected samples of the lower silty clay to clay gave liquid limits ranging from about 28 to 66 percent and plasticity indices ranging from about 13 to 42 percent, indicating clay of low to high plasticity. The results of the Atterberg limits tests are plotted on the plasticity chart on Figure A5.

A summary of the natural water contents, Atterberg limits, and undrained shear strengths for this deposit from the current investigation is shown on Figure 1.

The result of two grain size distributions carried out on selected samples of the clay is shown on Figure A4.

For scheduling reasons, laboratory consolidation (oedometer) tests were not performed on samples obtained from the boreholes carried out for the investigation and design of the overpass structure replacement. However, consolidation tests were performed on selected samples of the silty clay obtained from the investigation for the Tall Rock Fill embankment widening (Golder Report 021-1155-12, titled, "Tall Rock Fill Embankment Widening, Highway 417") located immediately adjacent to the overpass replacement area. The results of these consolidation tests were used to calibrate the estimates of the preconsolidation pressure profiles from the CPTs carried out in CPT02-401 and CPT02-405.

4.2.5 Silty Sand and Gravel Till to Sandy Silt Till

A 0.2 m to 1.4 m thick layer of silty sand and gravel till to sandy silt till containing some gravel was encountered below the silty clay to clay stratum in all of the boreholes. Measured SPT 'N' values of 2 blows per 0.3 m of penetration to 10 blows for 0.03 m and 0.18 m of penetration (before reaching refusal on bedrock) suggest a very loose to dense state of packing. The 1966 foundation investigation also encountered this thin layer of sandy till material, about 1.0 m thick, below the silty clay stratum.

4.2.6 Sandstone Bedrock

Sandstone bedrock underlies the till and sandy silt deposits at this site. In the five sampled boreholes (02-402 to 02-404, 06-1 and 06-2), the surface of the bedrock was encountered between Elevations 70.0 m and 70.8 m. The following table summarises the bedrock surface depth and elevation as encountered at the borehole locations.

Borehole Number	Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
02-402	94.0	23.2	70.8
02-403	87.0	16.8	70.2
02-404	92.5	22.3	70.2
06-1	92.7	22.7	70.0
06-2	86.5	16.5	70.0

The sandstone bedrock at the site is a member of the Nepean Formation; it is weak to medium strong (predominantly medium strong), fresh to slightly weathered and very thinly- to medium-bedded. Rock Quality Designation (RQD) values measured on recovered bedrock core samples ranged from 0 to 90 percent (but typically 36 to 90 percent) in the upper 0.5 m of the bedrock, and from 80 to 100 percent in the lower 2.5 m of the recovered bedrock core. The typical RQD values indicate that the bedrock is generally of good to excellent quality. The discontinuities observed in the rock core are typically horizontal to sub-horizontal, associated with the bedding planes.

A description of some of the terms used in the description of the bedrock samples from this site is provided on the *Lithological and Geotechnical Rock Description Terminology* sheet that precedes the Record of Borehole sheets included with this report.

4.3 Groundwater Conditions

Two standpipe piezometers were installed within the overburden soil deposits at this site. The water levels were measured in the piezometers on January 8, 2003. These piezometers were found on August 23, 2006 and a measurement was taken in 02-402. The piezometer in borehole 02-404 was broken or plugged at 2 m depth. The observations are summarised in the following table:

Borehole No.	Location	Water level on January 8, 2003		Water level on August 23, 2006	
		Elevation (m)	Depth (m)	Elevation (m)	Depth (m)
02-402	STA 12+650, 5 m north of centerline stake	84.3	9.7	84.7	9.3
02-404	STA 12+721, 5.3 m north of centerline stake	82.0	10.5	-	-

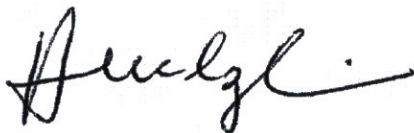
Note: piezometer in borehole 02-404 was blocked at 2 m depth on August 23, 2006

At the time of the 1966 foundation investigation, the groundwater levels were observed to be about 1 m to 3 m below the ground surface (or about Elevation 85.6 m to 82.9 m). It should be noted that groundwater levels are expected to fluctuate seasonally and are expected to rise during wet periods of the year.



4.4 CLOSURE

The technicians supervising the field work were Mr. Paul Hulan and Mr. Dave Brown from the Golder Ottawa office. The drilling company was Marathon Drilling Ltd. of Ottawa, Ontario. The laboratory testing was performed by Golder in their Mississauga lab. The Foundation Investigation Report was prepared by Mr. Chris Ng, P.Eng. and Dr. Allen Li, P.Eng. and reviewed by Dr. J. Paul Dittrich, Ph.D., P.Eng., an Associate with Golder. Mr. Fintan Heffernan, P.Eng., Golder's Designated MTO Contact for this project, conducted an independent quality review of the report.

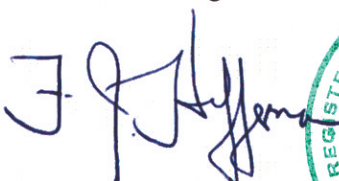
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ALL/CN/JPD/FJH/cn/all/jpd/sm

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PART B

**FOUNDATION DESIGN REPORT
OCR OVERPASS REPLACEMENT
STRUCTURE SITE 3-257 (EBL)
HIGHWAY 417
G.W.P. 4254-05-01**

5.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the proposed replacement of the EBL OCR bridge structure. The recommendations are based on interpretation of the factual geotechnical data obtained from the boreholes and CPTs advanced during the subsurface investigations at this site.

The interpretation and recommendations provided are intended only to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed structure foundations. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

5.1 General

It is understood that the proposed EBL replacement structure will consist of a 45 m long by 29 m wide, three-span, pre-cast concrete girder construction supported on concrete abutments and piers. It is further understood that the existing structure, including the abutments and piers are to be removed, however, the existing steel piles are to be left in place (as discussed below).

Based on the information provided on the General Arrangement (GA) Drawing provided by TSH dated September 2005, the grade of the new bridge deck varies between about Elevations 94.9 m and 95.9 m at the east and west abutments, respectively. The top of existing pavement at the east and west abutments is at about Elevation 93.3 m and 94.7 m, respectively. As such, grade raises of about 1.6 m to 1.2 m are required on the top of the existing approach embankments. In addition, the wider replacement structure will require wider approach embankments that necessitate the placement of up to about 2.5 m of fill within the existing median area on the north side of the existing east and west approaches.

5.2 Bridge Foundation Options

The firm to stiff silty clay found in this area is relatively compressible and makes shallow foundations impractical for this site. The proposed bridge piers and abutments should therefore be supported on deep foundations (similar to the existing bridge structures), which transfer the foundation loads to more competent bearing at depth.

Three options that could be considered for these foundations are:

- Steel H-piles driven to end-bearing on bedrock;

- Micropiles drilled and socketted into the bedrock; or
- Cast-in-place concrete caissons founded on bedrock.

Recommendations for steel H-piles, micropiles and cast-in-place concrete caissons for the bridge abutments and piers are presented in the following sections. A summary comparison of the advantages, disadvantages, relative costs, and risks associated with these foundation options is presented in Table 1 following the text of this report.

5.3 Steel H-Pile Foundations

Steel H-piles driven to found on the sandstone bedrock may be used for support of the abutments and the piers. The elevations of the pile caps are based on the Hwy 417 (EBL) OCR Overpass General Arrangement drawing (Drawing No. 1) by Totten Sims Hubicki Associate Ltd., dated September 2005.

Location	Pile Cap Elevation (m)	Pile Tip Elevation (m)	Approximate Length of Pile (m)	Reference Borehole
East Abutment	89.0	±70.0	19.0	06-1 02-404
East Pier	84.8	±70.0	14.8	06-2 02-403
West Pier	84.2	±70.8	13.4	02-402
West Abutment	90.3	±70.8	19.5	02-402 66-1

It should be noted that the native marine (Champlain Sea) clay at this site is a sensitive soil and may “flow” when subjected to disturbance (i.e. such as may be caused by the extraction of existing pile foundations). This flow is associated with a collapse of the soil structure which could result in settlements of the ground surface in the vicinity of the extracted piles. Given the proximity of the existing OCR track to the existing piles, ground surface settlements could impact the performance of the rail track. As such, it is recommended that the existing steel H-piles not be extracted but remain in place following removal of the original bridge structure. It will therefore be necessary to carefully plan the location and batter of the new piles to avoid conflicts with the existing piles during driving. Based on discussions with TSH and a review of the Foundation Plan Drawing for the existing bridge (dated June 1967) we understand that there is likely sufficient spacing between the existing piles to allow for a practical layout and installation of the new H-piles. However, given the potential for variations between the ‘design drawing’ and what may have actually been originally constructed in the field, it is recommended that a flexibility be incorporated into the new foundation design (i.e. in terms of allowable tolerances on

the location of the piles) that will allow for some changes to the actual pile layout if conflicts arise during construction.

The bridge foundations should be designed with a maximum batter of 1H:3V on the steel H-piles (for seismic considerations). In addition, as discussed below, all battered piles should be equipped with rock points for adequate seating and prevention of slippage along the bedrock surface during driving.

5.3.1 Axial Geotechnical Resistance

For steel H-piles driven to found on the sandstone bedrock, the following factored axial resistances at Ultimate Limit States (ULS) for various pile sizes may be assumed for design.

Pile Type/Size	Factored Axial Resistance at ULS (kN)
HP 310 x 110	2,000
HP 310 x 132	2,400
HP 310 x 152	2,750

The values tabulated above take into account the structural capacity limitation of the pile. A Serviceability Limit States (SLS) value is not provided because the sandstone bedrock is considered to be an unyielding material. Under these conditions, the SLS values (for 25 mm of settlement) do not govern design because the SLS value is higher than the ULS value.

Since it is recommended that the existing piles not be extracted and remain in place, there could potentially be conflicts between the old piles and the new piles during installation. In addition, as discussed in Section 5.3.4, downdrag loads need to be taken into account in the design of the pile foundations. For these reasons, consideration should be given to using a heavier pile section (as per the above table) which could reduce the total number of piles required and provide more flexibility for relocating a pile, if required, during construction. It should however be noted that the heavier pile sections may not be readily available and could be harder to source.

The pile tips for vertical piles should be suitably reinforced with flange reinforcement. The thick sensitive clay deposits and very thin basal till layer may offer only limited resistance to sliding of battered piles along the bedrock surface; in this regard and as noted above, battered piles should

be fitted with Standard Bearing Points, Titus or equivalent to enhance seating in the bedrock. The contract drawings should include a note specifying this requirement as discussed in Section 5.3.3.

It should be noted that cobbles and boulders were encountered within the fills in the boreholes advanced near the abutment locations. As such, appropriate equipment and procedures may be required to penetrate these obstructions that could be encountered during pile driving.

5.3.2 Set Criteria

Set criteria are highly dependent on pile driving hammer type and the selected pile. The set criteria can be established through a variety of methods, including empirical correlations and wave equation analyses, at the time of construction once the hammer and pile types are known. The choice of set criteria is dependent on the experience of the engineer and traditional use where a substantial database has been developed over the years. The criteria needs to be set to also avoid overdriving and possibly damaging the piles.

All pile installation/driving should be in accordance with SP 903S01. A refusal rate of 20 blows per 25 mm should not be exceeded in order to prevent/minimize damage to the hammer and the pile.

5.3.3 Pile Driving Note

The pile driving note to be added to the contract drawings is Note 4 in Clause 2.5.11 of the Structural Manual – “Piles to be driven to bedrock”. In addition, the contract drawings should also incorporate a note stating that the piles shall be equipped with flange reinforcement and Standard Bearing Points, Titus or equivalent (as discussed above).

5.3.4 Downdrag Load (Negative Skin Friction)

It should be noted that the new fill materials required for the grade raise and widening of the existing approach embankments (including the new fill to be placed on the embankment front slopes immediately adjacent to the piers) will result in an increase in the vertical stress in the silty clay deposit which underlies the bridge abutments and piers. Compression of this deposit under the stress change will lead to consolidation settlement with time due to dissipation of the excess pore pressures. If the piles are driven prior to new fill placement and/or completion of this time-dependent settlement, since the piles will be end-bearing on bedrock, a small amount of settlement of the clay relative to the pile will result in the development of negative skin friction on the piles. Therefore, negative skin friction or downdrag loads will need to be taken into account during design of the piles supporting the new abutments and piers.

The magnitude of the downdrag load acting on the piles is a function of the adhesion (skin friction) between the pile and the cohesive soils and the friction between the pile and cohesionless soils, and the surface area of the pile within the deposits that will undergo settlement following installation. The unit negative skin friction acting on a unit area along a single pile can be calculated using the following equations:

For cohesionless soils

$$f_{sn} = \beta \sigma_v' \quad \text{where}$$

f_{sn} is the unit negative skin friction (kN)
 β is the shaft resistance factor = 0.6
 σ_v' is the effective vertical (overburden) pressure (kPa)

For cohesive soils

$$q_n = \alpha \tau_u \quad \text{where}$$

q_n is the unit negative skin friction (kN)
 α is the reduction coefficient ranging from 0.5 to 1.0
 τ_u is the undrained shear strength (kPa)

For this site σ_v' , can be calculated (approximately) for design purposes as:

$$\sigma_v' = \gamma' z \quad \text{where}$$

γ' is the buoyant unit weight of soil below groundwater table (assume 8 kN/m³) and the bulk unit weight of soil above the groundwater table (assume 19 kN/m³)
 Z is the depth below pile cap elevation (kPa)

For design purposes, the following are the values of τ_u and $\alpha\tau_u$ used to calculate negative skin friction:

East Abutment and East Pier

Soil Unit	τ_u	$\alpha\tau_u$
Firm to stiff weathered silty clay crust from about Elevation 81.5 m to 79 m (abutment) and about Elevation 85 m to 82.5 m (pier)	80 kPa	35 kPa
Firm unweathered silty clay from about Elevation 79 m to 71 m (abutment) and about Elevation 82.5 m to 77 m (pier)*	40 kPa	25 kPa

*Note: neutral plane taken at elevation were settlements predicted to be less than about 12 mm.

West Abutment and West Pier

Soil Unit	τ_u	$\alpha\tau_u$
Firm to stiff weathered silty clay crust from about Elevation 88 m to 84 m (abutment) and about Elevation 84 m to 82.5 m (pier)	80 kPa	35 kPa
Firm unweathered silty clay from about Elevation 84 m to 74 m (abutment) and about Elevation 82.5 m to 74 m (pier)*	50 kPa	30 kPa

*Note: neutral plane taken at elevation where settlements predicted to be less than about 12 mm.

The total downdrag load is a function of the surface areas of the pile within the soil strata and the vertical effective stress or undrained shear strength mobilised from the top of the embedding layer down to the neutral point (Briaud, 1994). The neutral plane was assumed to be at the elevation in the soil strata below which point the foundation soil settlements are estimated to be less than about 12 mm. The load calculated in this manner is a nominal (unfactored) load. The structural engineer needs to multiply this load by a load factor of 1.25, as defined in Section 6.8.3 of the CHBDC, and include it as part of the load acting on the pile as described in the CHBDC.

Using the method described above, the estimated downdrag loads acting on a single pile at the abutment and pier foundations are summarised in the following table. The loads given are the estimated nominal (unfactored) downdrag loads acting on HP 310 x 110 steel piles for the structure.

Location	Reference Borehole	Nominal (unfactored) Downdrag Load
East Abutment	06-1 02-404	750 kN
East Pier	06-2 02-403	250 kN
West Pier	02-402	350 kN
West Abutment	02-402 66-1	575 kN

It should be noted that the downdrag loads estimated above are based on the assumption that a combination of ultra-lightweight fill (i.e. slag) and EPS fill will be used to provide the required grade raise. If conventional earth or rock fill is utilized, the downdrag loads will increase.

The estimated downdrag loads are relatively large in comparison to the axial resistance of the piles, especially at the abutments. Downdrag loads could be reduced or eliminated by installing the piles after the consolidation settlements (due to the new grade raises) are completed, however, the estimated time to complete greater than 90% of the primary consolidation is in excess of 5 to 10 years.

5.3.5 Lateral Loads (due to Horizontal Soil Deformations)

In addition to downdrag loads, the effect of lateral loading on the piles caused by horizontal soil deformations (i.e. due to consolidation of clayey silt strata and lateral spreading under new embankment loading) should also be considered in the pile design.

Where the clayey foundation soils are not preloaded prior to pile installation, there will be some additional lateral loads acting on the abutment piles. However, given the stiff to firm nature of the clayey strata and the nominal grade raises required, the magnitude of the lateral loads will be small and are difficult to quantify given the complex nature of the soil-structure interaction problem. The horizontal component of the soil deformations (i.e. lateral spreading due to the approach embankment loading on the compressible clayey silt soils) are anticipated to be on the order of less than about 15 mm after Ladd (1991). The magnitude of this deformation and the effect it could have on the integral abutments should also be considered in the design.

Lateral loads on the piles (and horizontal soil deformations) can be reduced or eliminated by constructing the embankment grade raises in the abutment areas as early as possible in the construction and allowing the settlement and lateral movement to occur prior to pile installation.

Alternatively, consideration could be given to pile supporting the ends of the abutment wing walls to provide a resistance to the rotation that could be caused by the horizontal soil deformation acting on the abutment piles.

5.3.6 Resistance to Lateral Loads

The design of piles subjected to lateral loads should take into account such factors as the batter of the piles (if any), the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moments, the soil resistance that can be mobilized, the tolerable lateral deflections at the head of the pile and pile group effects. For a longer, more flexible pile, the maximum yield moment of the pile may be reached prior to mobilization of the lateral geotechnical resistance. For design purposes, both the structural and geotechnical resistances should be evaluated to establish the governing case.

Lateral loading could be resisted fully or partially by the use of battered steel H-piles. For vertical piles, the resistance to lateral loading will have to be derived from the soil in front of the piles. Where integral abutments are under consideration, there will also be a requirement for the piles to move sufficiently to accommodate the bridge deck deflections.

The resistance to lateral loading in front of the piles may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h , is based on the following equations:

For cohesionless soils

$$k_h = \frac{n_h z}{B} \quad \text{where}$$

n_h is the constant of subgrade reaction (kPa/m)

z is the depth (m)

B is the pile diameter or width (m)

For cohesive soils

$$k_h = \frac{67 s_u}{b} \quad \text{where}$$

s_u is the undrained shear strength of the soil (kPa)

b is the pile diameter or width (m)

The following ranges for the value of n_h and s_u may be assumed in the structural analysis:

Foundation Unit	Soil Unit	Approx. Elevation (m)	s_u (kPa)	n_h (kPa/m)
East Abutment	Compact sand and gravel	89 – 81.5	-	6600
	Stiff to firm clayey crust	81.5 – 79	80	-
	Firm silty clay	79 – 71	40	-
East Pier	Stiff to firm clayey crust	85 – 82.5	80	-
	Firm silty clay	82.5 – 71	40	-
West Pier	Stiff to firm clayey crust	84 – 82.5	80	-
	Firm silty clay	82.5 – 71	50	-
West Abutment	Compact sand and gravel	90 – 88	-	6600
	Stiff to firm clayey crust	88 – 84	80	-
	Firm silty clay	84 – 71	50	-

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than eight pile diameters. Group action can be evaluated by reducing the

coefficient of lateral subgrade reaction in the direction of loading by a reduction factor, R , as follows:

Pile Spacing in Direction of Loading d = Pile Diameter	Subgrade Reaction Reduction Factor (R)
8d	1.00
6d	0.70
4d	0.40
3d	0.25

Reference: Foundations and Earth Structures – Design Manual 7.2, NAVFAC DM-7.2. Department of the Navy, Naval Facilities Engineering Command (1982).

The subgrade reaction reduction factor should be interpolated for pile spacings in between those listed above.

5.3.7 Frost Protection

The pile caps should be provided with a minimum of 1.8 m of conventional soil cover for frost protection. Alternatively, rigid insulation could be used to reduce the required thickness of soil cover over the foundation units. For preliminary design, it can be assumed that 25 mm of rigid insulation is equivalent to 0.6 m of conventional soil cover. The insulation should be installed on the abutment stem extending down from ground surface to the top of the pile cap and then extend to a distance of 1.8 m beyond the perimeter of each foundation unit.

5.4 Micropiles

As noted in Section 5.3, given the recommendation that the steel H-piles of the existing bridge foundation not be removed as part of the new construction, the potential exists for encountering the old piles during the new pile installation. In order to better deal with these unknowns and allow for more flexibility during construction (i.e. in terms of the ability to omit or move some of the new piles) consideration could be given to using micropiles for support of the new bridge abutments and piers.

For a 0.2 m diameter micropile socketted a minimum of 2 m into the fresh sandstone bedrock, a factored axial resistance at Ultimate Limit States (ULS) of 1000 kN may be assumed for preliminary design. Higher axial resistances could be achieved for longer rock sockets. The factored axial resistance (at ULS) provided is governed by the geotechnical capacity of the micropile and is based on an estimated bedrock UCS strength of 30 MPa.

A Serviceability Limit States (SLS) value is not provided because the sandstone bedrock is considered to be an unyielding material. Under these conditions, the SLS values (for 25 mm of settlement) do not govern design because the SLS value is higher than the ULS value.

The benefits of using a micropile foundation system include flexibility in the design and the ability to incorporate redundancy (or additional micropiles) into the design that could then be omitted if obstructions (i.e. old steel piles) are encountered during installation. However, it should be noted that the cost of micropiles is generally higher than that of conventional driven piles. Additional costs would also be incurred to carry out a detailed micropile analysis and to perform a sacrificial pile load test which is recommended prior to finalizing design and/or commencement of production piling.

Considering that the circumference of a 0.2 m diameter micropile is about one-half of the circumference of a HP 310 steel pile, the downdrag load on an individual micropile would be about one-half of those reported in Section 5.3.4 for the steel H-piles. Other recommendations for lateral loads, resistance to lateral loads and frost protection are as per the recommendations provided in Section 5.3. It should be noted that the maximum lateral resistance that long micropiles are capable of developing can be controlled by the structural capacity (i.e. maximum bending moment) of the micropile which can be lower than that of a conventional steel H-pile. However, when required, the cross-section of a micropile can be designed to achieve large lateral resistances (i.e. by using a steel pipe section (219 mm outer diameter - Grade 350 steel), filled with 35 MPa concrete and a central 25 mm (M25 – Grade 400) diameter bar (or larger if required for high lateral loads)). In addition, if very high lateral loads are expected, the use of inclined micropiles will significantly assist in enhancing lateral movement performance.

5.5 Cast-in-Place Concrete Caissons

Cast-in-place concrete caissons could be used for support of the abutments. The elevations of the pile caps are based on the Hwy 417 (EBL) OCR Overpass General Arrangement drawing (Drawing No. 1) by Totten Sims Hubicki Associate Ltd., dated September 2005. The estimated caisson tip elevations and approximate length of caissons are shown below.

Location	Pile Cap Elevation (m)	Caisson Tip Elevation (m)	Approximate Length of Caisson (m)	Reference Borehole
East Abutment	89.0	±69.6	19.4	06-1 02-404
East Pier	84.8	±69.6	15.2	06-2 02-403
West Pier	84.2	±70.5	13.7	02-402
West Abutment	90.3	±70.5	19.8	02-402 66-1

It is noted that the native marine (Champlain Sea) clay at this site is a sensitive soil. The disturbed clay could “flow” into the auger hole during drilled shaft installation if left unsupported. The use of a permanent casing driven into place (not vibrated or twisted) will be required in order to advance the drilled shafts with minimal loss of ground. For caissons in the local clay, the casings shall not be removed during concreting.

The sandstone bedrock at the site is considered to be generally weak to medium strong. As such, socketting of the drilled shafts into the bedrock will require rock coring or churn drilling.

5.5.1 Axial Geotechnical Resistance

For caissons socketted nominally (0.3 m) into the fresh bedrock, the design should be based on end-bearing resistance a factored axial geotechnical resistance at ULS of 7 MPa should be used. As such, the following factored axial geotechnical resistances at ULS for different caisson diameters may be assumed for design.

Caisson Size	Factored Axial Resistance at ULS (kN)
0.9 m diameter	4,450
1.5 m diameter	12,350

Serviceability Limit States resistances do not apply, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

It should be noted that the caisson excavations must be cleaned prior to concreting. Considering the length/depth of the caissons, a method such as airlifting shall be employed and tremie concreting techniques shall be used for placing concrete.

5.5.2 Downdrag Load (Negative Skin Friction)

Using the method described in Section 5.3.4, the estimated downdrag loads acting on a single caisson with a permanent steel casing are summarised in the following table. The loads given are the estimated nominal (unfactored) downdrag loads acting on the 0.9 m and 1.5 m diameter caissons for the structure.

Location	Reference Borehole	Nominal (unfactored) Downdrag Load	
		0.9 m Dia. Caisson	1.5 m Dia. Caisson
East Abutment	06-1 02-404	1,750 kN	2,850 kN
East Pier	06-2 02-403	550 kN	950 kN
West Pier	02-402	800 kN	1,350 kN
West Abutment	02-402 66-1	1,300 kN	2,200 kN

It should be noted that the downdrag loads estimated above are based on the assumption that a combination of ultra-lightweight fill (i.e. slag) and EPS fill will be used to provide the required grade raise. If conventional earth or rock fill is utilized, the downdrag loads will increase.

The estimated downdrag loads are relatively large in comparison to the axial resistance of the caissons, especially at the abutments. Downdrag loads could be reduced or eliminated by installing the caissons after the consolidation settlements (due to the new grade raises) are completed, however, the estimated time to complete greater than 90% of the primary consolidation is in excess of 5 to 10 years. Alternatively, it may be feasible to construct the caissons with a permanent lining and bentonite slurry “slip” layer; however, such construction may also prove costly. Recommendations for this type of construction can be provided if it is determined that caisson foundations will be considered further.

5.5.3 Lateral Loads and Resistance to Lateral Loads

The effects of lateral loading on the caisson, the resistance to lateral loading developed by the soils in front of the caissons, and the reductions due to group effects, may be assessed as per the recommendations in Section 5.3.5 and 5.3.6.

5.5.4 Frost Protection

The pile caps should be provided with a minimum of 1.8 m of soil cover for frost protection.

5.6 Retaining Wall

A permanent retaining wall, up to 2.9 m in height and about 64 m in length is to be located along the toe of the front slope of the east approach embankment to the OCR Overpass EBL Replacement structure immediately adjacent to the NCC pathway at its base.

Based on the boreholes advanced at the site that are closest to the alignment of the proposed retaining wall (i.e. boreholes 02-403 and 06-2 located within about 3 m and 5 m of the north end and central portion of the wall, respectively) the subsoils are expected to consist of a very loose to compact, sand/sand and gravel fill overlying a very stiff to firm silty clay. At the closest borehole locations, the fill materials range in thickness from about 1.5 m to 4.3 m and the upper weathered crust of the silty clay stratum ranges in thickness from about 2.7 m to 6.1 m (underlain by a softer silty clay stratum). The following table outlines the information currently available regarding the proposed height of wall and subsoils in this area.

Approximate Location along Wall	Elevation of Existing Ground Surface (m)	Elevation of Proposed Top of NCC Pathway (m)	Elevation of Proposed Top of Retaining Wall (m)	Height of Wall* (m)	Range of Existing Fill Elevations (m)	Range of Silty Clay Crust Elevations (m)
North End (BH 02-403)	87.0	87.2	88.5	1.3	87 to 85.5	85.5 to 79.4
Middle (BH 06-2)	86.5	86.6	88.5	1.9	86.5 to 82.2	82.2 to 79.5

*Note: wall height is up to 2.9 m over an approximately 15 m long section near south end.

It should be noted that no boreholes have been advanced near or along the portion of the retaining wall alignment that extends south of the EBL bridge structure (i.e. as much as 30 m beyond the limits of our investigated area). In our opinion, the risk of not having a complete subsurface model along the proposed wall alignment is considered to be of medium level.

Given the subsoil conditions present in the area, it is anticipated that the proposed retaining wall will be founded on either shallow foundations placed on the compact sand and gravel fills, or on deep foundations (steel H-piles/soldier piles) advanced through the loose to compact fills and founded within the very stiff to stiff weathered silty clay crust. The following concepts can be considered for the proposed retaining wall:

- Soldier pile and concrete panel wall;
- Bin or crib wall;
- Cast-in-place concrete cantilever retaining wall;
- Mechanically-reinforced soil retaining wall system;

Foundation engineering recommendations and construction considerations applicable to each of these types of retaining wall systems are provided in the following sections. It should be noted that a Gabion wall system was also considered and is technically a feasible alternative, however, given the proximity to the NCC pathway and its poor facing aesthetics, it is not recommended at this location. A summary comparison of the advantages, disadvantages, relative costs, and risks associated with each of these wall options is presented in Table 2 following the text of this report.

From a foundations perspective, the Soldier Pile and Concrete Panel Wall is ranked as the preferred alternative at this site. However, given the lack of subsurface information over a portion of the wall length and the need to maintain the tip of the piles above the base of the silty clay crust (as discussed below) there is some risk associated with this alternative.

5.6.1 Soldier Pile and Concrete Panel Wall

A soldier pile and concrete panel wall would consist of steel H-piles set in concrete filled caissons advanced through the fills and into the very stiff to stiff weathered silty clay crust. This approach is recommended over a driven pile approach in order to provide as much lateral resistance as possible. Given the nominal wall heights anticipated (i.e. on the order of about 1 m to 3 m), it is anticipated that the soldier pile wall could be designed as a cantilever system without the need for lateral support from permanent soil anchors.

Axial Geotechnical Resistance

For design, the factored axial resistance at Ultimate Limit States (ULS) for a 0.6 m diameter concrete caisson/socket founded within the upper portion of the very stiff to stiff weather silty clay crust may be taken as 120 kN. The axial resistance at Serviceability Limit States (SLS) for 25 mm of settlement may be taken as 75 kN.

In order to achieve the axial resistances the base of the caisson/socket must be located at least 1.5 m above the bottom of the silty clay crust. As such, based on the subsurface information available at boreholes 02-403 (near north end of wall) and 06-2 (near middle of wall), the tips of the soldier piles/base of the socket should not extend below about Elevation 81 m.

Resistance to Lateral Loads

The wall support must be designed to accommodate the loads applied from the earth pressures of the sloping backfill as well as from any surcharge pressures from area, line or point loads, if applicable. Guidelines for estimating the active lateral earth pressures acting on the wall are provided in a subsequent section.

