



May 10, 2013

## PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

REPLACEMENT OF ONR OVERHEAD, SITE 47-037  
HIGHWAY 11, EARLTON, ONTARIO  
MINISTRY OF TRANSPORTATION, ONTARIO  
GWP 5265-01-00

**Submitted to:**  
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REPORT

**GEOCRES NO.:** 31M-99

**Report Number:** 11-1191-0025-1

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# PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT  
REPLACEMENT OF ONR OVERHEAD BRIDGE, SITE 47-037  
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MINISTRY OF TRANSPORTATION, ONTARIO  
GWP 5265-01-00





## **1.0 INTRODUCTION**

Golder Associates Ltd. (Golder) has been retained by URS Canada Inc. (URS) on behalf of the Ministry of Transportation, Ontario (MTO) to provide preliminary foundation engineering services for the replacement of the Ontario Northland Railway (ONR) Overhead Bridge (Site 47-037), located on Highway 11, Earlton, Ontario. This report presents the results of the preliminary foundation investigation carried out for the replacement of the ONR Overhead Bridge.

The Terms of Reference and the Scope of Work for the foundation engineering services are outlined in MTO's Request for Proposal dated June 2011. The work has been carried out in accordance with Golder's Supplementary Specialty Quality Control Plan for foundations engineering services for this project dated December 2011. The Base Plan (General Arrangement Drawing) showing the alignment of the ONR Overhead was provided to Golder by URS in July, 2012.

## **2.0 SITE DESCRIPTION**

The Highway 11 ONR Overhead Bridge is situated between Highway 571 and 10<sup>th</sup> Street in Earlton, Ontario. The surrounding land is generally flat with nearby residential development and farmland.

The existing structure, constructed in 1961, consists of an approximately 135 m long and 11 m wide, nine-span bridge supported by deep foundations that are anticipated to be founded on or in proximity to bedrock based on available information. The approach embankments are approximately 9 m high with earth fill stability berms extending along both sides of the embankments. The stability berms consist of an upper berm approximately 6 m high and 17 m wide decreasing in width away from the bridge, and a lower berm approximately 3 m high and 10 m wide, also decreasing in width away from the bridge. The existing approach embankments are at approximately Elevation 257 m at the south and north abutments and the bridge crosses both the ONR tracks at approximately Elevation 249 m and Government Road at approximately Elevation 250 m.

The orientation (i.e. north, south, east, west) stated in the text of the report is typically referenced to project north and/or up-chainage along the existing Highway 11 alignment. For purposes of this report, Highway 11 is oriented north-south.

## **3.0 INVESTIGATION PROCEDURES**

The fieldwork for this foundation investigation was carried out from July 17 to 24, 2012, at which time four boreholes (Boreholes ONR-1 to ONR-4) were advanced at the site. All boreholes were advanced using a CME 850 track-mounted drill rig supplied and operated by Landcore Drilling Inc. of Sudbury, Ontario. The borehole locations are shown on Drawing 1.

The boreholes were advanced using 108 mm inner diameter hollow-stem augers and Borehole ONR-1 was also advanced with NW casing and wash boring techniques to penetrate through a 200 mm size cobble (auger refusal) at 3.9 m depth. In Borehole ONR-4, a Dynamic Cone Penetration Test (DCPT) was advanced from the bottom of the borehole from a depth of 23.5 m to refusal at a depth of 25.0 m. Soil samples were obtained at intervals of depth of about 0.75 m to 1.5 m, using a 50 mm outer diameter (O.D.) split-spoon sampler operated by an automatic hammer on the drill rig, in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586). Selected samples of the cohesive soils were obtained using 76 mm O.D. thin-walled 'Shelby' tubes (ASTM D1587, Thin-Walled Tube Sampling) for relatively undisturbed samples. Field vane shear tests





were carried out in cohesive soils for determination of undrained shear strengths (ASTM D2573, Field Vane Strength Shear Test) using an MTO Standard 'N' size vane. All boreholes were backfilled upon completion in accordance with Ontario Regulation 903 Wells (as amended).

The groundwater conditions were observed in the open boreholes during, and immediately following, the drilling operations and a standpipe piezometer was installed in ONR-3 to permit monitoring of the groundwater level. The piezometer consists of a 60 mm diameter polyvinyl chloride pipe, with a slotted screen sealed within a sand filter pack at a selected depth interval within the borehole. Above the sand filter pack and piezometer screen, the annulus surrounding the piezometer pipe was partially backfilled with bentonite pellets to create a seal, then backfilled to near surface with cuttings from the boreholes and bentonite to within about 1 m of the ground surface. A seal of bentonite was placed to ground surface. The piezometer installation details and water level readings are indicated on the borehole record contained in Appendix A.

The fieldwork was supervised on a full-time basis by a member of Golder's staff who located the boreholes in the field, cleared the site for buried services, directed the drilling, sampling and in situ testing operations and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder's laboratory in Sudbury for further examination and laboratory testing. Index and classification tests consisting of water content, Atterberg limits and grain size distribution were carried out on selected soil samples. One-dimensional consolidation (oedometer) tests were carried out on Shelby tube samples of the cohesive soil deposit from two boreholes. The geotechnical laboratory testing was carried out according to applicable MTO LS standards.

The borehole locations and elevations were measured in the field by Golder personnel relative to stakes installed by Callon Dietz Inc. The borehole locations (referenced to the MTM NAD83 co-ordinate system), ground surface elevations (referenced to Geodetic datum) and borehole depths are presented on the Record of Borehole sheets in Appendix A and are summarized below.

<b>Borehole Number</b>	<b>MTM NAD83 Northing (m)</b>	<b>MTM NAD83 Easting (m)</b>	<b>Ground Surface Elevation (m)</b>	<b>Borehole Depth (m)</b>
ONR-1	5 286 646.6	393 643.3	256.8	18.5
ONR-2	5 286 672.3	393 598.9	248.3	15.2
ONR-3	5 286 722.5	393 557.5	251.4	16.4
ONR-4