It is assumed that the soldier pile wall will be unsupported and can be designed as a cantilevered system and as such, no recommendations are provided for the design of soil anchors.

The soldier piles will be extended through the existing sand and gravel fill, and into the underlying stiff to very stiff silty clay crust to develop sufficient lateral resistance. Passive toe restraint to the soldier piles may be determined using a triangular pressure distribution acting over an equivalent width equal to three times the pile socket diameter, using the following equation:

$$p_p = 3 B(K_p \gamma z)$$

- where z = the depth of the soldier pile below the ground surface in front of the wall at any point (in metres),
 K_p = is the coefficient of passive earth pressure for the ground in front of the soldier piles
 γ = soil unit weight (effective unit weights to be used for soils below groundwater table assumed to be at Elevation 83 m)
 B = socket diameter, in metres

The following table provides the coefficient of passive lateral earth pressure, K_p , and unit weight of the soil to be used in calculating the passive lateral resistance.

<i>Soil Deposit</i>	<i>K_p</i>	<i>γ (kN/m³)</i>
<i>North End of Wall</i>		
Sand and gravel fill (Elevation 87.2 m to 85.5 m)	3.2	20
Silty clay crust (Elevation 85.5 m to 83 m)	3.0	17
Silty clay crust (Elevation 83 m to 79.4 m)	3.0	7.2
Silty clay (below Elevation 79.4 m)	2.8	7.2

<i>South End of Wall</i>		
Sand fill (Elevation 86.6 m to 83 m)	3.2	20
Sand fill (Elevation 83 m to 82.2 m)	3.2	10.2
Silty clay crust (Elevation 82.2 m to 79.5 m)	3.0	7.2
Silty clay (below Elevation 79.5 m)	2.8	7.2

It should be noted that the given values of K_p include the effect of wall friction and have been factored down (by 1.5) to account for strain compatibility, since a large passive strain would be required for full mobilization of the passive resistance. The CHBDC recommends neglecting the passive resistance of the soil within the frost zone (i.e. to a depth of 1.8 m at this site). As such, the lateral resistance of the soil down to a depth of 1.8 m below the final ground surface should not be relied on in the design.

If it is necessary for the design to include the available passive resistance within the 1.8 m frost zone (i.e. over the approximately 15 m long section of wall where its height is in excess of 2 m) it is recommended that rigid insulation be installed to protect the soil in this area from freezing. To provide adequate frost protection, a minimum thickness of 75 mm of rigid insulation should be installed. The insulation should extend along the length of the wall where it is required and out from the face of the wall to a distance that is the equivalent of the depth of frost penetration (i.e. 1.8 m). To provide adequate protection to the insulation and minimize the chance for differential icing on the NCC pathway in front of the wall, it is recommended that the insulation be covered with 0.5 m of conventional soil cover.

It should also be noted that the range of values and depths of soils provided in the table above are based only on the limited borehole information in the vicinity of the proposed wall. The actual subsurface conditions encountered may vary from those assumed above. In addition, given the softer nature of the silty clay soils below the weathered crust, and as per the recommendations in the above section on axial geotechnical resistance, it is recommended that the pile sockets be terminated 1.5 m above the base of the silty clay crust and that the lateral resistance in the underlying firm silty clay not be relied on to support the wall.

Drainage and Insulation Requirements

Suitable drainage and insulation should be provided to the back of the wall lagging which will be supporting the existing sand and gravel fill and the new slag fill on the front slope of the approach embankment.

In order to maintain stability of the existing front slope, it is recommended that the amount of excavation into the toe of the existing slope be minimized. As such, to provide the required drainage behind the wall and to limit the excavation, a prefabricated drainage sheet should be utilized. The drainage sheet should be installed on the back of the wall (over the full height) and

connected to a subdrain running along the length of the wall as well as to weeping holes extending through the wall.

To prevent freezing behind the wall in areas adjacent to the existing embankment fill (and assuming that the ultra-lightweight slag used for the new backfill behind the wall is non-frost susceptible), it is recommended that an extruded polystyrene insulation (specifically designed for use with the chosen type of wall facing – i.e. most likely to be precast concrete panels) be installed.

5.6.2 Cellular Steel Retaining Wall System (Bin or Crib Type Wall)

A cellular steel wall is a gravity wall consisting of a system of rectangular closed-face cells sometimes referred to as bins or cribs. The cells are formed from lightweight, deep-corrugated steel modular components bolted together at the site and filled with earth. The fill material plus the weight of the steel acts as a gravity retaining wall against the soil pressures from above and behind the wall. The flexible design adjusts to minor ground movements without cracking or bulging of the wall face.

Use of this system is considered appropriate for construction of the proposed retaining wall, given that the maximum wall height will be about 3 m. The founding strata along the length of the wall (based on the existing boreholes) is expected to be variable and consist of sand/sand and gravel fill and/or silty clay; this will result in some total and differential settlement along the wall on the order of up to about 45 mm and 25 mm, respectively, which should be within the allowable limits for this system. Preloading the area for a 6 month period (i.e. by placing the new fill on the front slope prior to wall construction) is expected to reduce the total and differential settlements to about 30 mm and 20 mm, respectively. However, preloading may not be an option at this location as the preloading fills would extend onto and block the existing NCC pathway.

The cellular steel wall is expected to be founded predominantly on the existing sand and gravel fill assuming it will be placed at a nominal depth below the proposed ground surface adjacent to the NCC pathway. For the bin wall founded on compact sand and gravel fill, a factored geotechnical resistance at ULS of 175 kPa may be assumed for design (assuming a bin width of $0.55 \times \text{height of wall} = 1.6 \text{ m}$). The geotechnical resistance at SLS for 25 mm of settlement is estimated to be 120 kPa. However, the actual performance of the wall be governed by the dimensions of the bin wall and by the thickness of new fill to be placed behind the wall and along the front slope of the east approach embankment. As noted above, for the thickness of new slag fill to be retained behind the wall and on the embankment front slope, settlements of up to about 45 mm are expected to occur.

A coefficient of friction equal to 0.30 may be assumed between the steel base of the bin wall and the sand/sand and gravel fill founding soils.

The internal stability of the cellular wall should be checked by the supplier/designer. For the proposed height of wall (i.e. up to 2 m) and for the proposed 1.75H:1V slope (30°) of the backfill behind the wall, an adequate factor of safety against failure is obtained for the wall for global stability.

The bin wall should be constructed on a compacted granular leveling pad placed on the compact sand/sand and gravel fill soils. In this regard, all topsoil should be removed and any existing loose or very loose fill materials should be excavated and recompact and the subgrade should be inspected by qualified geotechnical personnel (in accordance with Special Provision SP 902S01) prior to the placement and construction of the bins. Only nominal embedment, on the order of about 0.5 m, is required for this wall type.

5.6.3 Cast-in-Place Concrete Cantilever Retaining Wall

For this option, the concrete cantilever wall could be supported on shallow spread footings. Given that the proposed ground surface of the NCC pathway is to vary from about Elevation 87.2 m (north end) to 86.6 m (south end), the highest acceptable founding elevation will vary from about 85.4 m (north end) to 84.8 m (south end) in order to provide a minimum of 1.8 m of earth cover for protection against frost penetration. Spread footings at this level will be placed on either the silty clay crust or on the sand/sand and gravel fill.

The settlement of footings founded on the compact sand/sand and gravel fill or on the stiff to very stiff silty clay crust will be dependent on the footing size and configuration as well as on the loading from the thickness of new fill to be placed behind the wall and along the front slope of the east approach embankment. For the proposed thickness of fill behind the retaining wall, it is estimated that total and differential settlement on the order of up to about 45 mm and 25 mm, respectively could occur. Preloading the area for a 6 month period (i.e. by placing the new fill on the front slope prior to wall construction) is expected to reduce the total and differential settlements to about 30 mm and 20 mm, respectively. However, preloading may not be an option at this location as the preloading fills would extend onto and block the existing NCC pathway.

The factored geotechnical resistance at ULS which should be used for footing design at the founding elevations noted above is 175 kPa. The geotechnical resistance at SLS for 25 mm of settlement is estimated to be 120 kPa. However, the actual performance of the wall be governed by the dimensions of the wall and by the thickness of new fill to be placed behind the wall and along the front slope of the east approach embankment. As noted above, for the thickness of new

slag fill to be retained behind the wall and on the embankment front slope, settlements of up to about 45 mm are expected to occur.

The geotechnical resistances provided are given under the assumption that the loads will be applied perpendicular to the surface of the footings; where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the Canadian Highway Bridge Design Code (CHBDC) and the footing should be designed such that the load resultant remains within the central 1/3 of the foundation width and length dimensions. The footing should be designed with a minimum width of 1 m.

Resistance to lateral forces / sliding resistance between the concrete footing and the subsoils should be calculated in accordance with the CHBDC assuming an unfactored coefficient of friction of 0.45 between the concrete and the sand and gravel or silty clay crust founding soils.

Any existing loose or very loose fill materials should be excavated and recompact and the subgrade should be inspected by qualified geotechnical personnel (in accordance with Special Provision SP 902S01) prior to the placement and construction of the wall.

5.6.4 Mechanically-Reinforced Soil Retaining Wall (Retained Soil System)

A mechanically-reinforced soil retaining wall system (Retained Soil System or RSS wall) consists of granular fill placed and compacted in layers, and reinforced with metal or fabric strips or grids. A facing material, typically pre-cast concrete panels mechanically fastened to the reinforcing strips or grids, is used to form the face of the reinforced soil structure and to prevent the loss of fill material. Use of this system may be appropriate for construction of the proposed retaining wall, given that the maximum wall height will be about 3 m. The founding strata along the length of the wall (based on the existing boreholes) is expected to be variable and consist of sand and gravel fill and/or silty clay. Based on the subsurface conditions and the new thickness of ultra-lightweight slag fill to be placed behind the wall on the front slope of the approach, total and differential settlement along the wall are estimated to be on the order of up to about 45 mm and 25 mm, respectively. Preloading the area for a 6 month period (i.e. by placing the new fill on the front slope prior to wall construction) is expected to reduce the total and differential settlements to about 30 mm and 20 mm, respectively. However, preloading may not be an option at this location as the preloading fills would extend onto and block the existing NCC pathway. Given that the normal facing of an RSS system is reportedly flexible enough to accommodate differential settlement of up to 1% of the wall length, for the proposed 64 m long wall, the expected differential settlements should be tolerable.

Considering the location of the wall (i.e. adjacent to the NCC Pathway with a 400 series highway passing above the RSS wall), the Site Performance Rating is classified as Medium and the Appearance Criteria is classified as High for this site.

The reinforced earth mass is expected to be founded predominantly on the existing sand and gravel fill assuming it will be placed at a nominal depth below the proposed ground surface adjacent to the NCC pathway. For the reinforced earth mass founded on compact sand/sand and gravel fill, a factored geotechnical resistance at ULS of 200 kPa may be assumed for design (assuming an embedment width of 0.8 x height of wall = 2.3 m). The geotechnical resistance at SLS for 25 mm of settlement is estimated to be 120 kPa. However, the actual performance of the wall be governed by the dimensions of the RSS wall and by the thickness of new fill to be placed behind the wall and along the front slope of the east approach embankment. As noted above, for the thickness of new slag fill to be retained behind the wall and on the embankment front slope, settlements of up to about 45 mm are expected to occur.

A coefficient of friction equal to 0.55 may be assumed between the granular fill of the RSS wall and the sand/sand and gravel fill founding soils.

The internal stability of the mechanically-reinforced soil wall should be checked by the RSS supplier/designer. For the proposed height of wall (i.e. up to about 3 m) and for the proposed 1.75H:1V slope (30°) of the backfill behind the wall, an adequate factor of safety against failure is obtained for the wall for global stability.

The granular fill material for the reinforced soil system should be placed on the compact sand/sand and gravel fill soils. In this regard, all topsoil should be removed and any existing loose or very loose fill materials should be excavated and recompact and the subgrade should be inspected by qualified geotechnical personnel (in accordance with Special Provision SP 902S01) prior to the placement of the granular materials. Only nominal embedment, on the order of about 0.5 m, is required for this wall type.

5.6.5 Lateral Earth Pressures for Wall Design

The active pressures to be considered for the wall design should be based on a triangular earth pressure distribution. The unfactored triangular earth pressure distribution (p in kN/m²; increasing with depth), can be calculated as follows:

$$p_a = K_a \gamma H$$

where H = the height of the wall above the base of the sockets, in metres

- $K_a =$ 0.7 for the sloping granular or slag fill behind the wall (assumed to be sloping at an angle of 30° to the horizontal)
- $\gamma =$ soil unit weight, where $\gamma_{\text{fill}} = 20 \text{ kN/m}^3$ for the compacted sand and gravel and $\gamma_{\text{fill}} = 12 \text{ kN/m}^3$ for the ultra-lightweight slag

Other recommendations for lateral earth pressure design should be as per those provided in Section 5.8. The Seismic Active Pressure Coefficients, K_{AE} for the yielding wall Case I and Case II should be taken as 0.88 and 0.82, respectively to account for the sloping backfill.

5.6.6 Design and Construction Considerations

5.6.6.1 Soldier Pile Wall

The following must be considered to ensure that the augered caisson sockets in which the steel H-piles will be founded can be successfully installed at this site:

- Installation of the caisson sockets may require special low head room augering equipment in order to meet the height restrictions imposed by the vertical clearance under the existing EBL structure.
- Although no cobbles and boulders were encountered during the drilling of boreholes 02-403 and 06-2 (the closest boreholes to the wall alignment), cobbles and boulders were encountered in the existing fills at other locations adjacent to the existing bridge structure. It is recommended that provision be made in the Contract Documents to ensure that the Contractor is equipped to handle such obstructions if they occur.
- If deep caisson sockets are required for support of the cantilever wall, it should be noted that augering below about Elevation 83 m may encounter the groundwater table and there could then be a need to employ temporary liners for construction/installation.

5.6.6.2 RSS or Bin Wall

For the proposed wall height (i.e. up to about 3 m), construction of an RSS wall or a bin-type wall will require removal of a portion of the existing fill material that comprises the front slope of the east approach. The typical width of these types of walls is as follows:

- RSS Wall – width = $0.8 \times \text{height}$; or an equivalent width of up to about 2.3 m.
- Bin Wall – width = $0.55 \times \text{height}$; or an equivalent width of up to about 1.6 m.

As such, based on the above, excavations extending as much as 1.6 m to 2.3 m (depending on the wall type) into the toe of the existing front slope will be required for construction. In considering these wall types, the structural designers must ensure that the stability of the existing bridge abutment and its pile cap will not be compromised by the short-term removal of this material. It should be noted that in this regard, an advantage of the Bin wall system over the RSS wall system is that by the nature of its design, the bin wall is modular and lends itself to construction in small, short sections. Removal of only small portions of the existing front slope fill (followed by installation and backfilling of a bin segment prior to starting the adjacent one) will enhance stability due to three-dimensional effects. Regardless, for both of these wall types, short-section construction (to enhance front slope stability) should be adopted for the length of wall located below the existing EBL structure.

5.6.6.3 Shallow Foundations for Concrete Cantilever Wall

At the proposed retaining wall location, excavations for shallow spread footings will extend a minimum of 1.8 m below the elevation of the proposed NCC pathway (or about 1.6 m to 1.7m below the existing ground surface), may partially cut into the existing approach embankment front slope, extend through the predominantly loose to compact sand/sand and gravel fill and, in places into the very stiff to stiff silty clay crust. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act and Regulations for Construction Activities. The very loose to compact sand and gravel fills at this site would be classified as Type 3 soil. Temporary open-cut slopes should be maintained no steeper than 1 horizontal to 1 vertical (1H:1V). Where space restrictions dictate, and considering the need to maintain the stability of the immediately adjacent existing front approach embankment fill slope, the excavation should be carried out in small sections (i.e. short section construction) and within a braced excavation or a trench box.

The silty clay crustal soils and the very loose fills (where encountered at the excavation subgrade level) may be sensitive to disturbance from ponded water, construction traffic and frost. All foundation excavations should be inspected by experienced geotechnical personnel prior to filling or concreting to ensure that soil having adequate bearing capacity has been reached and that the bearing surfaces have been properly prepared. Provision should be made to sub-excavate below founding level where unsuitable subgrade soils are encountered, and to replace and remove material with compacted granular soil or lean mix concrete.

Groundwater seepage into the excavations through the fills could occur, although based on the groundwater levels measured in the piezometers installed in boreholes 02-402 and 02-404 (the closest piezometers to the proposed wall alignment), the groundwater table is likely to be located at about Elevation 83 m in the vicinity of the wall (i.e. below the proposed founding level of the

concrete cantilever wall). As such, groundwater seepage is expected to be minor and pumping from properly filtered sumps or a filtered drain located at the base of the excavation outside of the actual footing limits will provide adequate groundwater control during foundation excavations. Surface water run-off should be directed away from the excavation.

5.7 Site Coefficient

For seismic design purposes, the Site Coefficient, S , for this site in accordance with Section 4.4.6 of the CHBDC may be taken as 1.5, consistent with Soil Profile Type III.

5.8 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and any associated wing walls / retaining walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

The following recommendations are made concerning the design of the walls. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope in accordance with CHBDC Section 6.9.1(e) and 6.9.2.2 and the associated *Commentary to the CHBDC*.

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' Type II but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 percent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3501.00 and 3504.00.
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with CHBDC Section 6.9.3 and Figure 6.9.3. Compaction equipment should be used in accordance with Special Provision SP105S10. Other surcharge loadings should be accounted for in the design, as required.
- The granular fill may be placed either in a zone with width equal to at least 1.8 m behind the back of the wall stem (see Case I in Figure C6.9.1(l)(i) of the *Commentary*

to the CHBDC) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (see Case II in Figure C6.9.1(1)(ii) of the *Commentary to the CHBDC*).

- For Case I, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used assuming the use of Select Subgrade Material (SSM):

Soil unit weight:	20 kN/m ³
Coefficients of static lateral earth pressure:	
Active, K_a	0.33
At rest, K_o	0.50
Passive, K_p	3.0

- For Case II, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:

	GRANULAR 'A'	GRANULAR 'B' TYPE II
Soil unit weight:	22 kN/m ³	21 kN/m ³
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.27
At rest, K_o	0.43	0.43
Passive, K_p	3.7	3.7

- Where EPS is installed behind the abutment wall, the pressures acting over the depth of the EPS may be calculated as follows:

EPS unit weight:	0.5 kN/m ³
Coefficients of static lateral earth pressure:	
Active, K_a	0.11
At rest, K_o	0.11
Passive, K_p	9.0

If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure, may be taken as:

- Rotation of approximately 0.002 about the base of a vertical wall;
- Horizontal translation of 0.001 times the height of the wall; or
- A combination of both.

If integral or semi-integral abutment design allows for movement of the bridge deck ends, passive earth pressures may be used in the geotechnical design of the structure. The movements required to fully mobilize passive pressure or resistance are much larger than those required to mobilize active pressure. In practice, movements may not be sufficient to mobilize the full passive resistance. The movement to allow passive pressures to develop within the backfill may be taken as:

- Rotation of approximately 0.1 about the base of a vertical wall;
- Rotation of approximately 0.02 about the top of a vertical wall;
- Horizontal translation of 0.05 times the height of the wall; or
- A combination of the above.

A resistance factor equal to 0.5 should be applied to the calculated total passive resistance (in accordance with Table 6.6.2.1 of the CHBDC).

It should be noted that the passive pressures in front of the abutment wall (i.e. on the front embankment slope) should not be relied on in design given the downward sloping surface of the fill in this area, the potential for disturbance of the fill due to frost action and the potential for lack of compaction in this area.

Seismic (earthquake) loading must also be taken into account in the design in accordance with Section 4.6 of the CHBDC. In this regard, the following should be included in the assessment of lateral earth pressures:

- Seismic loading will result in increased lateral earth pressures acting on the abutment stem and retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to Table A3.1.7 of the CHBDC, this site is located in Seismic Zone 4. The site-specific zonal acceleration ratio (A) for Ottawa is 0.2. Based on the overburden soil stratigraphy at the site and the embankment heights, a 50 per cent amplification of the ground motion may occur (i.e. Site Coefficient, $S=1.5$), resulting in an increase in the ground surface acceleration from 0.2g to 0.3g. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of $A = 0.3$.
- In accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its *Commentary*, for structures which allow lateral yielding, the horizontal seismic coefficient, k_h , used in the calculation of the seismic active pressure coefficient, is taken as 0.5 times the zonal acceleration ratio (i.e. $k_h = 0.15$). For structures that do not allow lateral yielding, k_h is taken as 1.5 times the zonal acceleration ratio (i.e. $k_h = 0.45$). The seismic active and passive earth pressure coefficients are also dependent on the vertical component of the

earthquake acceleration, k_v . Three discrete values of vertical acceleration are typically selected for analysis, corresponding to $k_v = +2.3k_h$, $k_v = 0$, and $k_v = -2/3 k_h$.

- The following seismic active earth pressure coefficients (K_{AE}) for the two cases (Case I and Case II) may be used in design; these coefficients reflect the maximum K_{AE} obtained using the k_h and three values of k_v as described above. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat.

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I	Case II		EPS
		Granular A	Granular B Type II	
Yielding wall	0.42	0.35	0.35	0.12
Non-yielding wall	1.75	1.24	1.24	0.47

Note : These CHBDC seismic K_{AE} values include the effect of wall friction ($\delta = \phi'/2$).

- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A (mm), where A is the design zonal acceleration ratio of 0.3. This corresponds to displacements of up to 75 mm at this site.
- The earthquake-induced dynamic active lateral pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$p = K \gamma' d + (K_{AE} - K) \gamma' H$$

Where: p is the total (static plus seismic) pressure distribution (kPa)
 K is either the static active earth pressure coefficient (K_a) or the static at rest earth pressure coefficient (K_o);
 K_{AE} is the seismic active earth pressure coefficient;
 γ' is the effective unit weight of the soil (kN/m^3), as given previously
 d is the depth below the top of the wall (m); and
 H is the height of the wall above the toe (m)

- The following seismic passive earth pressure coefficients (K_{pE}) may be used in design of the bridge deck ends for integral and semi-integral abutment design; these coefficients reflect the maximum K_{pE} obtained using the k_h and three values of k_v as described above. It should be noted that these seismic earth pressure coefficients

assume that the back of the wall/deck is vertical and the ground surface behind the wall is flat.

SEISMIC PASSIVE PRESSURE COEFFICIENTS, K_{PE}

Case I (sand)	Case II		EPS
	Granular A	Granular B Type II	
4.3	6.4	6.4	20

- The earthquake-induced dynamic passive lateral pressure distribution, which is to be subtracted from the static passive earth pressure distribution, is a linear distribution with maximum pressure at the base of the wall and minimum pressure at the top (i.e. a triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$p = K_p \gamma' d - (K_{PE} - K_p) \gamma' H$$

Where: p is the total (static plus seismic) pressure distribution (kPa)
 K_p is the static passive earth pressure coefficient;
 K_{PE} is the seismic passive earth pressure coefficient;
 γ' is the effective unit weight of the soil (kN/m³);

- taken as soil unit weights given above for fill materials
- taken as 19 kN/m³ for the native materials
- taken as 0.5 kN/m³ for the EPS

d is the depth below the top of the wall (m); and
 H is the height of the wall above the toe (m)

5.9 Approach Embankment Design

The existing east-bound approaches to the OCR overpass bridge have net embankment heights of about 7.5 m each with side slope profiles of approximately 2H:1V to the south and moderate swales into the median to the north. The proposed embankment widening is to be about 10 m, which is to be accommodated mostly by placing additional fill within the median area, however, some new fill will be required to be placed on the south slopes and on the front slopes (i.e. towards the OCR tracks) of each approach. Grade raises of about 1.2 m to 1.6 m are required on the top of the existing approach embankments while up to about 2.5 m of fill is required within the existing median area. Erosion of the south slope of the west embankment is presently taking place from two culverts at about Station 12+600 and from a seepage source at about Station 12+620.

5.9.1 Stability

The stability of the proposed wider and higher approach embankments has been examined for selected sections that were considered to be representative of the critical conditions at the approaches.

For the west approach, a critical embankment cross-section at Station 12+610 was selected for the analysis based on field observations that indicate the presence of a natural slope on the south side of the embankment toe in this area. The natural slope is approximately 5 m high and about 100 m long with a profile of about a 2.5H:1V. The total new height of slope at this location (including the effects of the natural slope at the toe) will be about 14 m. For the east approach, a critical embankment cross section at Station 12+722 was selected for the stability analysis based on the configurations of the existing embankment and corresponding proposed design sections. The total new height of slope at this location will be about 9 m

The stability of the transverse sections of both the existing embankments and the proposed embankments at the design grade was examined. In addition, slope stability analyses of the longitudinal sections were also carried out to assess the front slopes below the bridge with the OCR tracks located at the toe.

Analysis Method

Slope stability analyses were performed using limit equilibrium methods employed in the commercially available program SLOPE/W Version 6.19 (Geo-Slope International Ltd. 2006). The minimum factor of safety against slope failure was computed using the Morgenstern-Price method of analysis. For all analyses, the factor of safety of numerous potential failure surfaces was computed in order to establish the minimum factor of safety. Different failure modes and mechanisms were considered in the analyses including potential shallow, deep-seated, circular and non-circular slip surfaces. Both total stress and effective stress parameters, as applicable for the appropriate soil types, were used in the analysis as described below.

Design Criteria

A target factor of safety of 1.3 is normally used for the design of embankment slopes under static conditions. This factor of safety is considered adequate for the embankments at this site considering the design requirements and the field data available. The stability analyses were performed to check that the target minimum factor of safety was achieved for the proposed embankment heights and geometries.

Soil Properties

As is discussed in Section 4.2, the original ground surface at the site is underlain by an extensive deposit of silty clay that provides foundation support for the highway embankment. Therefore, the undrained shear strength of the silty clay (s_u) is the governing parameter for assessing the stability of the proposed wider and higher approach embankments. The undrained shear strength of the silty clay strata was assessed based on the results of field vane tests, CPT profiling and inferred from laboratory consolidation (oedometer) tests. The field vane strengths were corrected using the following method and a Bjerrum correction factor of 0.90 based on the average plasticity of the silty clay deposits.

$$s_{u(mob)} = \mu s_{u(FV)} \quad (\text{after Bjerrum, 1973})$$

where :

- $s_{u(mob)}$ = average mobilized undrained shear strength (kPa)
- $s_{u(FV)}$ = undrained shear strength from field vane test (kPa)
- μ = Bjerrum's correction factor based on Plasticity Index
= 0.9 on average for the silty clays at this site

The following equation was used to estimate the undrained shear strength profile of the silty clay from the results of the CPT (Lunne, 1997). It should be noted that the factor, N_k , is assessed based on the measured undrained shear strength from the field vane tests.

$$s_u = \frac{q_t - \sigma_{vo}}{N_k} \quad \text{where}$$

- s_u is the average mobilised undrained shear strength (kPa)
- q_t is the net tip resistance (kPa)
- σ_{vo} is the total vertical stress (kPa)
- N_k is the correlated factor calculated as 18 for this site

The corrected undrained shear strength profiles are presented on Figure 1. The results of the boreholes and CPTs advanced through the embankment indicate that, below this previously loaded area, the weathered silty clay crust has an undrained shear strength of about 200 kPa decreasing linearly to about 50 kPa, and that the unweathered silty clay below the crust has an undrained shear strength ranging from about 35 kPa and 75 kPa. It should be noted that the undrained shear strength used in the analysis to represent the operational strength in the weathered silty clay crust has been estimated to be 75 kPa for this site based on the procedures recommended by Lefebvre et al. (1978) and Tavenas and Leroueil (1980).

The shear strength profile of the original silty clay deposit (i.e. outside the zone of influence of the existing embankment loading) was inferred from the borehole 06-4 (advanced at the toe of the natural slope immediately south of the west approach embankment, where only about 0.7 m of fill was encountered) and from the old borehole 66-1 (advanced in the area of the southeast corner of the west approach prior to embankment construction). The testing results from borehole 06-4

(borehole log included in the Golder Report 021-1155-12, titled, “Tall Rock Fill Embankment Widening, Highway 417”) indicate that the weathered crust beyond the toe of the slope is relatively thin and underlying silty clay has an undrained shear strength ranging from about 25 kPa and 35 kPa. The testing results from borehole 66-1 (borehole log included in Appendix C) indicate that at this location the crust is slightly thicker and that the range of undrained shear strengths is similar.

The silty clay under the existing embankment has likely experienced significant strength gain as a result of consolidation since completion of the original highway. The measured relatively high undrained shear strength of the clay below the embankment (relative to that measured at the toe) is a manifestation of this strength gain. The strength of the silty clay below the embankment slope was assumed in the analysis to be the average strength of the consolidated silty clay below the embankment and the original silty clay beyond the embankment toe.

The undrained shear strength profiles used in the stability analyses for each critical section were assessed based on the results of the boreholes and CPTs in the vicinity of the embankment section. The specific design strength values for each zone and layer of the silty clay deposit at different locations are presented on Figures 3 through 8. Also shown on Figures 3 through 8 are the results of the stability analyses, which are to be discussed below.

Effective strength parameters for the embankment fill materials and cohesionless till deposit are estimated based on Golder’s experience for similar materials and are also presented on Figures 3 through 8.

Analysis Results

The results of the stability analyses (along with the corresponding figure) are summarised as in the following table.

Embankment Slope	Section	Station	Reference Test Locations	Min. Factor of Safety	Figure
Existing East Approach (south slope)	Transverse	12+722	06-1, 02-404 and CPT02-405	1.6	3
Proposed East Approach (south slope)				1.3	4
Existing West Approach (south slope)	Transverse	12+610	06-4, 02-402 and CPT02-401	1.2	5
Proposed West Approach (south slope)				1.1	6
Proposed East Front Slope	Longitudinal	--	06-1 and 06-2	1.6	7
Proposed West Front Slope	Longitudinal	--	02-402 and CPT02-401	1.3	8

The results shown on Figures 3 and 4 indicate that the transverse sections of both the existing and proposed east approach embankments have factors of safety of about 1.3 or greater against potential deep-seated failure through the silty clay deposit.

However, as shown on Figures 5 and 6, the transverse sections for the west approach embankment the factor of safety against potential deep-seated failure is calculated to be about 1.2 and 1.1 for the existing embankment and proposed embankment, respectively. The results suggest that the factor of safety of the existing west embankment is currently below the acceptable minimum value of 1.3 that is required in the design criteria for static loading conditions. The 5 m high natural slope to a watercourse on the south side of the toe of this embankment is considered to be the main factor that is influencing the stability of the embankment. Therefore measures have to be taken to improve the stability of the embankment prior to construction of the proposed grade raise.

Slope stability analyses were also undertaken to examine potential instability through the embankment fill (i.e., shallow failure mechanism) and the results indicate that the factor of safety against potential shallow failure for the west approach embankment (with a maximum side slope of 2H:1V) is greater than 1.3. However, the factor of safety for shallow failures on the east approach embankment at Sta. 12+722, where the proposed embankment profile is to be approximately 1.3H:1V along the steep portion of the slope, is about 1.1 and 1.2 for assumed effective friction angles of the fill material of 34° and 38°, respectively. Therefore, it is recommended that the side slope of the approach embankments not be steeper than 2H:1V at any location.

Figures 7 and 8 (longitudinal sections) show that the front slopes of the approaches will generally have a factor of safety greater than 1.3 after the grade is raised. The west front slope with a factor of safety of 1.3 as shown on Figure 8 represents typical conditions near the centre of the highway, where the silty clay has the most strength gain under full depth of the embankment fill. The west front slope near the south side of the embankment (i.e. at the southwest corner of the approach) may have a slightly lower factor of safety due to three-dimensional effects and slightly lower undrained shear strengths of the foundation soils in this area. In this regard, the use of a combination of ultra-lightweight and EPS fill for the grade raise (as discussed in Section 5.9) would mitigate the slightly lower factor of safety in this localized area.

5.9.2 Liquefaction Potential and Deformations under Seismic Events

Screening level assessment of the liquefaction potential of the foundation soils indicates that the firm to very stiff silty clay strata is not liquefiable. Therefore, further liquefaction analyses are not required.

For seismic loading conditions, the estimated permanent deformations of the embankments under the design earthquake event were assessed using the procedure developed by Makdisi and Seed (1978). In this procedure, pseudostatic analyses were performed to compute the yield acceleration that is defined as the maximum seismic coefficient resulting in a factor of safety of unity against slope failure, and the consequent embankment deformations were estimated from the ratio of the yield acceleration to the maximum design acceleration for the site.

Pseudo-static analyses were performed for the west approach embankment (assuming the incorporation of the settlement/stability mitigation measures discussed in Section 5.9) and the proposed east approach embankment. The yield acceleration (k_y), which results in a factor of safety of unity against side slope failure, was calculated to be in the order of 0.07g. The estimated maximum horizontal acceleration induced by the earthquake for this site is 0.3g (as noted in Section 5.7). As such, based on Makdisi and Seed (1978) the estimated permanent displacement for the sliding of the embankment soil mass (for embankments ranging in height from about 9 m to 14 m) would be in the range of 20 cm to 35 cm for an earthquake event with magnitude 7.5. It is recommended that these modest estimated deformations planned to be managed by maintenance after the seismic event.

5.9.3 Settlement

Settlement analyses were performed on the critical sections of the proposed approach embankment widening and grade raise. For this report, critical sections are assumed to correspond to the areas where filling is greatest (i.e. at about Station 12+646 and Station 12+722

for the west and east approach, respectively). The settlement analyses were carried out using the commercially available program Unisettle (Version 3.2) produced by Unisoft Limited.

The silty clay deposit which underlies the site is relatively compressible and is normally consolidated at depth (i.e. it has not experienced loads in the past any greater than the current loads). Settlement analyses were carried out using the results of the borehole information, in situ field test data (field vane, SPT and CPT) and laboratory tests to estimate the deformation parameters of the subsoils.

Laboratory consolidation (oedometer) tests were not performed on samples obtained from the boreholes advanced directly within limits of the OCR overpass structure. However, a total of five (5) consolidation tests were performed on selected samples from immediately adjacent boreholes advanced as part of the investigation for the “Tall Rock Fill” embankment widening and grade raise immediately adjacent to the bridge area. The results from these tests have been used to refine the estimated deformation parameters for this analysis. For the details of the consolidation test results, reference should be made to Golder Report 021-1155-12, titled, “Tall Rock Fill Embankment Widening, Highway 417”. The results of these consolidation tests were also used to calibrate the estimates of the preconsolidation pressure profiles from the CPT results.

The over-consolidation ratio (OCR) profile required in the settlement analyses was established using correlations with the results of the consolidation tests, in situ vane tests and CPT tests. The following correlation proposed by Demer and Leroueil (2002) was employed to estimate preconsolidation pressure using net tip resistance, $q_t - \sigma_{vo}$, from the CPT:

$$\sigma_p' = \frac{q_t - \sigma_{vo}}{3.4} \quad \text{where}$$

σ_p' is the preconsolidation pressure (kPa)
 q_t is the corrected tip resistance (kPa)
 σ_{vo}' is the in-situ vertical effective stress (kPa)

In addition to the above, the following correlation proposed by Mesri (1975) was employed to estimate preconsolidation pressure based on the average mobilised undrained shear strengths assessed from the field vane tests:

$$s_u = 0.22 \sigma_p' \quad \text{where}$$

s_u is the average mobilised undrained shear strength (kPa)
 σ_p' is the preconsolidation pressure (kPa)

The calculated profile of preconsolidation pressure below natural ground surface is shown on Figure 2. The CPT results indicate a preconsolidation pressure of about 1100 kPa decreasing linearly to about 200 kPa to 350 kPa for the weathered silty clay; below this, the preconsolidation pressure is constant with depth between 200 kPa to 350 kPa for the unweathered silty clay.

The piezometric conditions used in the analyses were based on the groundwater levels noted during drilling and measured in the standpipe installations. In general, the groundwater level was assumed to be located at about the elevation of the natural ground surface in the analyses. It should be noted that all settlement analyses assume that all of the surface and near-surface organic layers have been removed prior to construction.

The following tables summarise the simplified stratigraphy along with the unit weight and deformation parameters employed in the settlement analyses for each of the abutment areas. The maximum estimated settlement of the foundation soils in these areas is presented, and a discussion on the rate of settlement is provided.

East Abutment

Soil Deposit	Thickness (m)	Unit Weight (kN/m ³)	OCR*	e _o *	C _r *	C _c *
Weathered Silty Clay Crust	1.0	19	4.2 / 4.1	0.6 / 0.8	0.010 / 0.014	0.40 / 0.49
	3.0	19	4.1 / 1.2	0.8 / 1.4	0.014 / 0.025	0.49 / 0.75
Unweathered Silty Clay	8.6	19	1.2 / 1.0	1.4 / 1.2	0.025 / 0.025	0.75 / 0.75

*Values shown are at the top/bottom of the deposit, with values between interpolated linearly.

West Abutment

Soil Deposit	Thickness (m)	Unit Weight (kN/m ³)	OCR*	e _o *	C _r *	C _c *
Weathered Silty Clay Crust	1.0	19	7.0 / 6.5	0.6 / 0.8	0.010 / 0.014	0.40 / 0.49
	3.0	19	6.5 / 1.4	0.8 / 1.4	0.014 / 0.025	0.49 / 0.75
Unweathered Silty Clay	7.8	19	1.4 / 1.0	1.4 / 1.3	0.025 / 0.025	0.75 / 0.75
	5.6	19	1.0 / 1.0	1.3 / 1.2	0.025 / 0.025	0.75 / 0.75

*Values shown are at the top/bottom of the deposit, with values between interpolated linearly.

A coefficient of consolidation (c_v) of 2.1×10^{-3} (cm²/s) has been used to estimate the rate of consolidation of the silty clay strata beneath the embankments, based on the average of the values from the laboratory consolidation tests (for the appropriate stress range considering the proposed

grade raises) and based on the empirical correlation found in US Navy (1982) for normally consolidated clays using the average value of the liquid limits from the laboratory index testing.

Based on the results of the settlement analyses, the maximum primary settlement of the foundation soils and the estimated time for 90 percent of the settlement to occur, assuming two-way drainage, is summarised in the table below. The settlement of the cohesionless soils (i.e. earth fills, sandy silt and silty sand till layers) are expected to be minimal and occur immediately after filling has completed and as such are not included in the primary settlement results.

Location	Thickness of Silty Clay Deposit	Primary Settlement (10 m LT of Centreline)	Primary Settlement (at Centreline)	Estimated Time to 90 percent Settlement
East Abutment	12.6 m	135 mm	115 mm	5 years
West Abutment	17.4 m	155 mm	135 mm	10 years

The estimated time for 90 percent of the settlement to occur is relatively long for the modest amount of filling due to the thickness of the silty clay deposits and the low coefficient of consolidation.

The magnitude of the secondary (creep) settlement has also been analysed, using the correlation for estimating $C_{\alpha(\epsilon)}$ proposed by Mesri (1973) based on natural water content. A value of 3.75×10^{-3} for $C_{\alpha(\epsilon)}$ was used in the analysis. The results of the secondary settlement analyses are summarised below. Secondary settlement is expected to occur once 90 percent of the primary settlement has been completed.

Location	Thickness of Silty Clay Deposit	Secondary Settlement per log cycle	Time for Secondary Compression to Occur
East Abutment	12.6 m	35 mm	5 to 50 years
West Abutment	17.4 m	50 mm	10 to 100 years

5.10 Mitigation of Stability Issues / Time Dependent Settlements

As discussed in Section 5.8, the presence of the extensive silty clay deposit at the site will result in some time dependent settlements along the approach embankments due to the required grade raises and widenings. Since the bridge structure is to be supported on deep foundations and can therefore be considered a fixed point, the settlements will be most severe (i.e. all differential) in the immediate vicinity of the bridge. Under the existing fill at the west and east abutments there is evidence of rotation of the approach slabs and settlements beyond the slabs. In addition, the presence of the 5 m high natural slope along the south side of the west embankment toe results in a factor of safety of less than 1.3 for the global stability of the proposed grade raise on this approach.

As such, consideration needs to be given to adopting a design to achieve a minimum factor of safety of 1.3 at the west approach and to limit the post-construction settlements (in accordance with the criteria described in the following section) and subsequent maintenance of the new roadway pavement structure at both approaches to the bridge. The following sections outline the options and recommendations for achieving the target factor of safety for the required embankment geometry and for minimizing the time dependent, post-construction settlements that could effect the performance of the roadway. The advantages, disadvantages, relative costs and risks/consequences for the mitigation options at the east and west approach areas are summarized and ranked in Table 3 and 4, respectively. At this site, a combination of ultra-lightweight (slag) and EPS fill is considered to be the most practical and preferred settlement and stability mitigation option.

5.10.1 Post-Construction Settlement Criteria

The following settlement criteria (as recommended by MTO Eastern Geotech) was used as a guide for assessing the mitigation requirements for the design of the embankments.

Distance Behind Abutment	Allowable Settlement
0 m to 30 m	10 mm to 25 mm
30 m to 70 m	25 mm to 50 mm
70 m to 170 m	50 mm to 100 mm
Greater than 170 m	100 mm to 200 mm

5.10.2 Ultra-Lightweight (Slag) and EPS Fill

In order to reduce the loads imposed by the up to 1.6 m grade raises for the approach embankments (plus the required filling on existing side and front slopes), the use of a combination of ultra-lightweight (slag) fill (with a unit weight not greater than 12 kN/m^3) and expanded polystyrene (EPS) fill should be considered. The use of this material for the embankment grade raises and widenings would eliminate the need for a stabilizing toe berm on the south side of the west approach and would reduce the time-dependent, post-construction (consolidation) settlement of the foundation soils below both approaches.

In order to reduce the post-construction settlements to less than 25 mm at the approaches, it is estimated a zero net loading would be required at the east approach (i.e. thickness of EPS = required grade raise) and that a net unloading would be required at the west approach (i.e. thickness of EPS > required grade raise). Since the EPS must be covered by a concrete slab and at least 1 m of conventional earth fill, it will be necessary to sub-excavate a minimum of 1 m below the existing grade prior to installation of the EPS. In addition, to provide the necessary cover on the sides of the EPS and for filling on the existing side slopes and front slopes where EPS cannot be practically employed, it will be necessary to use ultra-lightweight slag fill. The details of the extent of the required EPS (within the approach areas) and the estimated post-construction settlements are provided in the following table. Table 5 and 6 following the text of this report detail the fill types and fill thickness as required along the east and west embankments, respectively (i.e. within and beyond the approaches) to satisfy the post-construction settlement criteria for this project.

East Approach

Station		Thickness of EPS Required		Estimated Settlement	
From	To	LT Side (Median)	Centreline and RT Side	LT Side (Median)	Centreline and RT Side
12+709 (Abutment)	12+719	2.5 m	1.5 m	15 mm	15 mm
12+719	12+729	2 m	1 m	20 mm	20 mm

West Approach

Station		Thickness of EPS Required		Estimated Settlement	
From	To	LT Side (Median)	Centreline and RT Side	LT Side (Median)	Centreline and RT Side
12+664 (Abutment)	12+654	3 m	2 m	20 mm	10 mm
12+654	12+644	2 m	2 m	10 mm	<5 mm

As an alternative to the use of a combination of EPS and slag fill, consideration could be given to employing only slag fill (i.e. no EPS) to reduce the loading from the grade raises. However, given that the ratio of the bulk unit weight of the conventional earth fill (20 kN/m^3) to ultra-lightweight slag fill (12 kN/m^3) is about 1.75, thicknesses of slag fill on the order of about 1.8 m to 5.2 m would be required to fully replace the thicknesses of EPS. Since the operation of the existing roadway is to be maintained during construction, excavations on the order of 4 m to 5 m would require temporary shoring/roadway protection to allow the installation of the slag. The additional costs associated with this option will make this alternative impractical.

It should be noted that utilizing the combination of EPS and slag fill at these locations will reduce the new loading and driving forces at the top of the west embankment sufficiently that a toe berm will no longer be required for stability (see Figure 9). It should also be noted that post-construction secondary consolidation (i.e. creep) settlements on the order of about 10 mm to 40 mm per log-cycle of time should still be expected with this option.

5.10.3 Preloading and Toe Berm

If the proposed grade raises and widenings are constructed with conventional earth fill, primary consolidation settlements on the order of about 155 mm (left side) to 115 mm (centerline to right side) are expected to occur. We understand that the current construction schedule may be able to facilitate a preload period ranging from about 6 months to 12 months in length. However, given the thickness of the silty clay strata at this site (and the corresponding long drainage path for the dissipation of excess pore pressures) it is estimated that only a relatively small portion of the primary consolidation would be completed within these time frames. As such, post-construction settlements greater than the settlement criteria for the approach area would still occur. The following outlines the estimated percentage of primary consolidation that would be completed within various preload times and the estimated remaining primary settlement after the preload period.

Preload Period (months)	Estimated Primary Consolidation Complete	Estimated Remaining Primary Settlement* (mm)
6	25% to 30%	80 to 110
9	30% to 40%	70 to 100
12	35% to 45%	60 to 95

*Note: behind the abutment, within the approach embankment area

In addition, if preload were to be adopted and if conventional earth fill is utilized, it will be necessary to construct a stabilizing berm approximately 14 m wide by 5 m high by 90 m long (i.e. between about Station 12+664 and Station 12+576) along the south toe of the west approach to achieve a factor of safety of 1.3 for global embankment stability (see Figure 10).

It should be noted that post-construction settlements remaining after the preload period in conjunction with the long-term settlements due to secondary consolidation (i.e. creep) will require roadway maintenance operations to maintain performance with this option.

5.10.4 Surcharging and Toe Berms

As noted above, if conventional earth fill is used for the construction of the approach embankment grade raises and widenings, a toe berm is required to achieve adequate stability of the south slope of the west approach. If a surcharge was to be employed to accelerate consolidation settlements, an even larger toe berm would be required in this area and a toe berm would also be required at the toe of the front slope of the west approach to maintain stability which would block the OCR tracks at the base of the existing fills. In addition, a toe berm would also have to be provided on the south slope of the east approach to maintain embankment stability under the surcharge loading. Given the potential for conflict with the existing OCR tracks and the large amount of temporary fill that would have to be placed and then removed to maintain global embankment stability, surcharging is not considered a practical settlement mitigation measure for this site.

However, a partial surcharge (without the need for a toe berm) could be considered to be placed on top of the new fills to be constructed in the median area only. Left in place for a 6 month period, this option would help reduce the potential for transverse differential settlements between the thicker fills required on the widened portion of Highway 417 (in the median) and the thinner fills to be placed on top of the existing Highway 417 roadway. Stability analysis indicates that a surcharge of up to 2 m high could be placed on top of the fill required in the median area without reducing the factor of safety of the existing embankments. The surcharge fill would have to be limited to the median area (i.e. from 3 m LT to 17 m LT on the east embankment and from 4 m LT to 14 m LT on the west embankment). In addition, to minimize the potential of the surcharge fill causing settlement and downdrag loads on the existing abutment pile foundations, it is recommended that the surcharge fill not be placed within a zone extending 10 m back from the east abutment (i.e. only east of Station 12+719) and 20 m back from the west abutment (i.e. only west of Station 12+644).

5.10.5 Wick Drains

Due to the presence of the thick fills at the existing approaches (i.e. up to 12 m thick) and considering the potential for the presence of cobbles and boulders within the fills near the bridge abutment areas and rock fill away from the abutment areas, the installation of wick drains through the embankments and into the underlying silty clay strata could be problematic at this site. As such, wick drains are not recommended as a mitigation option for this site.

5.10.6 Full Sub-excavation

Given the thickness of the existing approach embankment fills (up to 12 m thick), the depth of the silty clay stratum (up to 16 m deep) and the requirement to maintain the flow of the existing traffic on the roadway during the new construction, full sub-excavation of the compressible clayey strata is not considered a feasible mitigation option for this site.

5.11 Subgrade Preparation and Embankment Construction

Based on the borehole results, the embankment subgrade soils will consist of the existing embankment fill materials overlying a thick deposit of sensitive marine clay. Any topsoil, organic matter and softened / loosened soils should be stripped from below the existing median areas and front and side slopes, within the limits of the widenings/grade raises. All subgrade soils should be proof-rolled prior to fill placement.

In terms of constructability, ultra-lightweight slag fill will require careful placement and controlled light compaction in order to maintain its composition and desired low bulk unit weight (i.e. 12 kN/m³). The compaction should be carried out by a light roller and by a limited number of passes (2 to 4). Inspection and field density testing should be carried out by qualified personnel during placement operations to ensure that the appropriate material is used and that only the required levels of compaction is achieved. To reduce surface water erosion, following placement, the ultra-lightweight slag on the side slopes and front slopes should be covered by a minimum of 150 mm of earth fill plus 50 mm of topsoil, then seed and mulch or pegged sod.

In order to minimise differential settlement between the widened portions of Highway 417 and the existing embankments, the use of a combination of expanded polystyrene (EPS) fill and ultra-lightweight slag fill is recommended for the widening and grade raise. The majority of settlement of the slag fill itself will occur during construction. In addition, keying of the new embankment fill into the existing side slopes and front slopes along the eastbound lanes of Highway 417 would help to reduce the effects of differential settlement. The new embankment fills should be benched into the existing embankment in accordance with OPSD 208.010.

NSSPs should be included in the Contract Documents to address the requirements for the supply and installation of Expanded Polystyrene (EPS) and Ultra-Lightweight slag fill materials (see examples for each in Appendix B).

5.12 Design and Construction Considerations

5.12.1 Excavations

The excavations for installing the EPS and for construction of the abutments will extend through the existing embankment fills. For the pier construction, the excavations will likely extend through surficial fills and into the weathered silty clay crust. Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities. These soils are classified as Type 3 soils according to the OHSA and therefore excavations for the east abutment, west abutment and the west pier should be made with side slope no steeper than 1 horizontal to 1 vertical. However, due to the proximity of the railway tracks temporary shoring will be required as discussed in Section 5.11.2.

5.12.2 Temporary Shoring

Due to the proximity of the rail tracks, there is insufficient distance to satisfy the 1H:1V side slopes requirement for temporary excavations, and therefore a support system will be required to minimise the movement of the railroad track and to maintain the stability of the existing highway embankment. The support system would be located at the edge of the railroad track ballast and at the toe of the existing highway embankment. In addition, it is expected that temporary shoring will be necessary for the construction of the abutments and may be necessary for the installation of the EPS and/or slag fill in order to maintain traffic on the existing roadway.

The temporary excavation support systems required for work adjacent to the railroad track shall be designed in accordance with AREMA, as specified in the Special Provision 105S19M (dated November 2006 – see Appendix B) and designed to Performance Level 1. Monitoring of the performance of shoring/support system during the work shall be carried out to the satisfaction of the railway. All roadway protection should also be in accordance with Special Provision 105S19M, to Performance Level 2.

5.12.3 Groundwater and Surface Water Control

Given that the abutment pile cap elevations are above the measured groundwater level, it is anticipated that the excavations at the east and west abutments will not experience significant groundwater inflows. However, a small amount of groundwater flow is expected for the east and

west pier excavations, and it is anticipated that adequate groundwater control can be achieved by pumping from properly filtered sumps in the excavation.

Surficial drainage may be also required around the perimeter of the west pier excavation due to the undercutting of the existing drainage ditch.

5.12.4 Obstructions

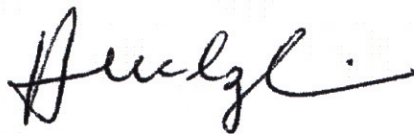
Although not encountered at all of the locations during the field investigation, it is expected that cobbles and boulders may be present at the abutment areas, and pre-augering through the earth fill could be required prior to installation of piles.

A thin layer of basal silty sand till was encountered beneath the silty clay deposit at this site and this material may contain cobbles and boulders. However, given the thinness of this layer, it is not expected that this will affect the seating of piles and caissons. Nevertheless, provision should be made in the Contract Documents to ensure that the Contractor is equipped to handle such obstructions

5.13 CLOSURE

This Foundation Design Report was prepared by Mr. Chris Ng, P.Eng. and Dr. Allen Li, P.Eng. and reviewed by Dr. J. Paul Dittrich, Ph.D., P.Eng., an Associate with Golder. Mr. Fintan Heffernan, P.Eng., Golder's Designated MTO Contact for this project, conducted an independent quality review of the report.

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ALL/CN/JPD/FJH/cn/all/jpd/sm

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LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. General

π	3.1416
$\ln x$,	natural logarithm of x
\log_{10}	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ϵ	linear strain
ϵ_v	volumetric strain
η	coefficient of viscosity
ν	poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a) Index Properties (continued)

w	water content
w_l	liquid limit
w_p	plastic limit
I_p	plasticity index $= (w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p) / I_p$
I_C	consistency index $= (w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction $= \tan \delta$
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 + \sigma_3)$
S_t	sensitivity

- Notes:**
- 1 $\tau = c' + \sigma' \tan \phi'$
 - 2 shear strength $= (\text{compressive strength})/2$
 - * density symbol is ρ . Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density x acceleration due to gravity)

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
SS	Split-spoon
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Consistency

	c_u, s_u	kPa	psf
Very soft		0 to 12	0 to 250
Soft		12 to 25	250 to 500
Firm		25 to 50	500 to 1,000
Stiff		50 to 100	1,000 to 2,000
Very stiff		100 to 200	2,000 to 4,000
Hard		over 200	over 4,000

Dynamic Cone Penetration Resistance; N_d :

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure

PM: Sampler advanced by manual pressure

WH: Sampler advanced by static weight of hammer

WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm² pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

Note: 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering.

Faintly weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable.

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	> 2 m
Thickly bedded	0.6 m to 2m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	< 6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	> 3 m
Wide	1 - 3 m
Moderately close	0.3 - 1 m
Close	50 - 300 mm
Very close	< 50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	> 60 mm
Coarse Grained	2 - 60 mm
Medium Grained	60 microns - 2 mm
Fine Grained	2 - 60 microns
Very Fine Grained	< 2 microns

Note: * Grains > 60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varies from 0% for completely broken core to 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to (W.R.T.) Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes

An abbreviated description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

B - Bedding	P - Polished
FO - Foliation/Schistosity	S - Slickensided
CL - Cleavage	SM - Smooth
SH - Shear Plane/Zone	R - Ridged/Rough
VN - Vein	ST - Stepped
F - Fault	PL - Planar
CO - Contact	FL - Flexured
J - Joint	UE - Uneven
FR - Fracture	W - Wavy
MF - Mechanical Fracture	C - Curved
- Parallel To	
⊥ - Perpendicular To	

RECORD OF BOREHOLE SHEETS

MIS-MTO 001 0211155-5100-MTO.GPJ GAL-MISS.GDT 18/10/06

Continued Next Page

+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

MIS-MTO 001 0211155-5100-MTO.GPJ GAL-MISS.GDT 18/10/06

+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

PROJECT <u>021-1155-5100</u>		RECORD OF BOREHOLE No 02-402				3 OF 3 METRIC											
W.P. <u>4254-05-01</u>		LOCATION <u>N 5021574.1 ; E 355243.2</u>				ORIGINATED BY <u>PAH</u>											
DIST <u>9</u> HWY <u>417</u>		BOREHOLE TYPE <u>CME 55 Bombardier</u>				COMPILED BY <u>CN</u>											
DATUM <u>Geodetic</u>		DATE <u>November 27 to 29, 2002</u>				CHECKED BY <u>JPD</u>											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100					WATER CONTENT (%) 25 50 75					
71.0	Silty Clay to Clay, occasional silty sand and clayey silt seams Firm to stiff Grey brown Moist to wet	[Hatched Pattern]	13	SS	WR	[Groundwater Symbol]	73										
72							72	×	+								
71							71	×									
70							70										
69							69										
68							68										
23.2	Inferred Silty Sand and Gravel (TILL) Sandstone (BEDROCK) Fresh Thinly to medium bedded Fine to medium grained Light grey Bedrock cored between 23.2 m and 26.1 m depth For bedrock coring details refer to Record of Drillhole 02-402	[Hatched Pattern]															
67.9																	
26.1	End of Borehole Note: Water level in piezometer at 9.7 m depth on January 8, 2003																

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Marathon Drilling

CHECKED: _____

PROJECT <u>021-1155-5100</u>		RECORD OF BOREHOLE No 02-403		1 OF 2 METRIC	
W.P. <u>4254-05-01</u>		LOCATION <u>N 5021592.4 ; E 355264.0</u>		ORIGINATED BY <u>DB / PAH</u>	
DIST <u>9</u> HWY <u>417</u>		BOREHOLE TYPE <u>CME 55 Bombardier</u>		COMPILED BY <u>CN</u>	
DATUM <u>Geodetic</u>		DATE <u>November 25, 2002</u>		CHECKED BY <u>JPD</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE LIQUID LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
87.0	Ground Surface																
0.0	Sand and gravel (FILL) Compact Brown Moist		1	SS	19												
85.5																	
1.5	Clayey Silt and Sandy Silt, trace to some clay Firm to very stiff Grey brown Moist to wet		2	SS	19												
			3	SS	10												
			4	SS	4												
			5	SS	4												
82.6																	
4.4	Silty Clay Firm to stiff Grey brown Wet		6	SS	5												
			7	SS	4												
			8	SS	2												
79.4																	
7.6	Silty Clay, containing silt seams Firm to stiff Grey Wet		9	SS	WH												
			10	SS	WH												

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

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

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Marathon Drilling

[illegible]

CHECKED: _____



MIS-MTO 001 0211155-5100-MTO.GPJ GAL-MISS.GDT 18/10/06

Continued Next Page

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

PROJECT <u>021-1155-5100</u>		RECORD OF BOREHOLE No 02-404		2 OF 3 METRIC	
W.P. <u>4254-05-01</u>		LOCATION <u>N 5021601.8 ; E 355290.1</u>		ORIGINATED BY <u>DB</u>	
DIST <u>9</u> HWY <u>417</u>		BOREHOLE TYPE <u>CME 55 Bombardier</u>		COMPILED BY <u>CN</u>	
DATUM <u>Geodetic</u>		DATE <u>November 26, 2002</u>		CHECKED BY <u>JPD</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
	--- CONTINUED FROM PREVIOUS PAGE ---																
	Clayey Silt to Clay Firm to stiff Grey brown Moist		7	SS	13												
			8	SS	6												
			9	SS	8												
			10	SS	6												
79.6 13.0	Silty Clay, containing silt seams Firm to stiff Grey-brown to Grey Wet		11	SS	2												
			12	SS	2												
			13	SS	WH												
			14	SS	PM												
			15	SS	WR												

Continued Next Page

+³, X³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT		021-1155-5100		RECORD OF BOREHOLE No 02-404		3 OF 3		METRIC									
W.P.		4254-05-01		LOCATION		N 5021601.8 ; E 355290.1		ORIGINATED BY DB									
DIST		9 HWY 417		BOREHOLE TYPE		CME 55 Bombardier		COMPILED BY CN									
DATUM		Geodetic		DATE		November 26, 2002		CHECKED BY JPD									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
70.9	Silty Clay, containing silt seams Firm to stiff Grey-brown to Grey Wet		16	SS	WR												
21.6	Silty Sand and Gravel (TILL) Compact Grey Wet																
70.3			17	SS	10/18												
22.3	Sandstone (BEDROCK) Fresh to slightly weathered Very thinly to medium bedded Fine to medium grained Light grey Bedrock cored between 22.3 m and 26.0 m depth For bedrock coring details refer to Record of Drillhole 02-404																
66.6																	
26.0	End of Borehole																
	Note: Water level in piezometer at 10.5 m depth on January 8, 2003																

MIS-MTO 001 0211155-5100-MTO.GPJ GAL-MISS.GDT 18/10/06

SHEET 1 OF 1

DATUM: Geodetic

DRILLING CONTRACTOR: Marathon Drilling

CHECKED: _____

PROJECT 021-1155-6000			RECORD OF BOREHOLE No 06-1			1 OF 3 METRIC		
W.P. 4254-05-01			LOCATION N 5021585.6; E 355310.7			ORIGINATED BY P.A.H.		
DIST 9 HWY 417			BOREHOLE TYPE CME 55 Bombardier			COMPILED BY J.M.		
DATUM Geodetic			DATE Aug. 18, 2006			CHECKED BY F.J.H.		
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100
92.7	ROAD SURFACE							PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%)
0.0	ASPHALTIC CONCRETE							
92.5								
0.2	Sand and Gravel, trace to some silt, trace clay (FILL) Compact Grey Moist		1	A.S.			92	
			2	SS	12		91	○
90.4							90	
2.3	Sand, some gravel with cobbles and boulders (FILL) Compact to dense Brown Moist		3	SS	22			
			4	SS	38		88	
							87	
87.2								
5.5	Sand, trace to some silt, trace gravel (FILL) Compact to dense Brown Moist		5	SS	49		86	
			6	SS	18		85	○
			7	SS	29		84	
							83	

Continued Next Page




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

MIS-MTO 001 021-1155-6000.GPJ GAL-MISS.GDT 18/10/06

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Continued Next Page

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

PROJECT 021-1155-6000		RECORD OF BOREHOLE No 06-1				3 OF 3 METRIC											
W.P. 4254-05-01		LOCATION N 5021585.6; E 355310.7				ORIGINATED BY P.A.H.											
DIST 9 HWY 417		BOREHOLE TYPE CME 55 Bombardier				COMPILED BY J.M.											
DATUM Geodetic		DATE Aug. 18, 2006				CHECKED BY F.J.H.											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
71.4	Silty Clay to Clay, some thin silty fine sand seams at depth Firm Grey Wet		14	SS	WR												
21.3	Sandy Silt, some gravel (TILL) Very loose Grey Wet		15	SS	2												
70.0	Sandstone (BEDROCK) Slightly weathered to fresh with depth Very thinly to medium bedded Fine to medium grained Light grey Bedrock cored between 22.7m and 25.7m depth. For bedrock coring details refer to Record of Drillhole 06-1.																
22.7																	
67.0	End of Borehole																
25.7																	

MIS-MTO 001 021-1155-6000.GPJ GAL-MISS.GDT 18/10/06

PROJECT: 021-1155-6000

RECORD OF DRILLHOLE: 06-1

SHEET 1 OF 1

LOCATION: N 5021585.6; E 355310.7

DRILLING DATE: Aug. 18, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	PENETRATION RATE RUN No. (m/min)	FLUSH	COLLOUR % RETURN	JN - Joint		BD - Bedding		PL - Planar		PO - Polished		BR - Broken Rock	NOTES
								FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided	UN - Undulating	ST - Stepped	Ro - Rough	MB - Mechanical Break		
								SHR - Shear	CO - Contact	OR - Orthogonal	CL - Cleavage	IR - Irregular					
		ROCK SURFACE		70.00													
23	Rotary Drill NQ Core	Sandstone (BEDROCK) Slightly weathered to fresh with depth Very thinly to medium bedded Fine to medium grained Light grey		22.70													
24																	
25																	
26		End of Drillhole		67.00													
27				25.70													
28																	
29																	
30																	
31																	
32																	

DEPTH SCALE

1 : 50



LOGGED: P.A.H.

CHECKED: F.J.H.

MIS-RCK 002 021-1155-6000-ROCK.GPJ GAL-MISS.GDT 24/11/06 JM

PROJECT <u>021-1155-6000</u>		RECORD OF BOREHOLE No 06-2		1 OF 2 METRIC	
W.P. <u>4254-05-01</u>		LOCATION <u>N 5021571.9; E 355292.4</u>		ORIGINATED BY <u>P.A.H.</u>	
DIST <u>9</u> HWY <u>417</u>		BOREHOLE TYPE <u>CME 55 Bombardier</u>		COMPILED BY <u>J.M.</u>	
DATUM <u>Geodetic</u>		DATE <u>Aug. 21, 2006</u>		CHECKED BY <u>F.J.H.</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa				WATER CONTENT (%)				
86.5	GROUND SURFACE						20	40	60	80	100	W _p	W	W _L		
86.3	Sand and Gravel (FILL)															
0.2	Grey															
	Sand, trace silt (FILL)															
	Brown															
85.0																
1.5	Sand, trace gravel and silty clay (FILL)		1	SS	9											
	Very loose to loose															
	Red brown to grey brown															
	Moist to wet															
			2	SS	3											
			3	SS	3											
82.2																
4.3	Silty Clay															
	Very stiff to stiff with depth		4	SS	11											
	Grey brown															
	Wet															
			5	SS	2											
79.5																
7.0	Silty Clay to Clay with fine sand and silt layers at depth															
	Firm		6	SS	WH											
	Grey															
	Wet															

Continued Next Page

+³, X³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

MIS-MTO 001 021-1155-6000.GPJ GAL-MISS.GDT 18/10/06

PROJECT <u>021-1155-6000</u>			RECORD OF BOREHOLE No 06-2			2 OF 2 METRIC												
W.P. <u>4254-05-01</u>			LOCATION <u>N 5021571.9; E 355292.4</u>			ORIGINATED BY <u>P.A.H.</u>												
DIST <u>9</u> HWY <u>417</u>			BOREHOLE TYPE <u>CME 55 Bombardier</u>			COMPILED BY <u>J.M.</u>												
DATUM <u>Geodetic</u>			DATE <u>Aug. 21, 2006</u>			CHECKED BY <u>F.J.H.</u>												
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa										
--- CONTINUED FROM PREVIOUS PAGE ---							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					25 50 75 WATER CONTENT (%)						
71.3	Silty Clay to Clay with fine sand and silt layers at depth Firm Grey Wet		8	SS	WR													
			9	SS	WR													
			10	SS	WR													
71.3																		
15.2	Silty Clay to Clay with clayey silt layers Firm to soft Grey Wet		11	SS	WR													
70.7																		
15.9	Silty Sand, some gravel and cobbles (TILL) Loose to compact Wet																	
70.0																		
16.5	Sandstone (BEDROCK) Slightly weathered to fresh with depth Very thinly to medium bedded Fine to medium grained Light grey Bedrock cored between 16.5m and 19.5m depth. For bedrock coring details refer to Record of Drillhole 06-2.																	
67.0																		
19.5	End of Borehole																	

MIS-MTO 001 021-1155-6000.GPJ GAL-MISS.GDT 18/10/06

PROJECT: 021-1155-6000

RECORD OF DRILLHOLE: 06-2

SHEET 1 OF 1

LOCATION: N 5021571.9; E 355292.4


DRILLING DATE: Aug. 21, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (mm/min)	FLUSH	COLLOUR % RETURN	JN - Joint		BD - Bedding		PL - Planar		PO - Polished		BR - Broken Rock	NOTES: For additional abbreviations refer to list of abbreviations & symbols.	WATER LEVELS INSTRUMENTATION		
									FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided	SHR - Shear	CO - Contact	UN - Undulating	SM - Smooth				Ro - Rough	MB - Mechanical Break
									VN - Vein	OR - Orthogonal	ST - Stepped	IR - Irregular	IR - Irregular	IR - Irregular	IR - Irregular	IR - Irregular				IR - Irregular	IR - Irregular
		ROCK SURFACE		70.00																	
17	Rotary Drill NQ Core	Sandstone (BEDROCK) Slightly weathered to fresh with depth Very thinly to medium bedded Fine to medium grained Light grey		16.50																	
18																					
19																					
20		End of Drillhole		67.00																	
21				19.50																	
22																					
23																					
24																					
25																					
26																					

DEPTH SCALE

1 : 50



LOGGED: P.A.H.

CHECKED: F.J.H.

MIS-RCK 002 021-1155-6000-ROCK.GPJ GAL-MISS.GDT 24/11/06 JM

PROJECT <u>021-1155-5100</u>		RECORD OF BOREHOLE No CPT02-401				1 OF 2 METRIC										
W.P. <u>4254-05-01</u>		LOCATION <u>N 5021560.1 ; E 355216.3</u>				ORIGINATED BY <u>PAH</u>										
DIST <u>9</u> HWY <u>417</u>		BOREHOLE TYPE <u>CME 55 Bombardier</u>				COMPILED BY <u>CN</u>										
DATUM <u>Geodetic</u>		DATE <u>December 02 and 06, 2002</u>				CHECKED BY <u>JPD</u>										
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
93.0	Ground Surface						20	40	60	80	100					
0.0	Sand and gravel, cobbles and boulders (ROCK FILL) Brown Moist															
88.9																
4.1	Silty Clay Stiff Grey brown Moist		1	SS	12											
88.2																
4.9	End of Borehole Cone Penetration Test from 4.88m depth For details refer to Cone Penetration Test CPT02-401															

Continued Next Page

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



+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

MIS-MTO 001 0211155-5100-MTO.GPJ GAL-MISS.GDT 18/10/06




MIS-MTO 001 0211155-5100-MTO.GPJ GAL-MISS.GDT 18/10/06

Continued Next Page

+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

PROJECT <u>021-1155-5100</u>		RECORD OF BOREHOLE No CPT02-405		2 OF 3 METRIC	
W.P. <u>4254-05-01</u>		LOCATION <u>N 5021613.4 ;E 355303.0</u>		ORIGINATED BY <u>DB</u>	
DIST <u>9</u> HWY <u>417</u>		BOREHOLE TYPE <u>CME 55 Bombardier</u>		COMPILED BY <u>CN</u>	
DATUM <u>Geodetic</u>		DATE <u>November 29, 2002</u>		CHECKED BY <u>JPD</u>	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL	
	— CONTINUED FROM PREVIOUS PAGE —							20	40	60	80	100	W _p	W	W _L						
80.9	Silty Clay Firm to stiff Brown																				
			1	SS	PH																
78.8																					
11.9	End of Borehole Cone Penetration Test from 11.89m depth For details refer to Cone Penetration Test CPT02-405																				

Continued Next Page

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



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

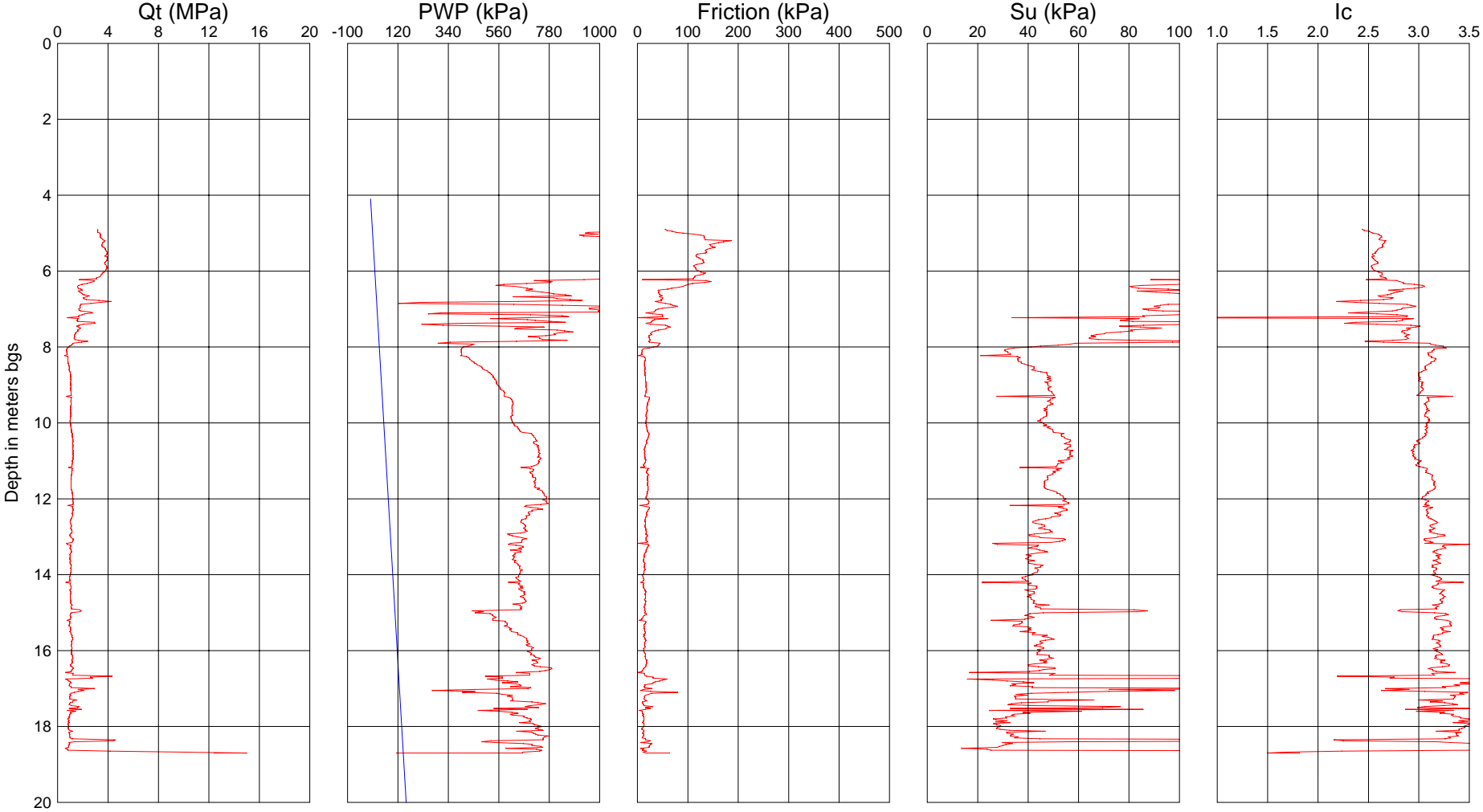
MIS-MTO 001 0211155-5100-MTO.GPJ GAL-MISS.GDT 18/10/06

Cone Penetration Test - CPT02-401

Test Date : December 06, 2002
Location : West Side of CN Rail Overpass

Operator : Golder Associates

Ground Surf. Elev. : 93.00
Water Table Depth : 4.10



Qt normalized for
unequal end area effects

$S_u = (Q_t - \sigma_v) / N_k$
 $N_k = 18$
 $\gamma = 19 \text{ kN/m}^3$

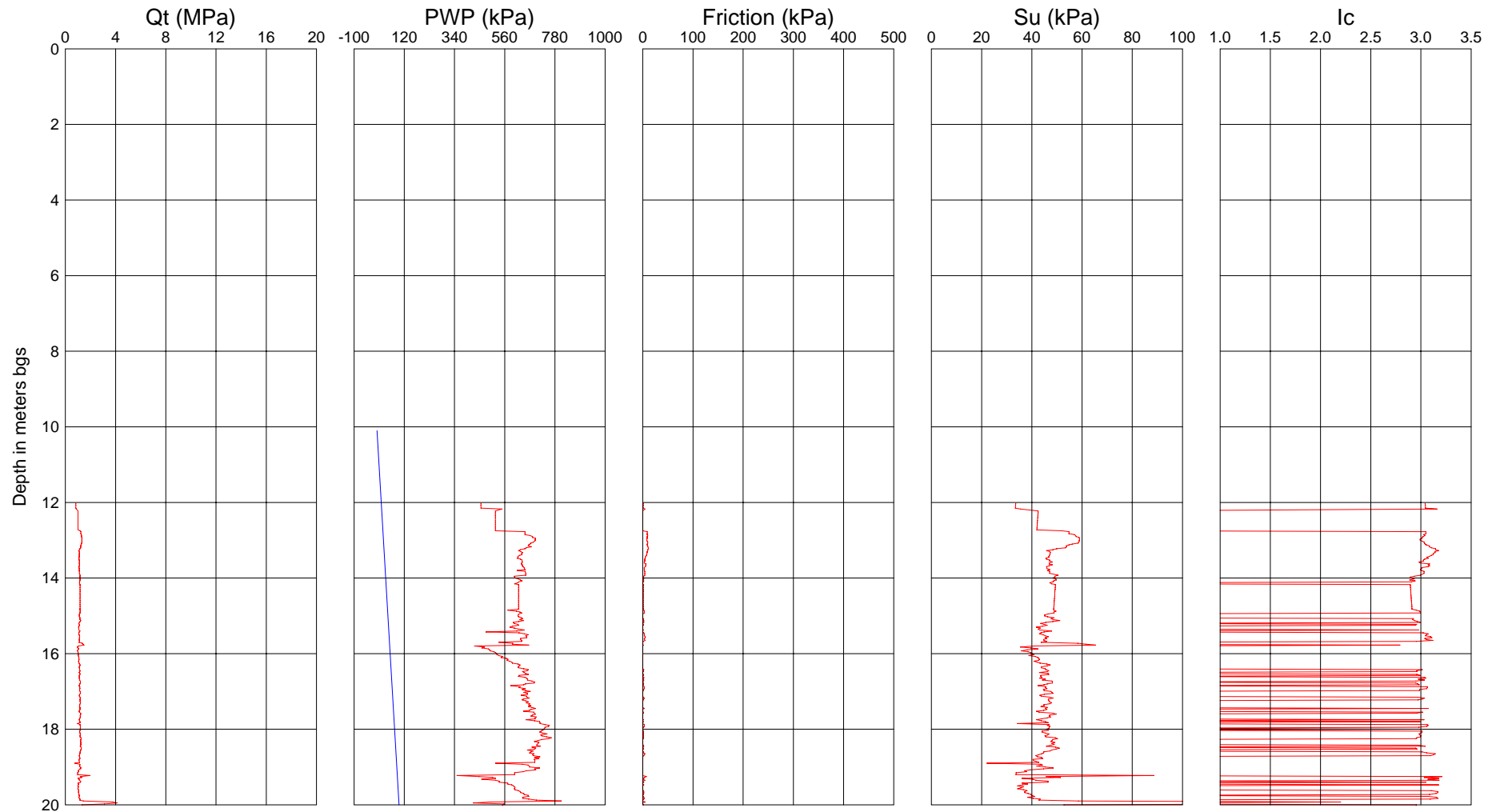
After Robertson and (Fear) Wride (1998)
 $I_c < 1.31$ - Gravelly sands
 $1.31 < I_c < 2.05$ - Clean to silty sand
 $2.05 < I_c < 2.60$ - Silty sand to sandy silt
 $2.60 < I_c < 2.95$ - Clayey silt to silty clay
 $2.95 < I_c < 3.60$ - Clays

Cone Penetration Test - CPT02-405

Test Date : November 29, 2002
Location : East Side of CN Rail Overpass

Operator : Golder Associates

Ground Surf. Elev. : 90.70
Water Table Depth : 10.10



Qt normalized for
unequal end area effects

$S_u = (Q_t - \sigma_{av}) / N_k$
 $N_k = 18$
 $\gamma = 19 \text{ kN/m}^3$

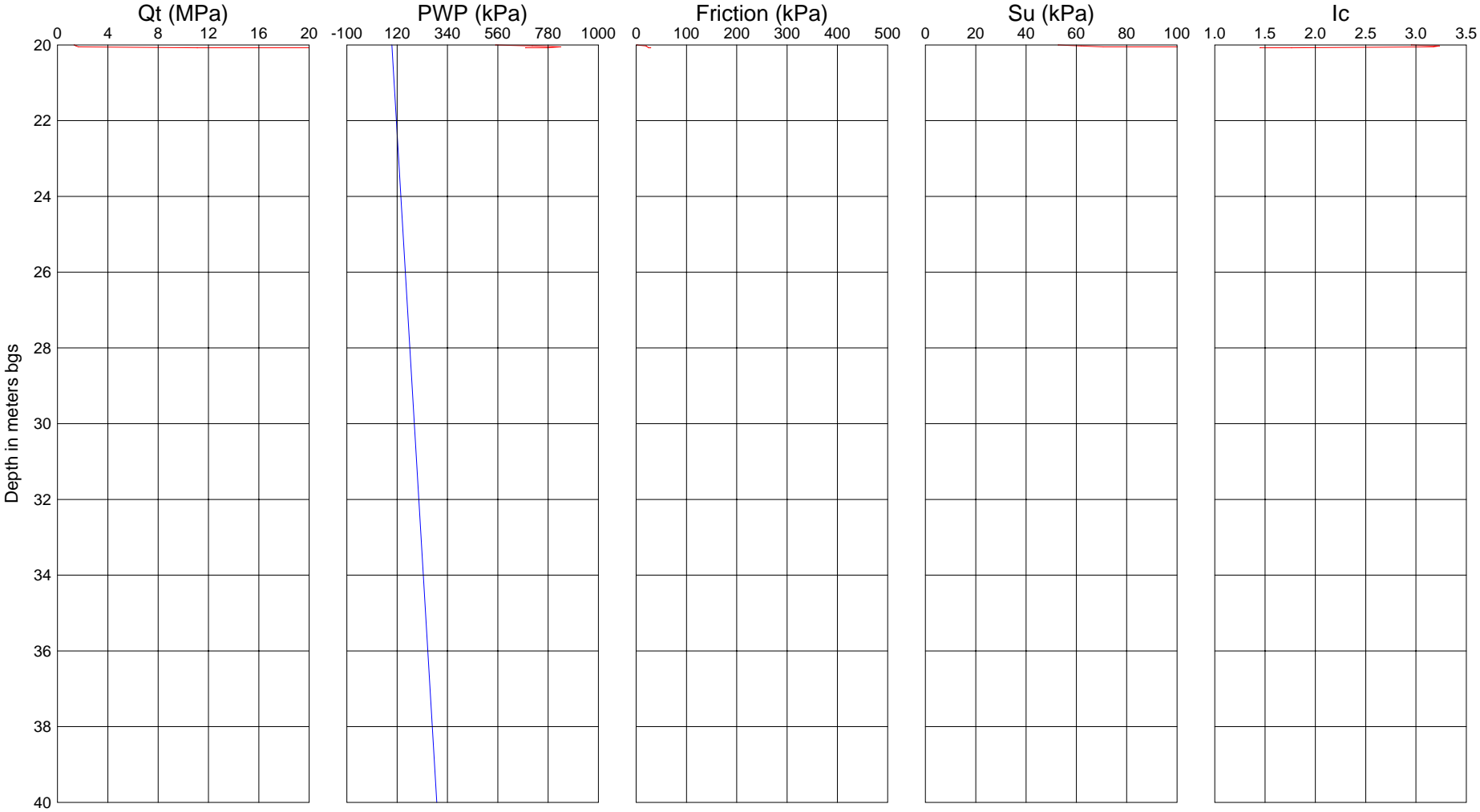
After Robertson and (Fear) Wride (1998)
 $I_c < 1.31$ - Gravelly sands
 $1.31 < I_c < 2.05$ - Clean to silty sand
 $2.05 < I_c < 2.60$ - Silty sand to sandy silt
 $2.60 < I_c < 2.95$ - Clayey silt to silty clay
 $2.95 < I_c < 3.60$ - Clays

Cone Penetration Test - CPT02-405

Test Date : November 29, 2002
Location : East Side of CN Rail Overpass

Operator : Golder Associates

Ground Surf. Elev. : 90.70
Water Table Depth : 10.10



Qt normalized for
unequal end area effects

$Su = (Qt - \sigma_v) / Nk$
Nk = 18
Gamma = 19 kN/m³

After Robertson and (Fear) Wride (1998)
Ic < 1.31 - Gravelly sands
1.31 < Ic < 2.05 - Clean to silty sand
2.05 < Ic < 2.60 - Silty sand to sandy silt
2.60 < Ic < 2.95 - Clayey silt to silty clay
2.95 < Ic < 3.60 - Clays

TABLE 1
EVALUATION OF FOUNDATION ALTERNATIVES
OCR OVERPASS STRUCTURE REPLACEMENT
G.W.P. 4254-05-01

<i>Footing Option</i>	<i>Rank</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Costs</i>	<i>Risks/Consequences</i>
Steel H Piles (driven to bedrock)	1	Relatively straight forward construction. Negligible settlement. Allows for integral abutment design at abutments.	Downdrag loads and lateral soil deformations have to be considered in design at both abutments and piers. Cobbles and boulders may be encountered during pile driving at abutments. Potential for conflict with existing (original) steel pile foundations during driving of new piles.	Less expensive than rock-socketted caisson or micropile option.	If existing piles are encountered during new pile driving operations, there may be a need to redesign the pile arrangement and pile cap.
Micropiles	2	Can more easily handle (i.e. drill through) cobbles and boulders if encountered. Numerous piles allows for redundancy in design and therefore flexibility during installation if existing piles encountered.	Smaller size piles have lower capacities than other pile foundation options and therefore more piles required to be installed. Downdrag loads and lateral soil deformations have to be considered in design.	More expensive than steel H-pile option.	
Cast-in-Place Concrete Caissons (socketted into bedrock)	3	Less piles to install, therefore greater likelihood of avoiding conflicts with existing pile foundations. High capacity. Negligible settlement.	Downdrag loads have to be considered in design and these loads will be relatively higher than those for the steel H-pile and micropile options. Permanent casings will be required to construct caissons. Cobbles and boulders may be encountered during drilled shaft installation. Coring or churn drilling will be required to form sockets in medium strong bedrock.	More expensive than steel H-pile option.	Socketting into the medium strong bedrock could be difficult and time-consuming.
Spread Footings on native silty clay soils and/or perched within embankment fill	X		Low geotechnical resistance. Differential settlements will occur between east and west abutments and piers due to consolidation of underlying firm silty clay under footing loads and loads from approach embankment grade raises.	Lower relative costs than piled foundations.	Not recommended due to potential for differential settlements anticipated between east and west abutments and piers.

Note: 'X' - indicates that the founding option is considered not feasible at this site.

TABLE 2
EVALUATION OF RETAINING WALL FOUNDATION ALTERNATIVES
EAST APPROACH EMBANKMENT – OCR OVERPASS STRUCTURE REPLACEMENT
G.W.P. 4254-05-01

<i>Foundation Option</i>	<i>Rank</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Costs</i>	<i>Risks/Consequences</i>
Soldier Pile and Concrete Panel Wall	1	Wall constructed at toe of existing front slope with no requirement for excavation at toe. Avoids front slope stability issues.	Requires augered sockets through fill (potentially containing cobbles) and into weathered crust to support soldier piles. Sockets longer than about 3.5 m to 4 m (i.e. extending below Elevation 83) could encounter groundwater table potentially requiring temporary liners. Low head room beneath existing EBL structure may require special equipment.	Least expensive option.	No borehole information available on southern half of wall alignment. Risk that augered piles may extend below weathered crust into softer underlying clayey soil that offers lower lateral resistance.
Cellular Steel (Bin or Crib) Wall	2	Modular nature of bin wall construction lends itself to excavation and construction in short segments enhancing temporary stability of existing front slope. Design only requires nominal width (about 0.55xheight) so relatively small excavation into toe of existing slope required. Flexible and can tolerate expected settlement along wall.	Deep-corrugated steel facing may not be aesthetically pleasing to NCC. May require special facing to cover bins.	More expensive than soldier pile and lagging wall.	No borehole information available on southern half of wall alignment. Risk that sand and gravel fill may not be present along full alignment.
Cast-in-place Concrete Cantilever Wall	3	Aesthetically pleasing wall face. Can be designed to be constructed in short segments thereby enhancing stability during construction.	Deeper excavation required to ensure wall is founded below depth of frost penetration (1.8 m). Deeper excavation into toe of existing slope required for construction.	Most expensive option.	No borehole information available on southern half of wall alignment. Risk that sand and gravel fill may not be present along full alignment.
Mechanically Reinforced Soil Retaining Wall (RSS)	4	Aesthetically pleasing wall face. Can be designed to be constructed in short segments thereby enhancing stability during construction.	Deeper excavation into toe of existing slope required for construction to ensure adequate embedment of reinforcing strips in backfill.	More expensive than soldier pile and lagging wall.	No borehole information available on southern half of wall alignment. Risk that sand and gravel fill may not be present along full alignment.

TABLE 2
EVALUATION OF RETAINING WALL FOUNDATION ALTERNATIVES
EAST APPROACH EMBANKMENT – OCR OVERPASS STRUCTURE REPLACEMENT
G.W.P. 4254-05-01

<i>Foundation Option</i>	<i>Rank</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Costs</i>	<i>Risks/Consequences</i>
Gabion Wall	5	Can be designed to be constructed in short segments thereby enhancing stability during construction. Flexible and can tolerate expected settlement along wall.	Gabion style will not be aesthetically pleasing to NCC. Deeper excavation into toe of existing slope required for construction.	Less expensive option.	No borehole information available on southern half of wall alignment. Risk that sand and gravel fill may not be present along full alignment.
Soil Nail Wall	X				

Note: ‘X’ - indicates that the founding option is considered not feasible at this site.

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TABLE 3
EVALUATION OF SETTLEMENT MITIGATION ALTERNATIVES
EAST APPROACH EMBANKMENT – OCR OVERPASS STRUCTURE REPLACEMENT
G.W.P. 4254-05-01

<i>Settlement Mitigation Option</i>	<i>Rank</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Costs</i>	<i>Risks/Consequences</i>
Light Weight Fill (EPS) for Grade Raise (20 m long x 25 m wide x 1.5 m to 2 m high) and Ultra-lightweight slag on front and side slopes *EPS or slag fill also required beyond east limit of the east approach area – beyond Station 12+729 along Tall Rock Fill embankment)	1	Reduces loads on compressible soils thereby increasing stability and reducing settlement of foundation soils. Settlement of embankment fill minimized.	High material costs. Existing embankment grade would need to be excavated in order to install the EPS while allowing for a minimum 1 m of conventional fill over the EPS. Even larger excavation of existing embankment grade plus temporary shoring/roadway protection will be required for excavations up to 5 m deep if no EPS and all slag fill is employed.	Relative cost of fill is up to an order of magnitude higher than for the other options.	Settlements of foundation soils and embankment fill minimized.
Pre-Loading	2	Relatively simple operation; no subexcavation or temporary shoring required.	Lengthened construction time required. Only about 30% of the primary consolidation settlement of foundation soil due to loading from conventional fill completed in the 6 month preload period. Post-construction primary settlements ranging from about 80 mm to 95 mm (plus creep settlements) will still occur.	Relatively low cost.	Post-construction settlement of embankment/ foundation soils will occur requiring long-term maintenance.
Surcharging and Toe Berms	X	Relatively simple operation; no subexcavation or temporary shoring required. Reduced post-construction settlements.	Toe berm required for stability of south slope and may be required for stability of front slope (depending on height of surcharge adopted).	Increased cost of construction and material for surcharge and toe berms.	Not recommended due to potential for conflict between OCR tracks if stability berm required at toe of front slope.
Wick Drains	X	Reduce the preload / surcharge duration.	Increased time for installation of wicks. Potential difficulties installing through existing thick embankment fill. Monitoring of settlements and pore pressures required.	Additional costs associated with wicks, pre-drilling costs for installation, and instrumentation and monitoring.	Not practical due to the additional efforts required to install through existing embankment fills.

TABLE 3
EVALUATION OF SETTLEMENT MITIGATION ALTERNATIVES
EAST APPROACH EMBANKMENT – OCR OVERPASS STRUCTURE REPLACEMENT
G.W.P. 4254-05-01

<i>Settlement Mitigation Option</i>	<i>Rank</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Costs</i>	<i>Risks/Consequences</i>
Full Subexcavation and Replacement (through up to 12 m of existing embankment fill and up to 12 m of sensitive silty clay)	X	Post-construction stability and long-term settlement issues minimized since all or nearly all firm and compressible materials are removed.	Very deep excavations required beneath and adjacent to existing roadway that is to remain in operation. Extensive shoring/roadway protection would be required.	Additional costs for sub-excavation and disposal of soft soils. Additional costs for partial removal of existing embankment and shoring/roadway protection.	Not practical due to deep excavations through existing embankment fill and through sensitive clayey strata.

Note: ‘X’ - indicates that the founding option is considered not feasible at this site.

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TABLE 4
EVALUATION OF SETTLEMENT AND STABILITY MITIGATION ALTERNATIVES
WEST APPROACH EMBANKMENT – OCR OVERPASS STRUCTURE REPLACEMENT
G.W.P. 4254-05-01

<i>Stability/ Settlement Mitigation Option</i>	<i>Rank</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Costs</i>	<i>Risks/Consequences</i>
Light Weight Fill (EPS) for Grade Raise (20 m long* x 25 m wide x 2 m to 3 m high) and Ultra-lightweight slag on front and side slopes *EPS fill also required beyond west limit of the west approach area – Station 12+664 to 12+576 (up to about 90 m long x 19 m wide x 2 m high)	1	Reduces loads on compressible soils thereby increasing stability and reducing settlement of foundation soils. Settlement of embankment fill minimized.	High material costs. Existing embankment grade would need to be excavated in order to install the EPS while allowing for a minimum 1 m of conventional fill over the EPS. Even larger excavation of existing embankment grade plus temporary shoring/roadway protection will be required for excavations up to 5 m deep if no EPS and all slag fill is employed.	Relative cost of fill is up to an order of magnitude higher than for the other options.	Settlements of foundation soils and embankment fill minimized. Factor of safety for stability increased between Stations 12+664 and 12+576.
Pre-Loading and Toe Berm (20 m long* x 14 m wide x 5 m high) *Toe berm also required beyond west limit of the west approach area – Station 12+664 to 12+576 (up to about 90 m long)	2	Relatively simple operation; no subexcavation or temporary shoring required.	Lengthened construction time required and additional effort to construct toe berm. Only about 25% of the primary consolidation settlement of foundation soil due to loading from conventional fill completed in the 6 month preload period. Post-construction primary settlements ranging from about 100 mm to 115 mm (plus creep settlements) will still occur. Toe berms required for stability at toe of south slope.	Relatively low cost.	Post-construction settlement of embankment/ foundation soils will occur requiring long-term maintenance.
Surcharging and Toe Berms	X	Relatively simple operation; no subexcavation or temporary shoring required. Reduced post-construction settlements.	Larger toe berm required for stability of south slope. Toe berm also required for stability of front slope which will interfere with operation of OCR track.	Increased cost of construction and material for surcharge and toe berms.	Not recommended due to conflict between OCR tracks and required stability berm at toe of front slope.
Wick Drains	X	Reduce the preload / surcharge duration.	Increased time for installation of wicks. Potential difficulties installing through existing thick embankment fill. Monitoring of settlements and pore pressures required.	Additional costs associated with wicks, possible pre-drilling costs for installation, and instrumentation and monitoring.	Not practical due to the additional efforts required to install through existing embankment fills.

TABLE 4
EVALUATION OF SETTLEMENT AND STABILITY MITIGATION ALTERNATIVES
WEST APPROACH EMBANKMENT – OCR OVERPASS STRUCTURE REPLACEMENT
G.W.P. 4254-05-01

<i>Stability/ Settlement Mitigation Option</i>	<i>Rank</i>	<i>Advantages</i>	<i>Disadvantages</i>	<i>Relative Costs</i>	<i>Risks/Consequences</i>
Full Subexcavation and Replacement (through up to 6 m of existing embankment fill and up to 16 m of sensitive silty clay)	X	Post-construction stability and long-term settlement issues minimized since all or nearly all firm and compressible materials are removed.	Very deep excavations required beneath and adjacent to existing roadway that is to remain in operation. Extensive shoring/roadway protection would be required.	Additional costs for sub-excavation and disposal of soft soils. Additional costs for partial removal of existing embankment and shoring/roadway protection.	Not practical due to deep excavations through existing embankment fill and through sensitive clayey strata.

Note: ‘X’ - indicates that the founding option is considered not feasible at this site.

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TABLE 5
DETAILS OF FILL-TYPE REQUIREMENTS TO ACHIEVE POST-CONSTRUCTION SETTLEMENT CRITERIA
EAST EMBANKMENT - OCR OVERPASS STRUCTURE REPLACEMENT
G.W.P. 4254-05-01

Location		Total Distance Behind Abutment (m)	Fill Type	Fill Thickness Required to Meet Post-Construction Settlement Criteria			
From Station	To Station			Left of 17 m LT	From 17 m LT to 3 m LT	From 3 m LT to 17 m RT	Right of 17 m RT
				(Median Side Slope)	(Existing Median)	(Existing Lane)	(Emb. Side Slope)
12+709 (East Abutment)	12+719	10	EPS	-	2.5 m	1.5 m	-
			Slag	Up to 2 m	-	-	Up to 6 m
			Earth Fill	-	-	-	-
12+719	12+729	20	EPS	-	2 m	1 m	-
			Slag	Up to 2 m	-	-	Up to 6 m
			Earth Fill	-	0.5 m	0.5 m	-
12+729	12+739	30	EPS	-	-	-	-
			Slag	Up to 2 m	2.5 m	1.5 m*	Up to 5.5 m
			Earth Fill	-	-	-	-
12+739	12+779	70	EPS	-	-	-	-
			Slag	-	2.5 m	1.5 m	-
			Earth Fill	Up to 2 m	-	-	Up to 5.5 m
12+779	12+879	170	EPS	-	-	-	-
			Slag	-	-	-	-
			Earth Fill	Up to 2 m	2.5 m	1.5 m	Up to 1 m

Note: All offsets are relative to the new centreline of the EBL

*Settlement criteria exceeded by up to 10 mm on the RT shoulder in this area

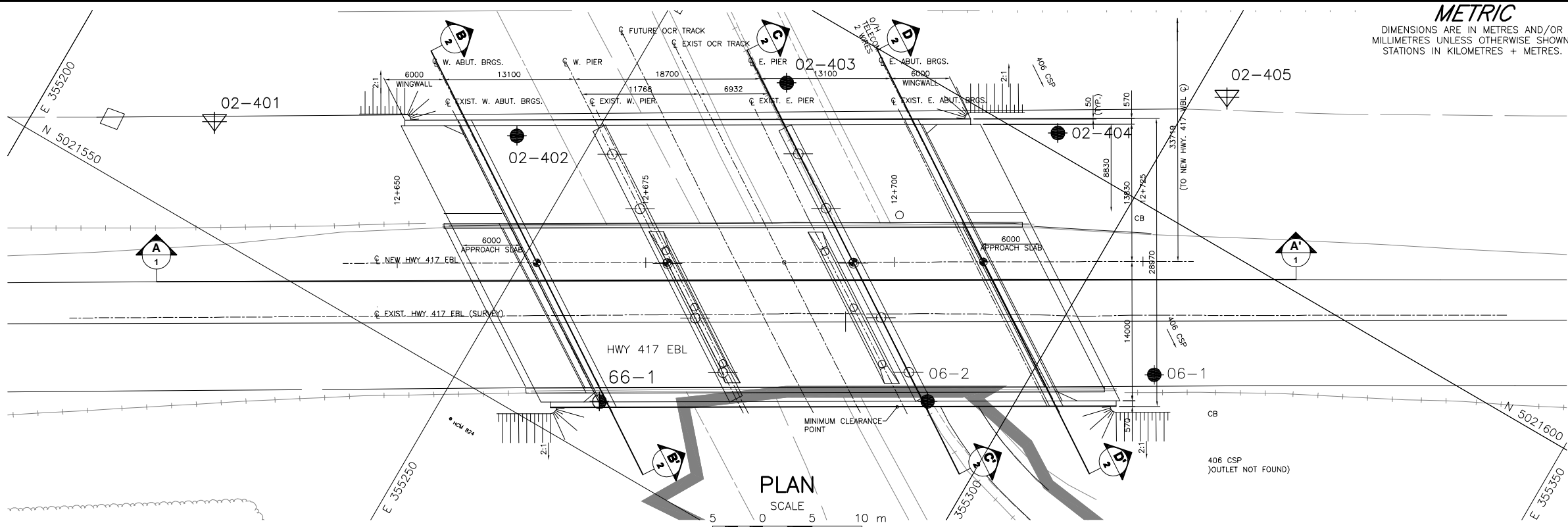
n:\active\2002\1100\021-1155\reports\final reports\ocr structure\tables\table5(final)_details of fill-type requirements to achieve post-construction settlement criteria.doc

TABLE 6
DETAILS OF FILL-TYPE REQUIREMENTS TO ACHIEVE POST-CONSTRUCTION SETTLEMENT CRITERIA
WEST EMBANKMENT - OCR OVERPASS STRUCTURE REPLACEMENT
G.W.P. 4254-05-01

Location		Total Distance Behind Abutment (m)	Fill Type	Fill Thickness Required to Meet Post-Construction Settlement Criteria			
From Station	To Station			Left of 14 m LT	From 14 m LT to 4 m LT	From 4 m LT to 15 m RT	Right of 15 m RT
				(Median Side Slope)	(Existing Median)	(Existing Lane)	(Emb. Side Slope)
12+664 (West Abutment)	12+654	10	EPS	-	2.5 m	2 m	-
			Slag	Up to 2 m	-	-	Up to 5 m
			Earth Fill	-	-	-	-
12+654	12+634	30	EPS	-	2 m	2 m**	-
			Slag	Up to 2 m	-	-	Up to 5 m
			Earth Fill	-	-	-	-
12+634	12+594	70	EPS	-	-	2 m**	-
			Slag	-	2.5 m	-	-
			Earth Fill	Up to 3 m	-	-	Up to 6 m
12+594	12+574	90	EPS	-	-	2 m**	-
			Slag	-	-	-	-
			Earth Fill	Up to 3 m	2 m	-	Up to 6 m
12+574	12+494	170	EPS	-	-	-	-
			Slag	-	-	-	-
			Earth Fill	Up to 2.5 m	2 m	1.5 m	Up to 2 m

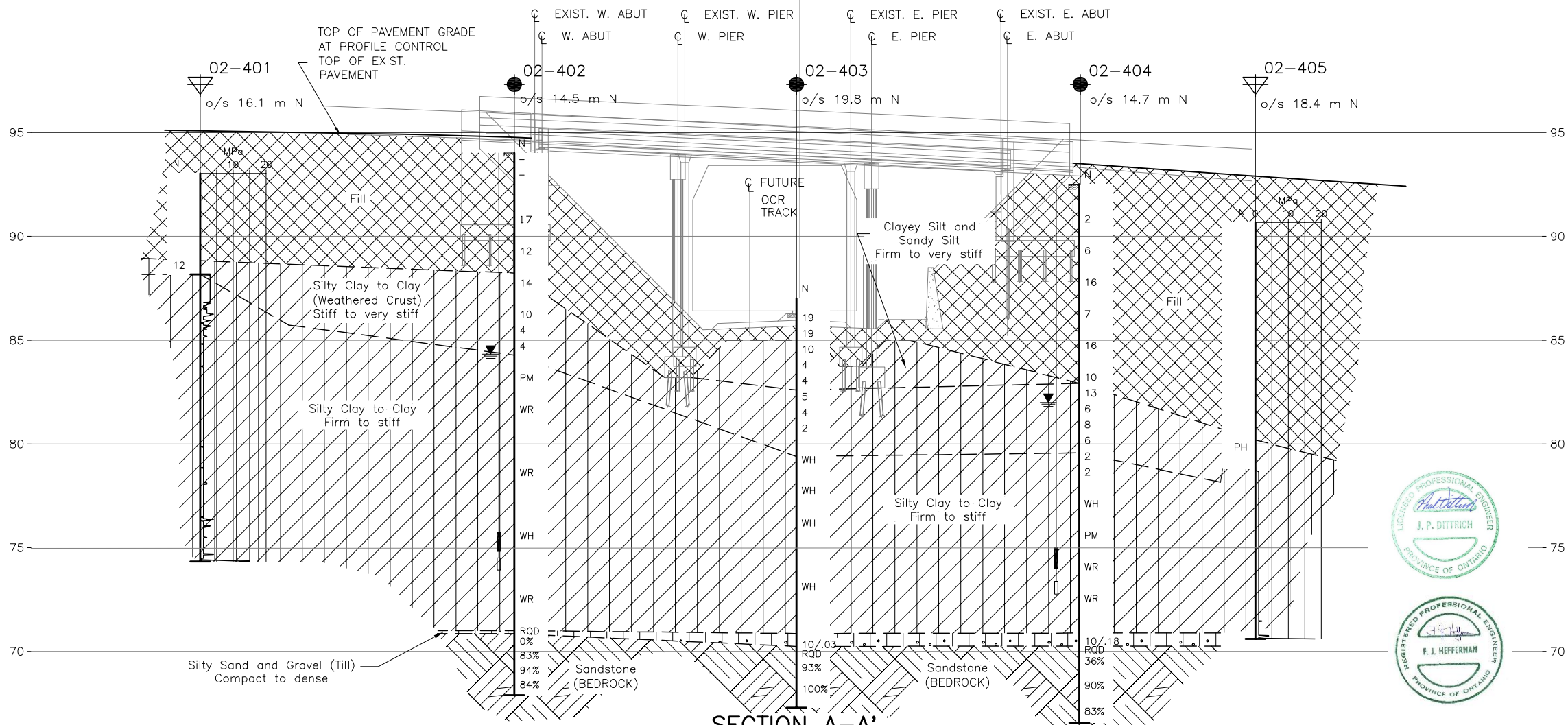
Note: All offsets are relative to the new centreline of the EBL

** 2m thickness of EPS required in these areas to achieve $FoS > 1.3$ for side slope stability without toe berm



PLAN

SCALE
5 0 5 10 m



SECTION A-A'

SCALE
2.5 0 2.5 5 m

REFERENCE

GA plans provided in digital format by Totten Simms Hubicki, drawing file no. 42-91061_HWY417-EBL_1-GA.dwg on April 7, 2006.
Base plans provided in digital format by Marshall Macklin Monaghan, on December 19, 2003.

CONT No.
WP No. 4254-05-01

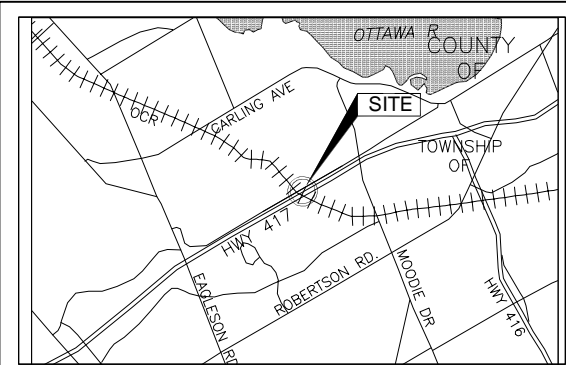
OCR OVERPASS REPLACEMENT
BOREHOLE LOCATIONS AND
SOIL STRATA



SHEET



Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

SCALE
1.5 0 1.5 km

LEGEND

- Borehole - Current Investigation
- Borehole and DCPT - 1966 Investigation
- Cone Penetration Test
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer
- WL upon completion of drilling

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
02-401	93.0	5021560.1	355216.3
02-402	94.0	5021574.1	355243.2
02-403	87.0	5021592.4	355264.0
02-404	92.5	5021601.8	355290.1
02-405	90.7	5021613.4	355303.0
06-1	92.7	5021585.6	355310.7
06-2	86.5	5021571.9	355292.4
66-1	88.7	5021555.2	355263.9

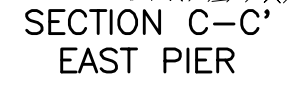
NOTES

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

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

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

NO.	DATE	BY	REVISION
Geocres No.			
HWY. 417	PROJECT NO. 021-1155		DIST.
SUBM'D. LCC	CHKD. FJH	DATE: NOV 2006	SITE: 3-257
DRAWN: MSM	CHKD. JPD	APPD.	DWG. 1

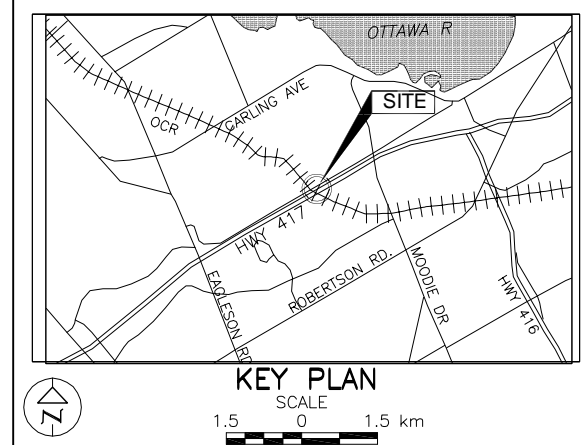


METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
E STATIONS IN KILOMETRES + METRES.








CONT No.
WP No. 4254-05-01

OCR OVERPASS REPLACEMENT
SOIL STRATA

SHEET



LEGEND

- | | |
|---|--|
|  | Borehole – Current Investigation |
|  | Borehole and DCPT – 1966 Investigation |
|  | Cone Penetration Test |
|  | Seal |
|  | Piezometer |
| N | Standard Penetration Test Value |
| 16 | Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow) |
| 100% | Rock Quality Designation (RQD) |
|  | WL in piezometer |
|  | WL upon completion of drilling |

No.	ELEVATION	CO—ORDINATES	
		NORTHING	EASTING
02—401	93.0	5021560.1	355216.3
02—402	94.0	5021574.1	355243.2
02—403	87.0	5021592.4	355264.0
02—404	92.5	5021601.8	355290.1
02—405	90.7	5021613.4	355303.0
06—1	92.7	5021585.6	355310.7
06—2	86.5	5021571.9	355292.4
66—1	88.7	5021555.2	355263.9

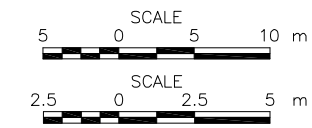
NOTES

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

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

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

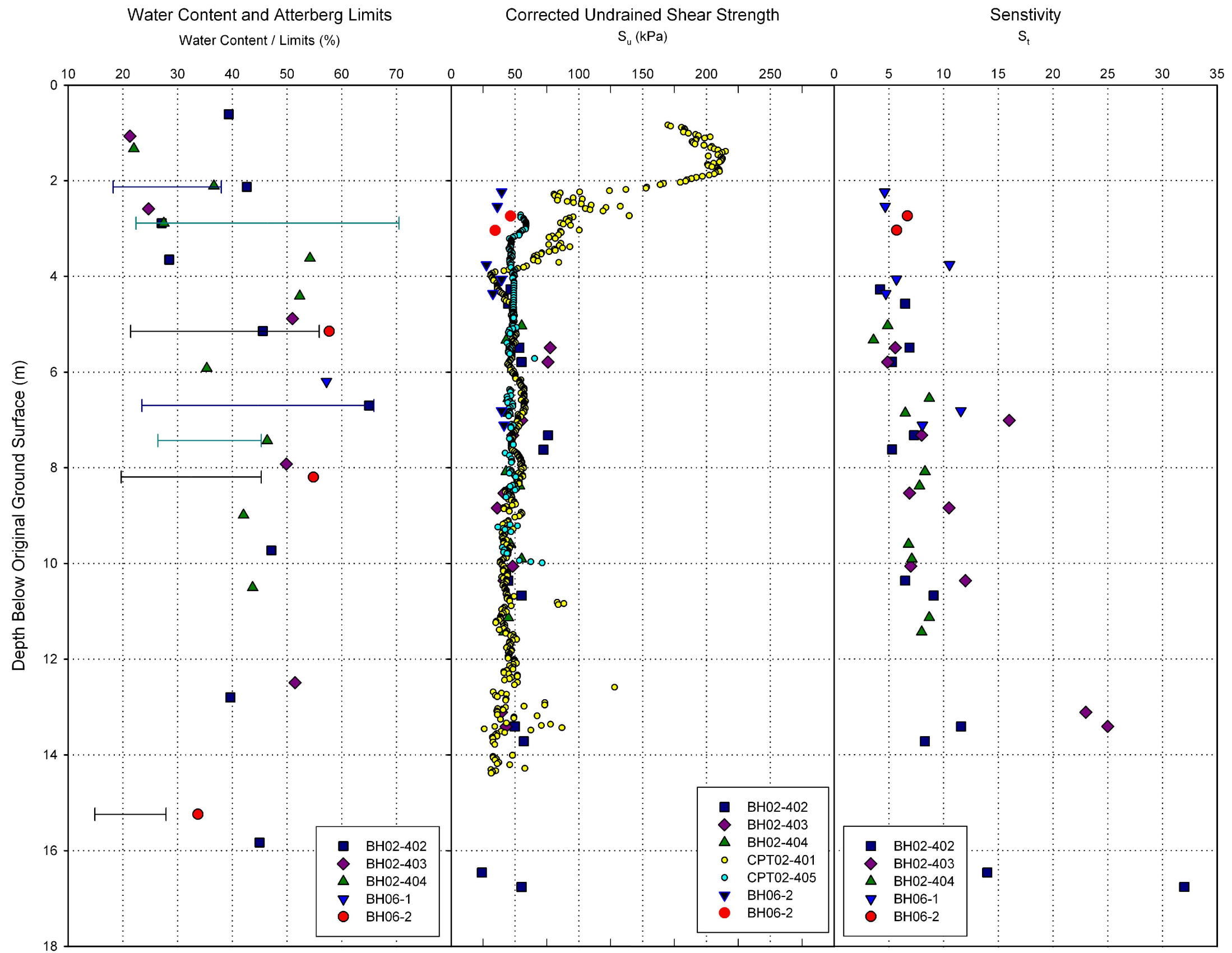
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Geocres No.				
HWY. 417		PROJECT NO. 021-1155		DIST.
SUBM'D. LCC	CHKD. FJH	DATE: NOV 2006		SITE: 3-257
DRAWN: MSM	CHKD. JPD	APPD.		DWG. 2




REFERENCE

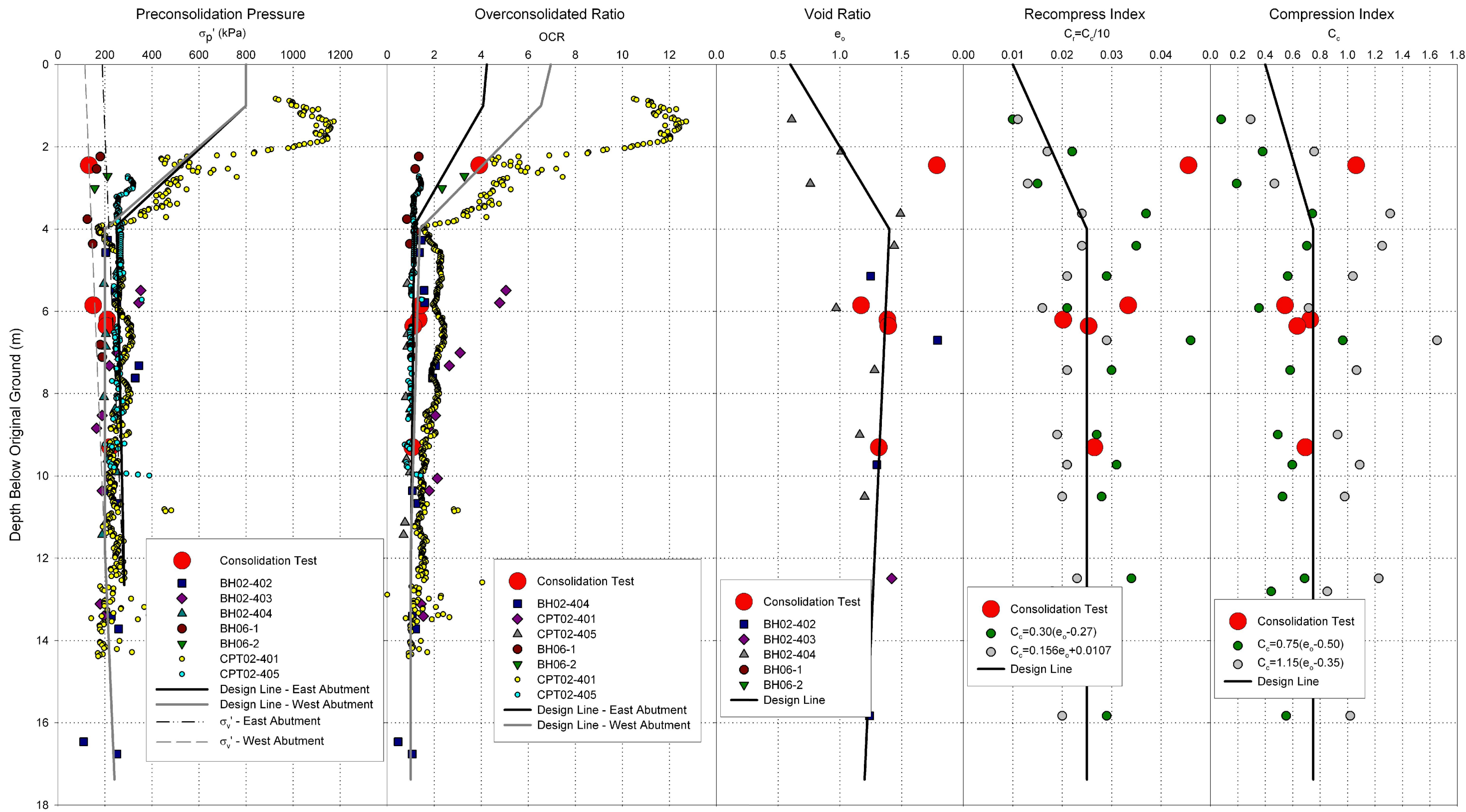
GA plans provided in digital format by Totten Simms Hubicki, drawing file no. 42-91061_HWY417-EBL_1-GA.dwg on April 7, 2006.
Base plans provided in digital format by Marshall Macklin Monaghan, on December 19, 2003.


PLOT DATE: November 24, 2006
FILENAME: T:\Projects\2002\021-1155\CA- (5100 Figures)\0211155CA001.dwg

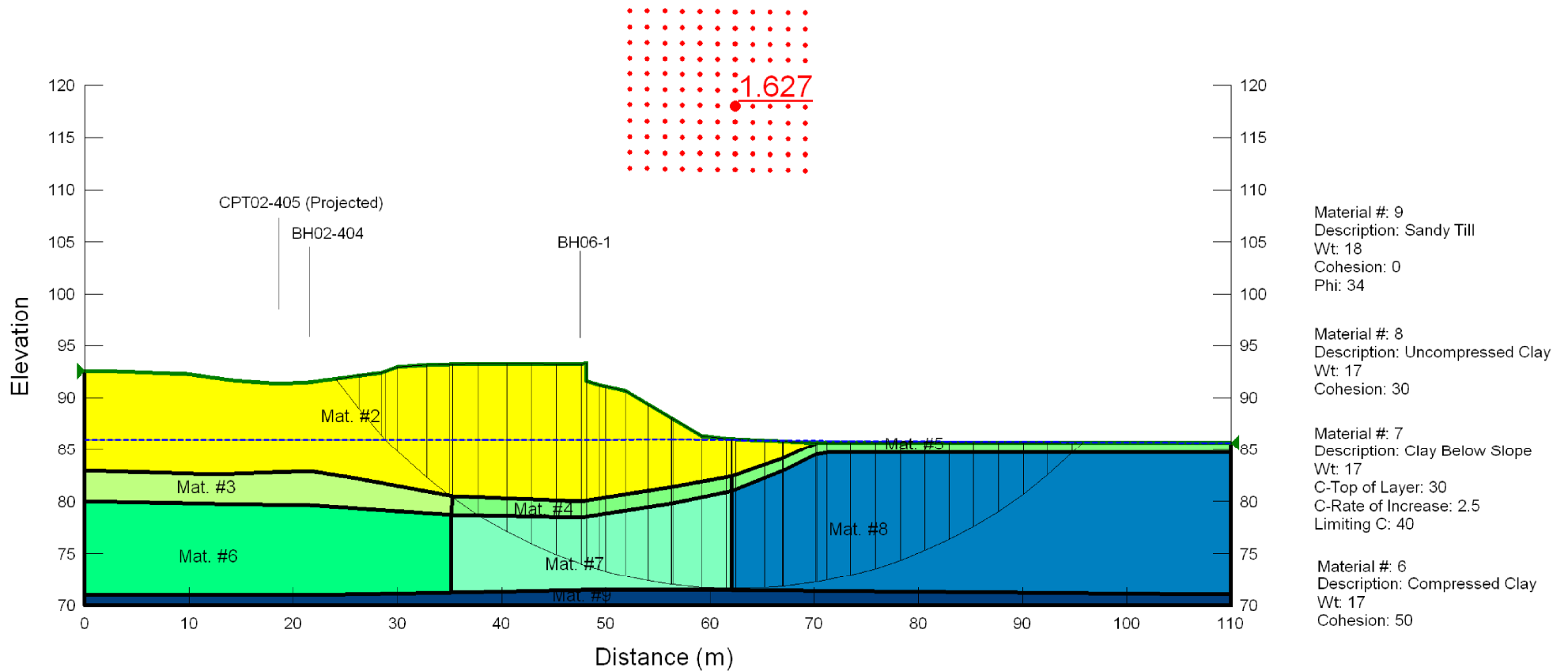


 Mississauga, Ontario, Canada			SCALE NOT TO SCALE		TITLE Summary of Water Contents, Atterberg Limits and Undrained Shear Strengths	
			DATE NOV. 24, 2006			
			DESIGN ALL			
			CAD MSM			
FILE No. 0211155CA001.dwg			CHECK ALL		HIGHWAY 417 (EBL) OCR OVERPASS BRIDGE REPLACEMENT	
PROJECT No. 021-1155		REV.	REVIEW JPD			
FIGURE 1						

PLOT DATE: November 24, 2006
FILENAME: T:\Projects\2002\021-1155\CA- (5100 Figures)\0211155CA002.dwg



 Mississauga, Ontario, Canada		SCALE: NOT TO SCALE		TITLE: Summary of Consolidation Properties	
		DATE: NOV. 24, 2006			
FILE No. 0211155CA002.dwg		CHECK: CN	HIGHWAY 417 (EBL) OCR OVERPASS BRIDGE REPLACEMENT		
PROJECT No. 021-1155	REV.	REVIEW: JPD			
			FIGURE: 2		



SCALE NOT TO SCALE
 DATE OCT. 3, 2006
 DESIGN ALL
 CAD MSM
 CHECK ALL
 REVIEW JPD

TITLE

Existing East Approach Embankment

FILE No. 0211155CA003.dwg

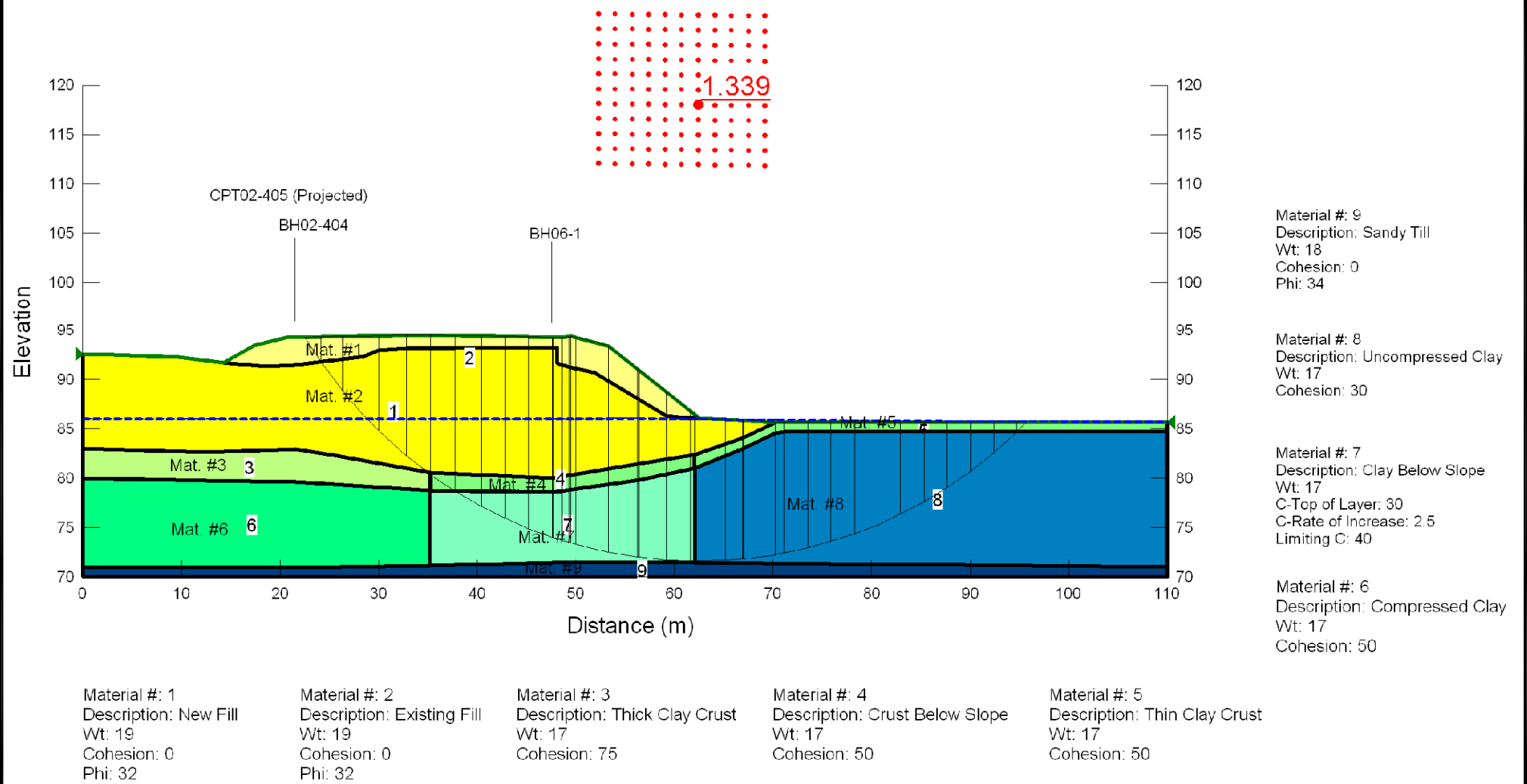
PROJECT No. 021-1155


REV.

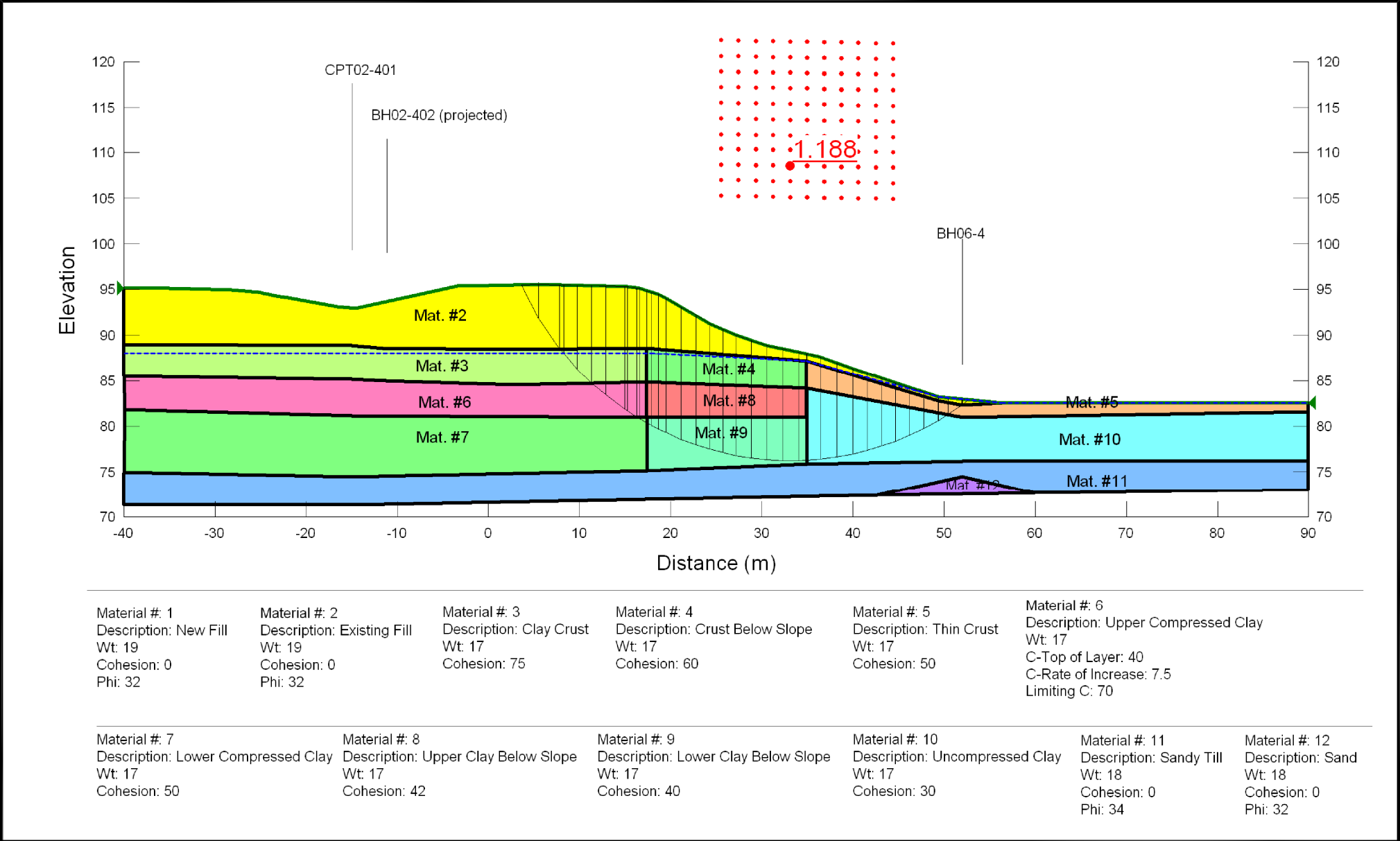
HIGHWAY 417 (EBL) OCR OVERPASS BRIDGE REPLACEMENT

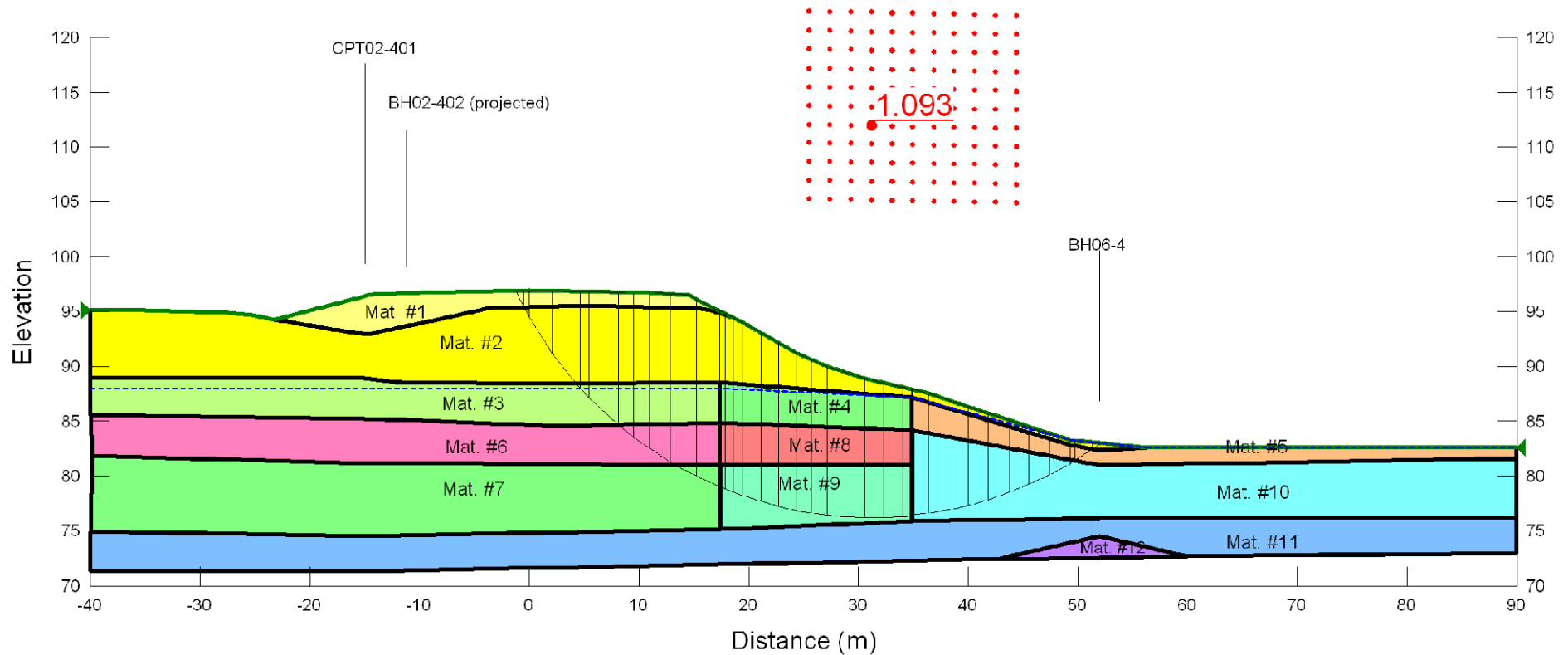
FIGURE

3



 Mississauga, Ontario, Canada			SCALE	NOT TO SCALE	TITLE	Proposed East Approach Embankment
			DATE	OCT. 3, 2006		
			DESIGN	ALL		
			CAD	MSM		
			CHECK	ALL		
FILE No. 0211155CA004.dwg			REVIEW	JPD	HIGHWAY 417 (EBL) OCR OVERPASS BRIDGE REPLACEMENT	FIGURE 4
PROJECT No. 021-1155		REV.				





Material #: 1 Description: New Fill Wt: 19 Cohesion: 0 Phi: 32	Material #: 2 Description: Existing Fill Wt: 19 Cohesion: 0 Phi: 32	Material #: 3 Description: Clay Crust Wt: 17 Cohesion: 75	Material #: 4 Description: Crust Below Slope Wt: 17 Cohesion: 60	Material #: 5 Description: Thin Crust Wt: 17 Cohesion: 50	Material #: 6 Description: Upper Compressed Clay Wt: 17 C-Top of Layer: 40 C-Rate of Increase: 7.5 Limiting C: 70
Material #: 7 Description: Lower Compressed Clay Wt: 17 Cohesion: 50	Material #: 8 Description: Upper Clay Below Slope Wt: 17 Cohesion: 42	Material #: 9 Description: Lower Clay Below Slope Wt: 17 Cohesion: 40	Material #: 10 Description: Uncompressed Clay Wt: 17 Cohesion: 30	Material #: 11 Description: Sandy Till Wt: 18 Cohesion: 0 Phi: 34	Material #: 12 Description: Sand Wt: 18 Cohesion: 0 Phi: 32



SCALE	NOT TO SCALE
DATE	OCT. 3, 2006
DESIGN	ALL
CAD	MSM
CHECK	ALL
REVIEW	JPD

TITLE

Proposed West Approach Embankment

FILE No. 0211155CA006.dwg

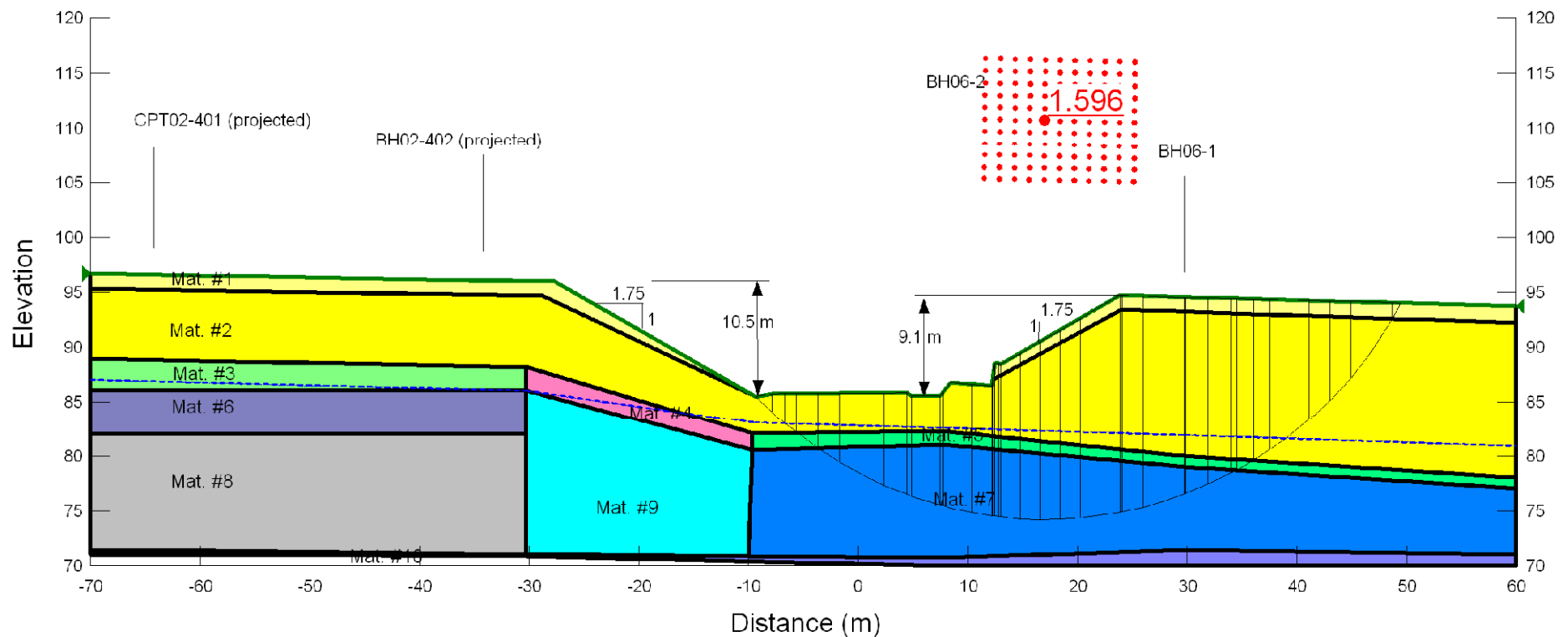
PROJECT No. 021-1155

REV.

HIGHWAY 417 (EBL) OCR OVERPASS BRIDGE REPLACEMENT

FIGURE

6



Material #: 1
 Description: New Fill
 Wt: 19
 Cohesion: 0
 Phi: 32

Material #: 2
 Description: Existing Fill
 Wt: 19
 Cohesion: 0
 Phi: 32

Material #: 3
 Description: Thick Clay Crust
 Wt: 17
 Cohesion: 75

Material #: 4
 Description: Crust Below Slope
 Wt: 17
 Cohesion: 60

Material #: 5
 Description: Thin Clay Crust
 Wt: 17
 Cohesion: 50

Material #: 6
 Description: Upper Compressed Clay
 Wt: 17
 C-Top of Layer: 40
 C-Rate of Increase: 7.5
 Limiting C: 70

Material #: 7
 Description: Firm Silty Clay East
 Wt: 17
 Cohesion: 35

Material #: 8
 Description: Firm Silty Clay West
 Wt: 17
 Cohesion: 50

Material #: 9
 Description: Silty Clay Below Slope
 Wt: 17
 Cohesion: 40

Material #: 10
 Description: Sandy Till
 Wt: 18
 Cohesion: 0
 Phi: 34



SCALE NOT TO SCALE
 DATE OCT. 3, 2006
 DESIGN ALL
 CAD MSM
 CHECK ALL
 REVIEW JPD

TITLE

Proposed East Front Slope

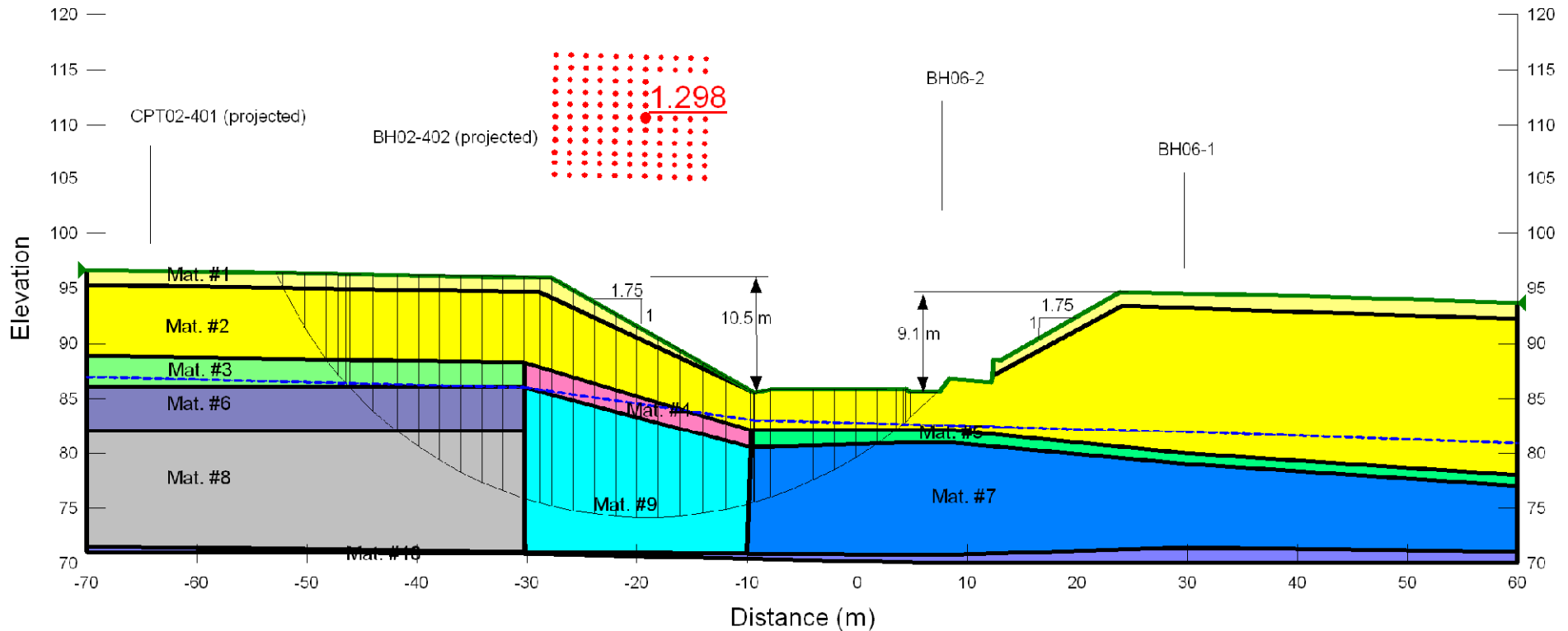
FILE No. 0211155CA007.dwg

PROJECT No. 021-1155 REV.

HIGHWAY 417 (EBL) OCR OVERPASS BRIDGE REPLACEMENT

FIGURE

7



Material #: 1
 Description: New Fill
 Wt: 19
 Cohesion: 0
 Phi: 32

Material #: 2
 Description: Existing Fill
 Wt: 19
 Cohesion: 0
 Phi: 32

Material #: 3
 Description: Thick Clay Crust
 Wt: 17
 Cohesion: 75

Material #: 4
 Description: Crust Below Slope
 Wt: 17
 Cohesion: 60

Material #: 5
 Description: Thin Clay Crust
 Wt: 17
 Cohesion: 50

Material #: 6
 Description: Upper Compressed Clay
 Wt: 17
 C-Top of Layer: 40
 C-Rate of Increase: 7.5
 Limiting C: 70

Material #: 7
 Description: Firm Silty Clay East
 Wt: 17
 Cohesion: 35

Material #: 8
 Description: Firm Silty Clay West
 Wt: 17
 Cohesion: 50

Material #: 9
 Description: Silty Clay Below Slope
 Wt: 17
 Cohesion: 40

Material #: 10
 Description: Sandy Till
 Wt: 18
 Cohesion: 0
 Phi: 34



SCALE NOT TO SCALE
 DATE OCT. 3, 2006
 DESIGN ALL
 CAD MSM
 CHECK ALL
 REVIEW JPD

TITLE

Proposed West Front Slope

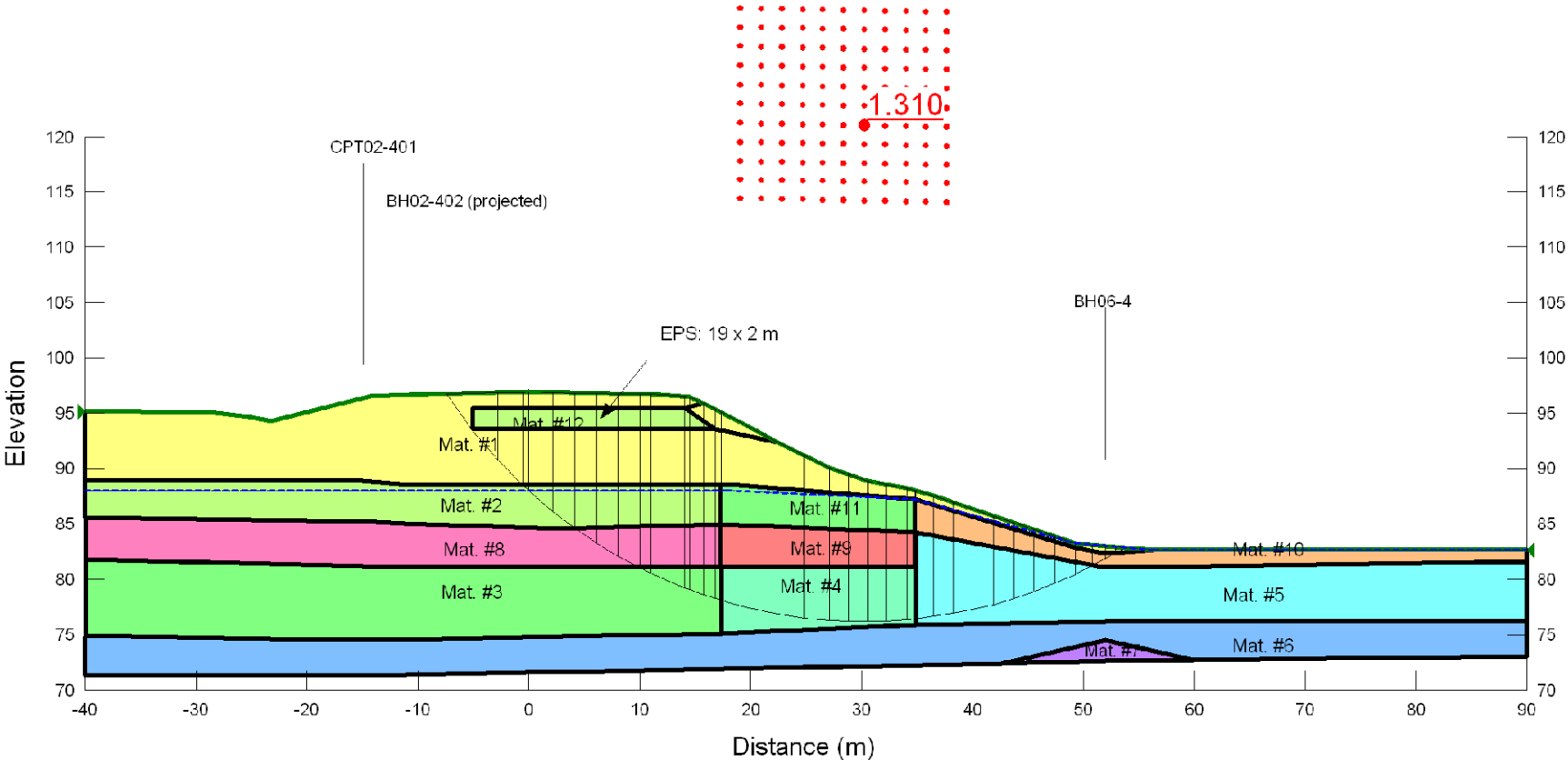
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PROJECT No. 021-1155 REV.

HIGHWAY 417 (EBL) OCR OVERPASS BRIDGE REPLACEMENT

FIGURE

8



Material #: 1 Description: Fill Wt: 19 Cohesion: 0 Phi: 32	Material #: 2 Description: Clay Crust Wt: 17 Cohesion: 75	Material #: 3 Description: Lower Compressed Clay Wt: 17 Cohesion: 50	Material #: 4 Description: Lower Clay Below Slope Wt: 17 Cohesion: 40	Material #: 5 Description: Uncompressed Clay Wt: 17 Cohesion: 30	Material #: 6 Description: Sandy Till Wt: 18 Cohesion: 0 Phi: 34	Material #: 7 Description: Sand Wt: 18 Cohesion: 0 Phi: 32
Material #: 8 Description: Upper Compressed Clay Wt: 17 C-Top of Layer: 40 C-Rate of Increase: 7.5 Limiting C: 70	Material #: 9 Description: Upper Clay Below Slope Wt: 17 Cohesion: 42	Material #: 10 Description: Thin Crust Wt: 17 Cohesion: 50	Material #: 11 Description: Crust Below Slope Wt: 17 Cohesion: 60	Material #: 12 Description: EPS Wt: 0.31 Cohesion: 100		



SCALE	NOT TO SCALE
DATE	OCT. 3, 2006
DESIGN	ALL
CAD	MSM
CHECK	ALL
REVIEW	JPD

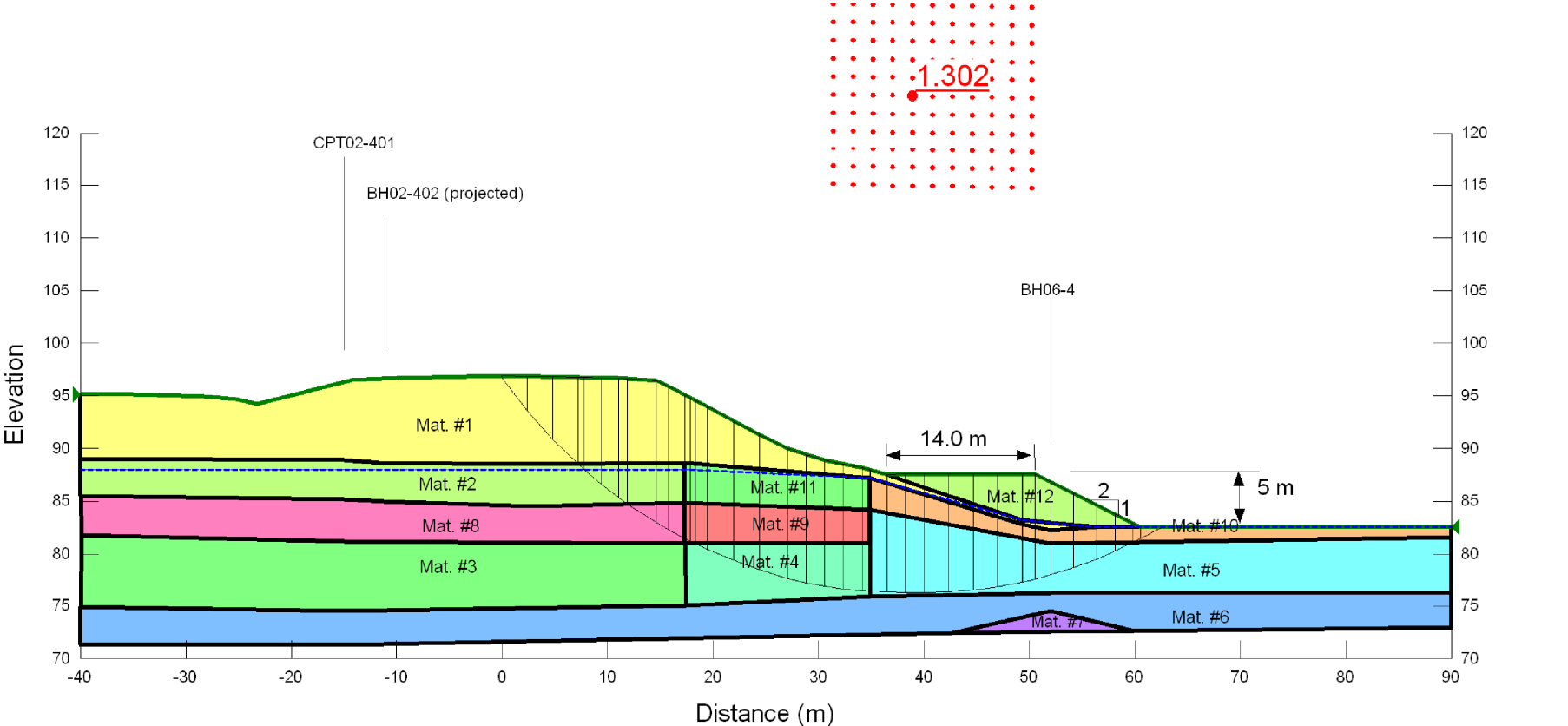
TITLE

Proposed EPS Mitigation Option

HIGHWAY 417 (EBL) OCR OVERPASS BRIDGE REPLACEMENT

FIGURE

9



Material #: 1 Description: Fill Wt: 19 Cohesion: 0 Phi: 32	Material #: 2 Description: Clay Crust Wt: 17 Cohesion: 75	Material #: 3 Description: Lower Compressed Clay Wt: 17 Cohesion: 50	Material #: 4 Description: Lower Clay Below Slope Wt: 17 Cohesion: 40	Material #: 5 Description: Uncompressed Clay Wt: 17 Cohesion: 30	Material #: 6 Description: Sandy Till Wt: 18 Cohesion: 0 Phi: 34	Material #: 7 Description: Sand Wt: 18 Cohesion: 0 Phi: 32
Material #: 8 Description: Upper Compressed Clay Wt: 17 C-Top of Layer: 40 C-Rate of Increase: 7.5 Limiting C: 70	Material #: 9 Description: Upper Clay Below Slope Wt: 17 Cohesion: 42	Material #: 10 Description: Thin Crust Wt: 17 Cohesion: 50	Material #: 11 Description: Crust Below Slope Wt: 17 Cohesion: 60	Material #: 12 Description: Fill for Berm Wt: 19 Cohesion: 28 Phi: 0		



SCALE	NOT TO SCALE
DATE	OCT. 3, 2006
DESIGN	ALL
CAD	MSM
CHECK	ALL
REVIEW	JPD

TITLE

Proposed Stabilizing Berm Mitigation Option

FILE No. 0211155CA0010.dwg

PROJECT No. 021-1155

REV.

HIGHWAY 417 (EBL) OCR OVERPASS BRIDGE REPLACEMENT

FIGURE

10

APPENDIX A
LABORATORY TEST DATA

TABLE A1

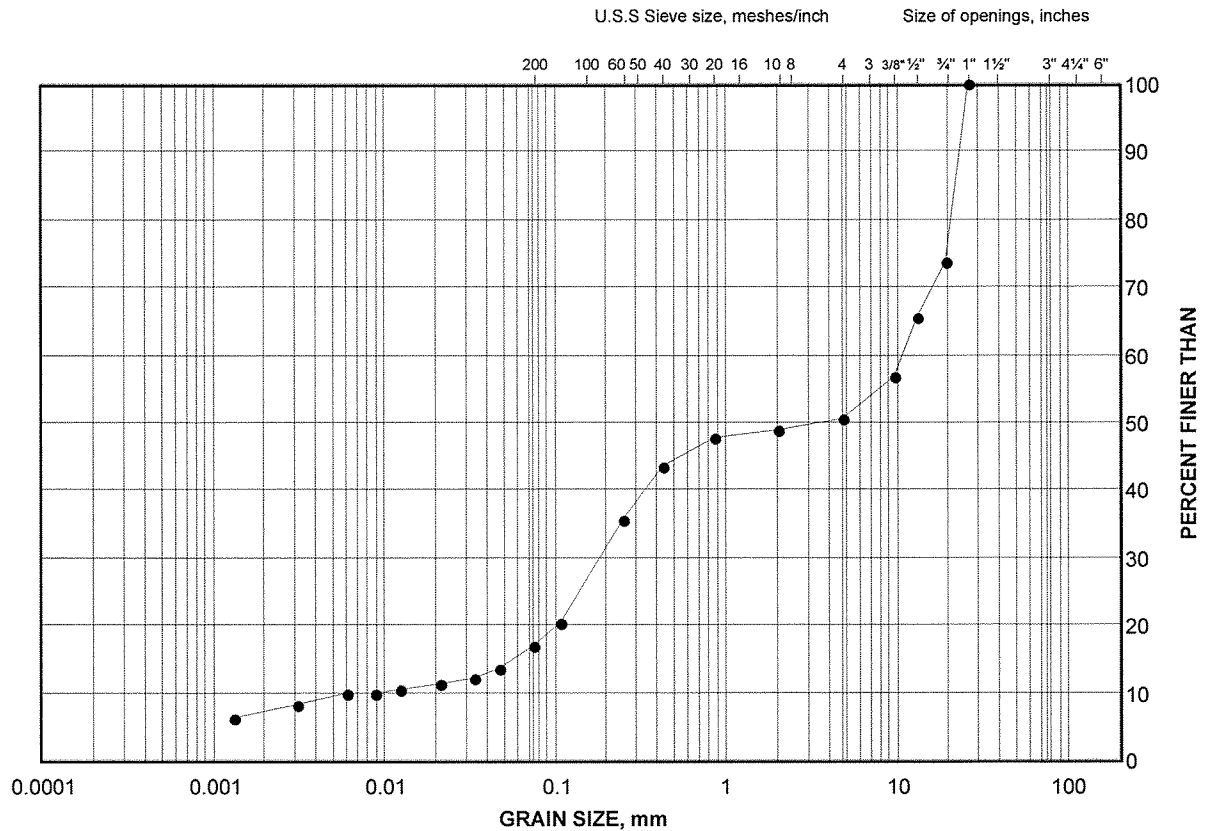
SUMMARY OF WATER CONTENT DETERMINATIONS

PROJECT NUMBER 021-1155					
PROJECT NAME		MMM / Design / Hwy 7			
DATE		October, 2006			
Borehole No.	Sample No.	Depth (ft)	Depth (m)	Water Content (%)	Atterberg Limits LL, PL, PI
02-402	1	0.5-2.0	0.15-0.61	19.8%	LL=38.0, PL=18.2, PI=19.8
02-402	2	3.0-5.0	0.91-1.52	7.9%	
02-402	5	20.0-22.0	6.10-6.71	39.4%	
02-402	6	25.0-27.0	7.62-8.23	42.6%	
02-402	7	27.5-29.5	8.38-8.99	27.1%	
02-402	8	30.0-32.0	9.14-9.75	28.5%	LL=65.9, PL=23.5, PI=42.4
02-402	9	35.0-37.0	10.67-11.28	45.6%	
02-402	10	40.0-42.0	12.19-12.80	65.0%	
02-402	11	50.0-52.0	15.24-15.85	47.1%	
02-402	12	60.0-62.0	18.29-18.90	39.7%	
02-402	13	70.0-72.0	21.34-21.95	45.0%	LL=70.5, PL=22.4, PI=48.1
02-403	3	7.5-9.5	2.29-2.90	21.3%	
02-403	5	12.5-14.5	3.81-4.42	24.7%	
02-403	8	20.0-22.0	6.10-6.71	51.0%	
02-403	10	30.0-32.0	9.14-9.75	49.9%	
02-403	12	45.0-47.0	13.72-14.33	51.5%	LL=45.3, PL=26.4, PI=18.9
02-404	1	5.0-7.0	1.52-2.13	21.5%	
02-404	4	20.0-22.0	6.10-6.71	4.3%	
02-404	8	35.0-37.0	10.67-11.28	22.0%	
02-404	9	37.5-39.5	11.43-12.04	36.6%	
02-404	10	40.0-42.0	12.19-12.80	27.5%	LL=55.9, PL=21.4, PI=34.5
02-404	11	42.5-44.5	12.95-13.56	54.2%	
02-404	12	45.0-47.0	13.72-14.33	52.3%	
02-404	13	50.0-52.0	15.24-15.85	35.4%	
02-404	14	55.0-57.0	16.76-17.37	46.4%	
02-404	15	60.0-62.0	18.29-18.90	42.0%	LL=45.3, PL=19.7, PI=25.6
02-404	16	65.0-67.0	19.81-20.42	43.7%	
06-1	2	5.0-7.0	1.52-2.13	3.5%	
06-1	6	25.0-27.0	7.62-8.23	4.9%	
06-1	13	60.0-62.0	18.29-18.90	57.2%	
06-2	7	30.0-32.0	9.14-9.75	57.7%	LL=27.9, PL=14.9, PI=13.0
06-2	9	40.0-42.0	12.19-12.80	54.8%	
06-2	11	50.0	15.2	33.7%	

GRAIN SIZE DISTRIBUTION

Sand and Gravel (Fill)

FIGURE A2



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
•	02-402	2	1.5

Project Number: 021-1155

Checked By: JPD

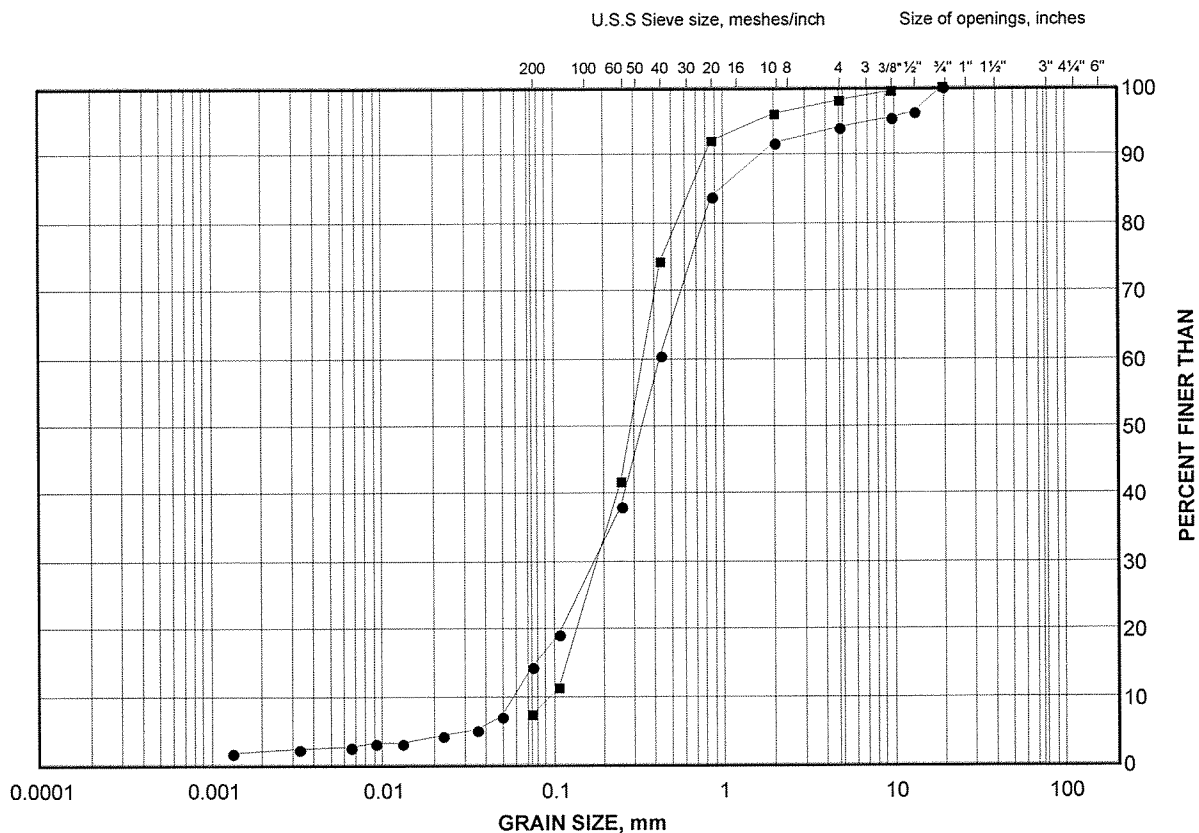
Golder Associates

Date: 18-Oct-06

GRAIN SIZE DISTRIBUTION

Sand (Fill)

FIGURE A3



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	02-404	4	6.7
■	06-1	6	7.60 - 8.20

Project Number: 021-1155

Checked By: JPD

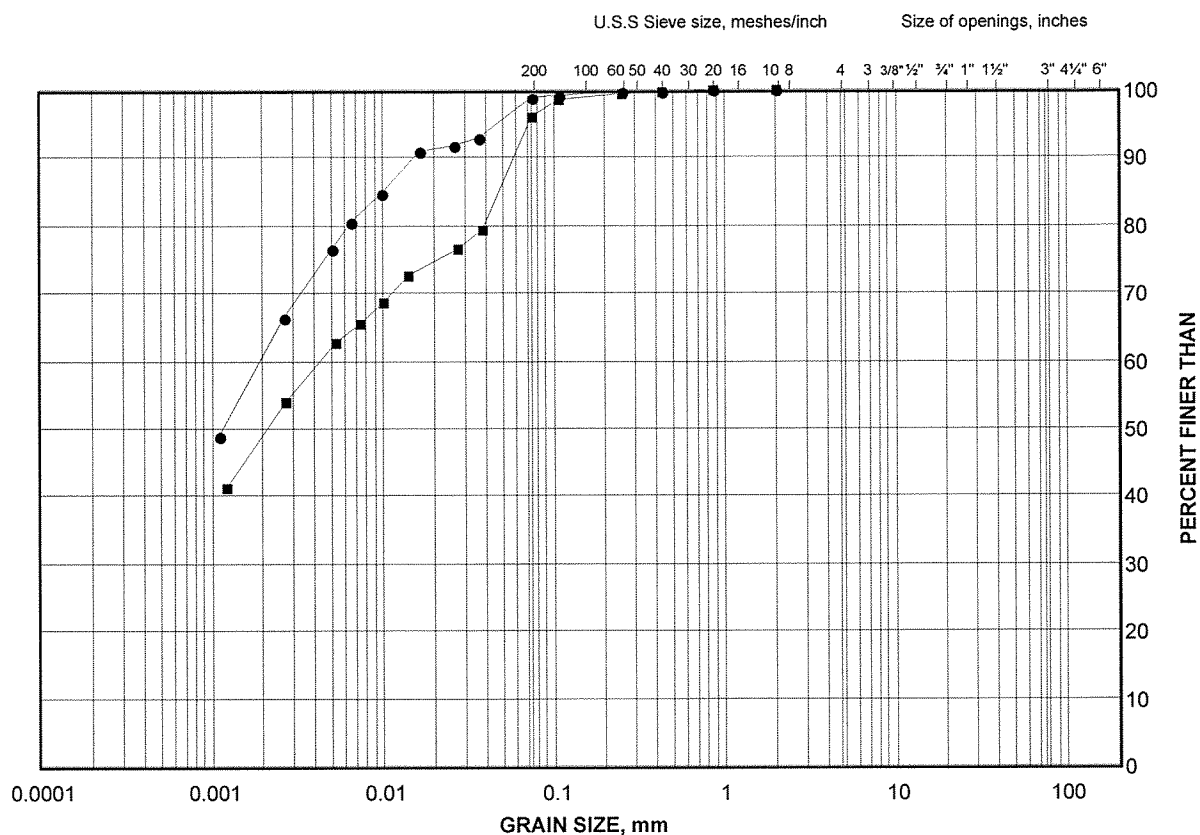
Golder Associates

Date: 18-Oct-06

GRAIN SIZE DISTRIBUTION

Clay

FIGURE A4



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	02-402	10	12.8
■	06-1	13	18.30 - 18.90

Project Number: 021-1155

Checked By: JPD

Golder Associates

Date: 18-Oct-06

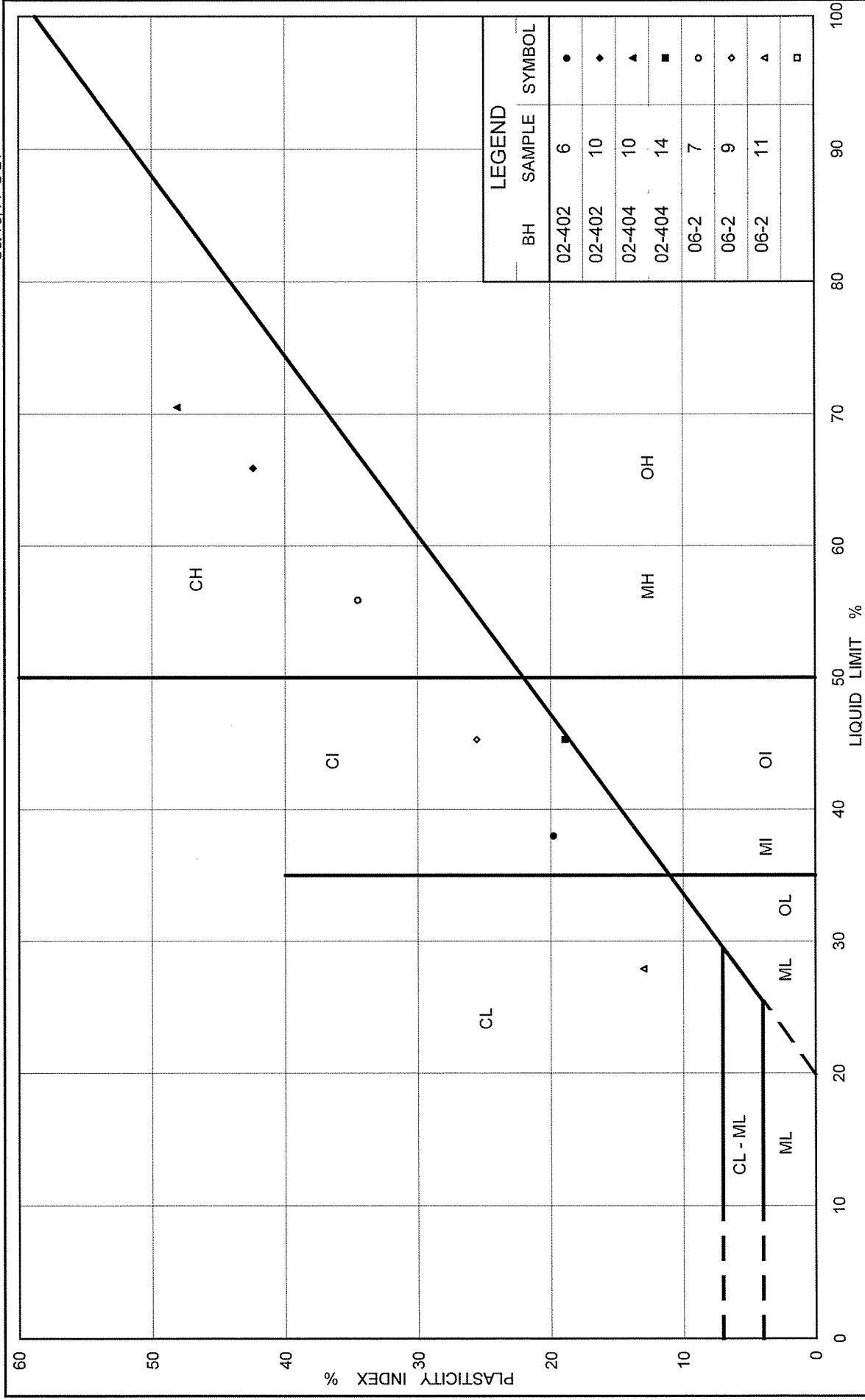


FIG No. A5

PLASTICITY CHART
Clayey Silt to Clay

Ministry of Transportation



Project No. 021-1155

APPENDIX B

SAMPLE NON-STANDARD SPECIAL PROVISIONS

CONCRETE BASE – Item No.

Special Provision

The item Concrete Base shall refer to the Concrete Pad as shown in the Contract drawings.

1.0 Scope

This special provision covers the requirements for the construction of the concrete pad associated with the rigid expanded polystyrene fill.

2.0 References

This special provision refers to the following standards, specifications or publications.

Ontario Provincial Standard Specifications, Construction:

OPSS 904	-	Concrete Structures
OPSS 905	-	Concrete Reinforcement
OPSS 919	-	Formwork and Falsework

Ontario Provincial Standard Specifications, Material:

OPSS 1002	-	Aggregates – Concrete
OPSS 1212	-	Hot-Poured Rubberized Asphalt Joint Sealing Compound
OPSS 1305	-	Moisture Vapour Barriers
OPSS 1306	-	Burlap
OPSS 1308	-	Joint Filler (Concrete)
OPSS 1315	-	White Pigmented Membrane Curing Compounds for Concrete
OPSS 1350	-	Concrete (Materials and Production)
OPSS 1440	-	Steel Reinforcement for Concrete

3.0 Submission and Design Requirements

3.1 Submission of Shop Drawings

At least three weeks before the commencement of work, the Contractor shall submit to the Contract Administrator six copies of shop drawings and method statement that provides full details of materials and construction procedure. Construction of the

concrete pad shall not commence until the submission is returned to the Contractor with the words “permission to construct”.

4.0 Materials

- 4.01 Concrete and concrete materials shall conform to OPSS 1350 with the following exceptions and/or additions.

Class of Concrete	36 MPa at 28 days
Coarse Aggregate	19 mm nominal maximum size
Air Content	7% \pm 1.5%
Maximum Slump	60 mm

- 4.02 Burlap

Burlap shall conform to OPSS 1306.

- 4.03 Moisture Vapour Barrier for Curing

Moisture vapour barrier for curing shall conform to OPSS 1305.

- 4.04 Curing Compound

White pigmented membrane curing compounds for concrete shall conform to OPSS 1315.

- 4.05 Water for Curing

Water for curing shall be free of any impurities, which would adversely affect the concrete.

- 4.06 Joint Materials

Expansion joint filler shall conform to OPSS 1308.

The joint sealing compound shall be hot poured rubberized asphalt conforming to OPSS 1212.

- 4.07 Reinforcement

The steel reinforcement shall conform to the requirements of OPSS 1440 and shall be placed in accordance with OPSS 905.

5.0 Construction

- 5.01 General

The work required includes the construction of the concrete pad as detailed in the Contract Drawings in accordance with the requirements of OPSS 904 unless otherwise noted.

5.02 Preparation Work

5.02.01 Setting Forms

Throughout their entire length, forms shall be set true to line and grade and directly in contact with the polyethylene sheeting over the rigid expanded polystyrene. Forms shall be anchored in such a manner as not to damage the polyethylene or polystyrene.

5.03 Joints

5.03.01 General

Joints shall be of the type and at the locations detailed in the contract.

The saw cutting of the joints shall be performed within sufficient time to prevent cracking.

5.03.02 Transverse Joints – Construction

Transverse construction joints shall be made at the end of each day's run or when interruptions occur in the concreting operation. Transverse construction joints shall be formed at a contraction or expansion joint, except in exceptional cases of plant breakdown or adverse weather conditions. In these exceptional cases, a construction joint may be formed in the mid slab area subject to the provision that the portion of the slab placed, and the portion of the slab to be placed, is not less than 3 m in length.

5.04 Surface Tolerance

The surface of the concrete is to be such that when tested with a 3 m long straightedge placed anywhere, in any direction on the surface, except across the crown or drainage gutters, there shall not be a gap greater than 10 mm between the bottom of the straightedge and the surface of the pavement.

5.05 Traffic

Equipment other than rubber-tire sawing equipment shall not be permitted on the concrete until it has attained a minimum compressive strength of 24 MPa.

A lift of granular no less than 550 mm shall be placed on the concrete pad before traffic is permitted.

Equipment shall be limited in weight and size and restricted in operation to avoid damaging the expanded polystyrene as per the manufacturer's requirement.

5.06 Sampling and/or Testing

Field sampling and testing of concrete shall conform to the requirements OPSS 904 for construction category 1 in Table 2.

5.07 Measurement for Payment

5.07.01 Measurement – Concrete Pad

Measurement is by Plan Quantity as may be revised by Adjusted Plan Quantity of the area of concrete pad placed in squared metres.

5.08 Basis of Payment

5.08.01 Concrete Pad

Payment at the contract price for the above item(s) shall be full compensation for all labour, equipment and material required to do the work.

EXPANDED POLYSTYRENE EMBANKMENT - Item No.

Special Provision

1. Scope

This special provision covers the requirements for the supply and construction of the rigid expanded polystyrene embankment fill, including excavation, foundation preparation, leveling pad, polyethylene sheeting and associated works as shown on the contract drawings.

2. References

This special provision refers to the following standards, specifications or publications.

National Standards of Canada

CAN/CGSB - 51.20 M87 Thermal Insulation, Polystyrene, Boards and Pipe Covering

ASTM

ASTM D6817 Standard Specification for Rigid Cellular Polystyrene Geofoam

ASTM D1621 Test Method for Compressive Properties of Rigid Cellular Plastics

ASTM C203 Test Method for Breaking Load and Flexural Properties of Block Type Thermal Insulation

ASTM C177 Test Method for Steady State Heat Flux Measurements and Thermal Transmission Properties by Means of the Heat Flow Apparatus

ASTM D2842 Test Method for Water Absorption by Rigid Cellular Plastics

ASTM D2863 Test Method for Measuring the Minimum Oxygen Content

ASTM D2126 Test Method for Response of Rigid Cellular Plastics to Thermal and Humid Aging

OPSS - Ontario Provincial Standard Specification

OPSS 212 Borrow

OPSS 501 Compaction

OPSS 517 Dewatering

OPSS 1010 Aggregates – Granular A,B,M, and Selected Subgrade Material

OPSS 1605 Expanded Extruded Polystyrene Pavement Insulation

OPSS 1860 Geotextiles

3. **Subsurface Conditions**

The subsurface conditions at the site are described in the Foundation Investigation Report for this Contract.

4. **Definitions**

For the purpose of this special provision, the following definitions apply:

Rigid Expanded Polystyrene

Molded rigid blocks produced by a process of pre-expansion, aging and forming of a petroleum based raw material.

Rigid Extruded Expanded Polystyrene

Rigid boards made by extrusion of expanded polystyrene beads.

Production Lot

The quantity of rigid polystyrene blocks produced in a continuous period of manufacturing the same grade and thickness of product within the same production day.

Quality Verification Engineer: means an Engineer with a minimum of five (5) years experience related to the design and/or construction of expanded polystyrene systems of similar scope to that in the Contract, or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the Contract. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue of certificate(s) of conformance.

5. **Qualification**

The Contractor shall have on site at the commencement of the work, a representative of the supplier of the rigid expanded polystyrene to advise on recommended construction procedure.

The Contractor shall maintain liaison with the supplier throughout the construction of the embankment for advice and guidance as required. Periodic site visits by the supplier should be coordinated as required.

6. **Submission and Design Requirements**

6.1 **Submission of Shop Drawings**

At least three weeks before the commencement of work, the Contractor shall submit to the Contract Administrator six copies of the shop drawings and method statement signed and sealed by the Quality Verification Engineer that provides full details of materials and construction procedure.

6.2 **Delivery, Storage, Handling and Protection**

The Contractor shall submit the method of delivery, storage, handling and protection from damage by weather, traffic, construction staging and other causes as per the rigid expanded polystyrene manufacturers requirement.

6.3 Construction

The contractor shall submit full details of the following.

- a) The method of foundation excavation and preparation.
- b) Construction of leveling pad.
- c) The method of placement of expanded polystyrene blocks including temporary ballasting and protection of blocks during installation. The shop drawings shall indicate laying pattern and block dimensions on a layer by layer basis.
- d) The method and limits of placement of polyethylene sheeting.
- e) The method of placement of 125 mm reinforced concrete base pad (or equivalent).
- f) The method of placement of subbase material.
- g) The method of placement of side slope cover.

6.4 Quality Verification Engineer

- (1) The Contractor shall submit details of the sequence and method of installation to the Quality Verification Engineer for review. The submittals shall satisfy the specifications and at a minimum include a detailed description of proposed installation procedures. The details shall be submitted at least three weeks prior to the installation of the rigid expanded polystyrene embankments the Contractor shall also submit to the Contract Administrator, for information purposes, details of the sequence and method of installation. The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractor's Quality Verification Engineer.
- (2) The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one week prior to commencement of work under this item. The Certificate shall state that the installation procedures are in conformance with the requirements and specifications of the contract documents. Quality test certificates for each production lot supplied, showing compliance with all requirements of this special provision shall be obtained by the Contractor and submitted to the Contract Administrator prior to installation. ***Upon completion of the Expanded Polystyrene Embankment the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer stating that the Expanded Polystyrene Embankment has been constructed in conformance with the installation procedures and specifications of the contract documents.***

7. Materials

7.1 Granular Leveling Pad

The leveling pad shall consist of a Granular “A” or Granular “B” material with gradation and physical requirements as specified in OPSS 1010.

7.2 Rigid Expanded Polystyrene

7.2.1 General

7.2.1.1 The Contractor shall submit:

1. A general statement as to the type, composition, and method of production of the material.
2. The manufacturer’s name, address, phone number, identification of a contact person and description of experience background in the manufacturing of the rigid expanded polystyrene.
3. Certification of compliance of physical and mechanical properties.
4. An identification of a laboratory accredited by the Standards Council of Canada to conduct the testing of the physical and mechanical properties of the rigid expanded polystyrene.
5. The physical and mechanical properties of the rigid expanded polystyrene including:
 1. Geometry
 2. Nominal Density
 3. Compressive Strength
 4. Flexural Strength
 5. Thermal Resistance
 6. Dimensional Stability
 7. Oxygen Index
 8. Flammability
 9. Water Absorption
6. Aging and durability characteristics of the polystyrene including the chemical, biological and ultra-violet degradation resistance of the rigid polystyrene.
7. A sample of the expanded polystyrene material to the Quality Verification Engineer for review.
8. To the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of one week prior to commencement of work under this item. The Certificate shall state that the expanded polystyrene material is in conformance with the requirements and specifications of the contract documents.

7.2.1.2 Each block of the same production lot shall be stamped with the same production code showing plant identification, type and date of production. The polystyrene shall be free from defects affecting serviceability.

7.2.2 Detail Requirements

Requirements shall be as shown in Table 1 and as described below.

TABLE 1 – MATERIAL PROPERTIES

PROPERTY	UNIT	REQUIREMENTS	TEST PROCEDURE
Geometry	mm	1200 x 600 x 300	
- Linear		with tolerances $\pm 1\%$	
- Flatness		10 mm in 3 m $\pm 0.5\%$	
- Squareness		-3, +5	
- Thickness			
Compressive Strength at 5% strain	kPa (min)	115	ASTM D1621 (Procedure A)
Flexural Strength	kPa (min)	240	ASTM C203
Dimensional Stability	% linear change (max)	1.5	ASTM D2126
Thermal Resistance	m ² .°C/W (min for 25 mm thickness)	0.7	ASTM C177 or C518
Flammability	Limiting Oxygen Index (min)	24	ASTM D2863
Water Absorption	% by Volume (max)	4	ASTM D2842

7.2.2.1 Geometry

The expanded polystyrene shall be supplied in the form of rectangular parallel blocks of minimum acceptable dimensions of 1200 mm x 600 mm x 300 mm.

The maximum deviation from the specified linear dimensions shall be $\pm 1\%$. The flatness of the block faces shall be within ± 10 mm of a line formed by a 3 m straight edge.

The maximum difference in corner to corner dimensions (squareness) shall be 0.5%. The thickness shall be within -3 to +5 mm.

7.2.2.2 Compressive Strength

The minimum compressive strength, measured in accordance with ASTM D1621, Procedure A, shall be 115 kPa at a strain of not more than 5%. The maximum permissible permanent stress level should not exceed 30% of the compressive strength of the material at 5% strain.

7.2.2.3 Flexural Strength

The minimum flexural strength of the polystyrene shall be 240 kPa. The flexural strength shall be determined in accordance to ASTM C203, method 1, Procedure B.2.7.4 Dimensional Stability.

7.2.2.4 Dimensional Stability

Dimensional Stability shall be determined in accordance with ASTM D2126, Procedure G. A tolerance of 1.5% shall be satisfied.

7.2.2.5 Thermal Resistance

The thermal resistance shall be $0.7 \text{ m}^2\cdot\text{C}/\text{W}$ for a 25 mm thickness using the following equation and using the average value from three specimens:

$$R_{25\text{mm}} = \frac{R_{\text{measured}}}{\text{thickness (mm)}} \times 25$$

The thermal resistance shall be measured in accordance with ASTM C177 or C518.

7.2.2.6 Flammability

The expanded polystyrene shall be classified as to surface burning characteristics in accordance with CAN/ULC - 51022 having a flame spread rating less than 500. The expanded polystyrene shall have a minimum limiting oxygen index measured in accordance with ASTM D2863

7.2.2.7 Water Absorption

The water absorption as measured by ASTM D2842 shall be limited to 4% by volume.

7.2.2.8 Chemical Resistance

The expanded polystyrene shall be resistant to common inorganic acids and alkalis. A table identifying the chemical resistance as either resistant, limited or not resistant shall be submitted.

7.2.2.9 Biological Resistance

The expanded polystyrene shall be resistant to biological degradation caused by organisms or enzymes.

7.2.2.10 Environmental

The expanded polystyrene shall be inert, non-nutritive and highly stable and shall not produce undesirable gases or leachate.

8.0 Delivery, Storage and Handling

The product shall be suitably marked to identify its type, number and the manufacturer's name or trademark.

The Contractor shall protect the expanded polystyrene from exposure to sunlight to avoid ultraviolet degradation as per manufacturer's recommendation.

Protection of materials and works from damage by weather, traffic, construction staging, fire or vandalism and other causes shall be the responsibility of the Contractor.

9.0 Construction

9.1 Foundation Excavation

Foundation excavation shall be carried out to the design elevations shown on the drawings. Any softened, loosened or deleterious materials at the foundation footing elevation shall be subexcavated and replaced with Granular 'A' or Granular 'B' material.

9.2 Leveling Pad

Place, level and compact a layer of Granular 'A' or Granular 'B' material in accordance with OPSS 501 to within ± 30 mm of the design elevation. The leveling pad shall not deviate by more than 10 mm at any place on a 3 m straight edge over the limits of the bottom course of blocks. The leveling pad shall not be placed on frozen ground.

9.3 Installation of Blocks

- (1) The individually marked blocks shall be placed on the prepared leveling pad. The top surface of the first layer of blocks is to be set plane and level. Local trimming of the blocks may be necessary.
- (2) Subsequent successive layers shall be oriented with the long axis of blocks positioned at 90° to the previous layer in order to avoid continuous joints. Block joints shall be offset and staggered between layers.

A continuous check shall be kept to ensure the evenness of the blocks is satisfactory in each layer. Blocks shall be laid with joints with maximum opening of 10 mm between blocks. Differences in heights between adjacent blocks in the same layer should not exceed 5 mm.

- (3) Sloping end adjustments at the abutments shall be accomplished by leveling terraces in the subsoil in accordance with the block thickness.
- (4) Temporary ballast shall be provided as necessary to prevent movement of expanded polystyrene both in storage and as placed due to windy conditions. Timber fasteners or equivalent shall be used as necessary.
- (5) The expanded polystyrene embankment shall be protected from accidental ignition due to welding, smoking, grinding or cutting tools, etc. The Contractor shall take all necessary precautions to prevent ignition of the expanded polystyrene.
- (6) The expanded polystyrene shall be protected from organic solvents and other aggressive, harmful chemicals during construction. The proposed method of protection during construction shall be submitted to the Contractor's Quality Verification Engineer for review and to the Contract Administrator for information purposes.
- (7) Exposed blocks shall be covered immediately to avoid possible burrowing by animals.

- (8) Individually marked blocks shall be fabricated and placed to ensure the top surface matches the elevation and crossfall shown on the drawings.
- (9) The top surface and side surfaces of the expanded polystyrene shall be covered with 10 mil polyethylene sheeting extending onto adjacent work at the longitudinal ends of the embankment. All joints shall be lapped a minimum of 300 mm to provide a fully sealed enclosure.
- (10) The contractor shall install the concrete base pad as detailed elsewhere in the contract.
- (11) The side slope of the rigid expanded polystyrene embankment shall be covered with either ultra-lightweight fill or earth fill material as detailed elsewhere in this contract.

10. Equipment

All cutting of polystyrene materials shall be by electric equipment or by hand.

Heavy equipment shall be limited in weight and size and restricted in operation to avoid damaging the expanded polystyrene as per the manufacturer's requirement.

11. Quality Assurance

11.1 Quality Assurance

Quality test certificates for each production lot supplied, showing compliance with all requirements of this special provision shall be obtained by the Contractor and submitted to the Contract Administrator prior to installation.

11.2 Sampling and Testing

11.2.1 General

The Contract Administrator may undertake an independent testing program of the expanded polystyrene. Sampling and testing will be carried out in conformance with the relevant test procedure. The physical and thermal property testing identified in Table 1 will be conducted. The testing shall be conducted by a recognized testing laboratory accredited by the Standards Council of Canada.

11.1.2 Sampling Frequency

Sufficient sample material shall be obtained from blocks randomly selected by the Contract Administrator from each production lot as soon as the material arrives on site. As a minimum, three blocks shall be tested.

11.1.3 Acceptance/Rejection

Failure of any one of the sample blocks to comply with any requirements of this special provision shall be cause for rejection of the production lot from which it was taken. Replacement of the blocks shall be at the Contractor's expense.

12. Measurement for Payment

12.1 Actual Measurement

Measurement will be by volume in cubic metres measured in its original position and based on cross-sections.

13. Payment

13.1 Basis of Payment

The Concrete Base pad and granular leveling pad shall be paid for with the appropriate tender items as detailed elsewhere in the contract.

Payment at the contract price for the above tender item shall be full compensation for all labour, materials and equipment to do the work as described above and no extra payments will be made.

ULTRA LIGHTWEIGHT MATERIAL

Non Standard Special Provision

SCOPE

This non standard special provision covers the requirements for the supply and placement of ultra lightweight blast furnace slag.

DEFINITIONS

Quality Verification Engineer: means an Engineer with a minimum of five (5) years experience related to embankment materials and construction, or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the Contract. The Quality Verification Engineer shall be retained by the Contractor to certify that the work is in general conformance with the contract documents and issue of certificate(s) of conformance.

SUBMISSION AND DESIGN REQUIREMENTS

The Contractor shall submit to the Contract Administrator Certificates of Conformance sealed and signed by the Quality Verification Engineer as follows:

1. Prior to the placement of the ultra lightweight fill material on the Contract, the Contractor shall submit to the Contract Administrator a Certificate of Conformance stating that the material satisfies the material properties specified in Table 1. The material properties shall be determined using the test procedure specified in Table 1.
2. Following embankment construction, the Contractor shall submit to the Contract Administrator a Certificate of Conformance stating that the material satisfies the requirements of this specification and that the work has been carried out in general conformance with the contract documents and specifications.

In addition, the Contractor shall submit to the Contract Administrator, for information only, all Quality Control Test Results.

MATERIAL

The Ultra Lightweight Blast Furnace Slag shall satisfy the physical, mechanical and chemical property requirements specified in Table 1:

Table 1: Material Properties and Construction Requirements

Property	Requirement	Test Method
Angle of Internal Friction	> 35 °	ASTM 2850-95
Hydraulic Conductivity	> 8 E-03 cm/s	ASTM 5856-95, Method A
Chemical Composition	The material shall meet the Leachate Criteria Established Under Ontario Regulation 347.	
In-Situ Wet Unit Weight, maximum when placed and compacted in accordance with the requirements of this Special Provision	< 12.5 kN/m ³	ASTM D2922

The Contractor shall retain a laboratory that has been inspected and accepted by the MTO under the "Soil and Rock - High Complexity Testing" to undertake the testing of the material properties. Laboratory testing shall be signed and sealed by an Engineer, licensed to practice in the Province of Ontario.

CONSTRUCTION

The Contractor is advised that the ultra lightweight blast furnace slag is susceptible to crushing if overcompacted and that careful construction supervision is required.

The Contractor shall place the ultra lightweight fill material and shall achieve compaction without crushing the material since crushing increases its unit weight.

The Contractor shall place the ultra lightweight fill material without exceeding the specified in-situ unit weight and maintaining crushing of the material below 5%.

To prevent overcrushing and overcompaction, the ultra lightweight fill shall be placed as follows:

1. For embankments, the ultra lightweight fill shall be placed in lifts of 300 mm and compacted by three (3) passes using single drum vibratory equipment such as a Bomag 142 or equivalent.
2. For backfill to structures, the ultra lightweight fill shall be placed in lifts of 300 mm and compacted with 8 passes of manually guided tamper such as a Bomag BPR 30/38 D or equivalent.
3. The Contractor shall place and spread the loose lifts using a rubber tire front-end loader such as a Caterpillar 980 F or equivalent.

Compaction equipment technical details are provided in Table 2.

Table 2 – Compaction Equipment Technical Details

	Bomag 142 D	Bomag BPR 30/38 D
Weights		
▪ Operating weight (kg)	4690±	175±
▪ Mass per square metre of base plate (kg/m ²)	N/A	1439
Dimensions		
▪ Drum width (mm)	1426±	N/A
▪ Drum diameter (mm)	1058±	N/A
▪ Width of Base Plate (mm)	N/A	380
▪ Length of Base Plate (mm)	N/A	730
Drive		
▪ Performance DIN 6271 IFN (kW)	37±	3.7
▪ Performance SAE (Kw)	39.5	N/A
▪ Speed (rpm)	2300	3600
Vibratory System		
▪ Frequency (Hz)	32±	68±
▪ Amplitude (mm)	1.24±	N/A
▪ Centrifugal force (Kn)	66±	30±

QUALITY CONTROL

General

Quality Control (QC) testing shall be carried out by the Contractor for purposes of ensuring that the ultra lightweight fill material is placed and compacted to the requirements specified in the Contract. Field density and field moisture determination shall be made in accordance with ASTM D2922 and ASTM D3017.

Acceptability of compaction shall be based on achieving the target in situ unit weight.

Control Strip

Under the Supervision of the Quality Verification Engineer, the Contractor shall build a control strip to verify that the placement and compaction procedure will achieve the requirements of this Special Provision without evidence of crushing and without exceeding the specified maximum in-situ unit weight of 12.5 kN/m³.

Prior to incorporating any of the material into the work the Contractor shall build a minimum trial area of 400 m² in area consisting of two equal lifts of 300 mm thickness. The Contractor shall give the Contract Administrator written notice of the construction of the control strip 48 hours prior to commencement of

this work.

Material placed in the control strip shall have the moisture content that will yield the specified in-situ unit weight. For the Control strip determination, the nuclear gauge method will not be considered an acceptable method of determining the in-situ moisture content of the ultra lightweight material. Moisture content shall be determined by the oven dry method on selected compacted embankment material samples in accordance with ASTM D2216.

After the trial area is complete, samples for moisture content and in-situ unit weight determination testing shall be as per ASTM D2922.

In addition, Gradation as per ASTM D422-63 before and after compaction effort shall be performed to determine that crushing is kept within 5%.

All test results will be used to determine compliance with the specification. Any proposed changes to the specified compaction method shall be reviewed and approved by the Contract Administrator prior to implementation. The requirements of the control strip must be satisfied as part of the acceptance criteria of any proposed change to the specified compaction method of this Special Provision.

MEASUREMENT OF PAYMENT

The unit measurement will be cubic metres for the ultra lightweight fill material placed in situ as per the requirements of the contract.

BASIS OF PAYMENT

Payment at the contract price for the above tender item shall be full compensation for all labour equipment and materials required to do the work.

AMENDMENT TO OPSS 539, NOVEMBER 2003

Special Provision No. 105S19M

November 2006

OPSS 539, November 2003, Construction Specification for Protection Systems is deleted in its entirety and replaced with the following:

CONSTRUCTION SPECIFICATION FOR PROTECTION SYSTEMS

539.01 SCOPE

This specification covers the requirements for the design, construction, maintenance, monitoring and removal of a protection system made necessary by excavation or other work.

539.02 REFERENCES

This specification refers to the following standards, publications or specifications:

Ontario Provincial Standard Specifications, Construction:

OPSS 180 Management and Disposal of Excess Material

OPSS 903 Piling

OPSS 904 Concrete Structures

OPSS 906 Structural Steel

Ontario Provincial Standard Specifications, Material:

OPSS 1350 Concrete Materials and Production

OPSS 1601 Wood Material, Preservative Treatment and Shop Fabrication

Ontario Ministry of Labour:

Occupational Health and Safety Act, R.S.O. 1990, c.O.1, as amended

American Association of State Highways Transportation Officials:

AASHTO Guide Design Specification for Bridge Temporary Works, 1995

Canadian Standards Association

CAN/CSA-S6-00, Canadian Highway Bridge Design Code

539.03 DEFINITIONS

For the purpose of this specification, the following definitions apply.

Anchorage System: means a system consisting of tendons installed in predrilled holes in soil or rock and encapsulated in grout or concrete that derives its load carrying capacity in bond between the grout/concrete body and the surrounding soil or rock; or tie back to deadmen.

Bracing: means the system of walers, struts, anchorages and like members that connect frames, shores or panels of a sheathing system to resist external pressures and to provide stability against lateral movement.

Cofferdam: means a water-tight enclosure.

Design Engineer: means the Engineer retained by the Contractor who produces the original design and working drawings.

Design Checking Engineer: means the Engineer retained by the Contractor who checks the original design and working drawings.

Dredge Line: means the exposed lower limit of the Protection System.

Erector: means a person that undertakes the construction of a Protection System.

Protection System: means the construction necessary to mechanically support existing or proposed work such that its function will not be affected, or, construction necessary to support work, such as open excavations, during actual construction operations for safety and convenience.

Quality Verification Engineer: means the Engineer, retained by the Contractor, qualified to determine that the work is in general conformance with the Contract Documents and issue Certificate(s) of Conformance.

Raker: means a structural member inclined to the front of the shoring wall providing lateral support.

Shoring Wall: means a structural wall consisting of wood, steel, concrete or combination of these materials that supports earth or rock and any structure, materials, utilities or other facility contained in or on the supported earth or rock mass.

Stamped: means drawings or details that have been reviewed and stamped "In General Conformance with Contract Documents". The stamp shall include the date and signature of the Quality Verification Engineer.

Top of Shoring Wall: means the upper limit of the Protection System.

539.04 SUBMISSION AND DESIGN REQUIREMENTS

539.04.01 Submissions

539.04.01.01 Working Drawings

Three (3) copies of stamped working drawings shall be submitted to the Contract Administrator for information purposes at least one(1) week before commencement of construction of the protection system.

All submissions shall bear the seal and signature of the Design Engineer and Design Checking Engineer.

For contracts where another authority, such as a railway or navigable waters, is affected the Contractor shall submit working drawings to each authority (number of sets of drawings to be determined by the authority). The requirements of each authority shall be satisfied before commencement of protection system installation.

All design shall be carried out in accordance with AREMA. Monitoring of the performance of the protection system during construction shall be carried out to the satisfaction of the railway authority.

The Contractor shall have a copy of the stamped working drawings at the site during protection system construction.

For protection systems that are not specified in the Contract Documents, the Contractor shall submit to the Owner working drawings of these systems at least three weeks prior to the commencement of any construction.

539.04.01.02 Working Drawings/Details Requirements

539.04.01.02.01 Information To Be Shown on Working Drawings/Details

- a) Plans, Elevations and Details
 - i. Location of protection system and station limits.
 - ii. Plan and elevation of shoring showing the extent of the protection system.
 - iii. Details of the shoring system including cross-sections.
 - iv. Details of internal bracing.
- b) Design Criteria
 - i. Pressure diagrams including values of horizontal and vertical loads, dead load and live load surcharge.
 - ii. Design assumptions and parameters.
 - iii. Anchor bond stresses.
 - iv. Pile design.
 - v. Anchor System stressing schedule specifying working loads, stressing loads and lock in loads.
 - vi. Details of preload where required.
 - vii. For protection systems not specified in the Contract, the performance level shall be designated.
- c) Materials
 - i. Grade of structural steel and grade and species of structural wood.
 - ii. Concrete strengths.
 - iii. Grout strengths.
 - iv. Details of protection from rain and frost action.
 - v. Wood lagging and size.
 - vi. Mill certificates or test reports from an independent organization certified by the Standards Council of Canada certifying that the steel meets the requirements of the grade specified.
 - vii. Details of patented accessories, including load test data.
- d) Installation Procedure
 - i. Installation sequence and procedure including but not limited to the installation of piling, lagging, anchor systems and rakers.
- e) Monitoring Method
 - i. The proposed method of monitoring the performance of the Protection System during installation and use. The method of monitoring shall be consistent with the requirements specified in Section 539.07 of this special provision.
- f) Removal of Protection System
 - i. The details of the procedures associated with the removal of the protection system indicating: method, sequence of work, and removal limits.

539.04.01.03**Qualifications**

Design Engineer: The Design Engineer shall have demonstrated expertise for the work. The Design Engineer shall have a minimum of five (5) years experience in designing protection systems of similar nature and scope to the required work. One person cannot perform both the Design Engineer and Design Checking Engineer roles for a protection system.

Design Checking Engineer: The Design Checking Engineer shall have demonstrated expertise for the work. The Design Checking Engineer shall have a minimum of five (5) years experience in designing protection systems of similar nature and scope to the required work.

Erector: All supervisory personnel involved in the work performed under this specification shall be experienced in the method of construction of protection systems. Such experience shall have been obtained within the preceding five years on projects of similar nature and scope to the required work.

Quality Verification Engineer: The Quality Verification Engineer shall have a minimum of five(5) years experience in the design of comparable protection systems, or alternatively with demonstrated expertise through providing satisfactory quality verification services for a minimum of two (2) projects in which the work was of similar scope to that in the Contract. The Quality Verification Engineer shall be retained by the Contractor to determine if the work is in general conformance with the Contract Documents and to issue Certificate(s) of Conformance.

539.04.01.04**Certificates of Conformance****539.04.01.04.01****Excavation Depths Less Than or Equal to Three (3) metres**

For protection systems to facilitate excavation depths less than or equal to three (3) metres and provided that surcharge loading due to vehicular traffic, construction equipment and materials or other is beyond a horizontal distance defined by a 1H:2V line projected from the dredge line at the face of the protection system to the roadway surface, the Contractor shall submit, to the Contract Administrator, a Certificate of Conformance sealed and signed by the Quality Verification Engineer following the installation of Protection System to the Dredge Line.

Should traffic be within a horizontal distance defined by a 1H:2V line projected from the dredge line at the face of the protection system to the roadway surface, the certificate of conformance requirements as specified in clause 539.04.01.04.02 shall apply

Upon completion of the operation of the protection system and removal of the protection system, the Contractor shall submit to the Contract Administrator a final Certificate of Conformance sealed and signed by the Quality Verification Engineer. The Certificate of Conformance shall state that the protection system was monitored and subsequently removed, and it performed in general conformance with the stamped working drawings and contract documents.

539.04.01.04.02**Excavation Depths Exceeding Three (3) metres**

For protection systems to facilitate excavation depths that exceed three (3) metres, the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer upon completion of each of the following operations, prior to commencement of each subsequent operation:

- a) Layout and Extent of Protection System
- b) Piling
- c) Installation of Protection System including excavation to Dredge Line
- d) Removal and management (in accordance with OPSS 180 and as specified in the Contract).

The Certificates of Conformance shall state that the materials and work have been supplied and installed in general conformance with the working drawings.

Upon completion of the operation of the protection system and removal of the system, the Contractor shall submit to the Contract Administrator a final Certificate of Conformance sealed and signed by the Quality Verification Engineer. The Certificate shall state that the protection system was monitored and removed, and it performed in general conformance with the stamped working drawings and contract documents.

539.04.01.05 Amendments to Protection Systems

Work shall not proceed on amendments to the protection system until the Contractor has received sealed and signed approval to proceed from the original Design Engineer and Design Checking Engineer and has submitted a copy of the approval to the Contract Administrator.

Amendments to the Protection System shall be submitted to the Contract Administrator on revised Working Drawings/Details bearing the seal and signature of the original Design Engineer and Design Checking Engineer.

539.04.01.06 Preconstruction Survey

Prior to commencing the work, the Contractor shall submit to the Contract Administrator, a condition survey of property and structures that may be affected by the work. The survey shall include, but not be limited to, the locations and conditions of adjacent properties, buildings, underground structures, utility services and structures such as walls abutting the site within a horizontal distance of $2H_w$ from the face of the protection system, where H_w is the height of the wall from the ground surface to the dredge line

539.04.02 Design

539.04.02.01 General

The protection system shall be designed for the performance level specified in the Contract Documents.

Protection systems that are not specified in the Contract Documents shall be assigned an appropriate performance level for design by the Design Engineer. The Contract Administrator shall review the performance level selected at the time of submission of the specified working drawings.

The Contractor shall be responsible for the complete detailed design of the protection system needed to fulfill the requirements specified in the contract drawings.

The geotechnical/foundation portion of the design shall be based on a method published in AASHTO Guide Design Specification for Bridge Temporary Works and in general conformance with the CAN/CSA-S6-00 Canadian Highway Bridge Design Code (CHBDC). Design methods not meeting this design specification may be used on a particular contract only if prequalified by the Owner.

A protection system shall be designed to provide protection for excavations as required by the Occupational Health and Safety Act, at the locations specified in the Contract, and at any other location where the stability, safety or function of an existing structure and/or utility may be impaired by construction work.

The temporary slope geometry used to determine requirements of the protection system shall be in accordance with the Occupational Health and Safety Act.

Performance levels for protection systems are as follows:

Performance Level	Maximum Angular Distortion	Maximum Horizontal Displacement
1a	1:1000	5 mm
1b	1:1000	10 mm
2	1:200	25 mm
3	1:100	50 mm

Where:

$$\text{Angular Distortion} = \pm \Delta / H$$

Δ = Horizontal displacement (mm) at height H

H = Height (mm) above dredge line to point of measurement or height above the nearest system restraining support.

When performance level 1a is specified the bracing system shall be preloaded.

Where the bracing systems are preloaded, the effects of the preload shall not cause damage to adjacent facilities.

Protection systems with a face within a horizontal distance of 1/3 H of any part of a structure foundation shall be designed for performance level 1a.

539.04.02.02 Design Assumptions

The design assumptions shall accurately represent the subsurface conditions prevalent at the site, and shall be specific to the type of protection system used. The design shall address the subsurface conditions at the project site reported in the Foundation Investigation Report described in the Contract Documents.

539.04.02.03 Vertical and Horizontal Loadings

Vertical and horizontal design loadings used shall represent existing conditions and accepted design practice. Future loadings that are known and may affect the protection system during its useful life shall be considered.

539.05 MATERIALS

539.05.01 Wood

Wood shall be according to OPSS 1601, shall be of the size, grade and species shown on the working drawings and shall be in sound condition, free from defects which will impair its strength. Wood lagging does not have to be grade-stamped.

539.05.02 Structural Steel

539.05.02.01 Mill Certificates

The Contractor shall submit to the Contract Administrator at the time of delivery one copy of the mill certificate, indicating that the steel meets the requirements for the appropriate standards for H-piles, tube piles, casings and sheet piles.

Where mill test certificates originate from a mill outside Canada or the United States of America the Contractor shall have the information on the mill certificate verified by testing by a Canadian laboratory. The laboratory shall be accredited by a Canadian National Accreditation Body to comply with the requirements of ISO/IEC Guide 25 for the specific tests or type of tests required by the material standard specified on the mill test certificate. The mill test certificates shall be stamped with the name of the Canadian testing laboratory and appropriate wording stating that the material conforms to the specified material requirements. The stamp shall include the appropriate material specification number, the date and the signature of an authorized officer of the Canadian testing laboratory.

For structural steel that will not be incorporated in the final design of a structure and when a mill certificate is not available, the specified minimum yield strength shall be according to clause 14.6, Material Strengths of CAN/CSA-S6-00.

539.05.03 Proprietary Shoring and Patented Accessories

Where proprietary shoring or patented accessories are to be used, the Contractor shall follow the manufacturers' recommendations for load carrying capacity. The recommended load carrying capacities shall be supported by test results from an accredited testing laboratory approved by the Owner.

539.05.04 Concrete

Concrete shall be according to OPSS 1350.

539.05.05 Other Materials

The Design Engineer may consider other suitable materials when sufficient information is available to quantify the allowable design loads or when the manufacturer's recommendations as to load carrying capacities are supported by test results from an independent organization accredited by the Standards Council of Canada.

539.07 CONSTRUCTION

539.07.01 General

The Contractor shall be responsible for the design, materials, construction, maintenance, monitoring, and removal of a temporary protection system.

Protection systems shall be built according to the specifications and the stamped working drawings.

Concrete construction shall be according to OPSS 904.

Structural steel shall be according to OPSS 906.

Piling shall be according to OPSS 903.

Prestressed anchors shall be supplied, installed and stressed according to the Contract Documents.

The protection system shall be protected from the detrimental effects of rain and frost action.

Material used in the protection system shall remain the property of the Contractor unless otherwise specified.

Loss of soil from behind the shoring shall be prevented during and following the installation of the lagging.

The Contractor shall carry out dewatering as required to facilitate the installation of the protection system. Concrete shall be placed in the dry unless otherwise specified in the Contract. Where cofferdams are used they shall be sealed sufficiently to permit concrete to be placed in the dry. When concrete cannot be placed in the dry, tremie techniques shall be employed according to OPSS 904.

539.07.02 Removal of Protection Systems

Protection systems shall be removed from the right-of-way unless otherwise specified in the Contract that the protection system may be left in place.

The Contractor shall obtain approval from the Ministry of the Environment and other approving Authorities when all or any portion of the protection system is to be left in place.

Where piles are left in place the top shall be removed to at least 1.2 m below the finished grade or ground level or at least 0.6 m below the streambed.

The method and sequence of removal shall be such that there will be no damage to new work, existing work and the facility being protected.

Unless otherwise specified, the area remaining disturbed after removal of the protection system shall be restored to as close to its original condition as possible.

539.07.03 Quality Control

539.07.03.01 General

The Contractor shall complete a preconstruction condition survey and monitor the protection system installation as specified herein, or as shown on the Working Drawings.

539.07.03.02 Inspection of Welds

The Contractor shall be responsible for visual inspection of all welds. Any required testing of welds shall be as specified by the Design Engineer of the protection system.

539.07.03.03 Monitoring

539.07.03.03.01 General

Monitoring shall be conducted by a Registered Ontario Land Surveyor or an Engineer according to the program submitted with the construction drawings/details.

The minimum requirements for monitoring shall include the survey measurements of scaled targets attached to the shoring wall at the elevations specified. The scaled targets shall be placed at a maximum spacing of 6 metres with targets placed at the extreme ends and the targets distributed between the outer limits. The survey targets shall be monitored for horizontal displacement from the vertical at the frequency specified.

All test results, observations and records, including the construction survey taken during construction and operation of the protection system shall be available on the site for review by the Contract Administrator.

If movement of the protection system is more rapid than is expected, or if movement approaches the allowable limit, the Contract Administrator shall be notified immediately and suitable measures shall be taken to ensure stability of the protection system and to ensure movement does not exceed the performance level specified.

539.07.03.03.02 Excavation Depths Less Than or Equal to Three (3) metres

The protection systems shall be monitored during construction. Readings shall be taken during installation of the protection system at the top of the protection system at each construction stage during the installation of the protection system. After installation the above readings shall be taken bi-weekly.

539.07.03.03.03 Excavation Depths Exceeding Three (3) metres

The protection systems shall be monitored during construction. Readings shall be taken during installation of the protection system at the top, at each restraint point, at the dredge line and halfway between the restraint points at each construction stage during the installation of the protection system. After installation the above readings shall be taken weekly.

539.10 BASIS OF PAYMENT

539.10.01 Protection System – Item

Payment at the contract price for the above tender item shall be full compensation for all labour, equipment and material to do the work.

For protection systems not specified in the Contract Documents, the cost shall be included in the protection system tender item, if available, and shall be full compensation for all labour, equipment and material required to carry out the work, including subsequent removal of the protection system and any necessary restoration work.

If the protection system tender item is not included in the Contract Documents, the cost shall be included in the item or items directly associated with the protection system, and shall be full compensation for all labour, equipment and material required to carry out the work, including subsequent removal of the protection system and any necessary restoration work.

WARRANT: All contracts.

APPENDIX C
REFERENCE INFORMATION

DEPARTMENT OF HIGHWAYS - ONTARIO

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

MATERIALS & TESTING DIVISION

JOB 66-F-17

LOCATION Queensway and CNR Crossing-Ottawa-142/75, O/S 90' Lt.

ORIGINATED BY P.L.W.

W.P. 108-65

BORING DATE March 14, 1966

COMPILED BY P.L.W.

DATUM Geodetic

BOREHOLE TYPE Wash-boring & Diamond Drill

CHECKED BY

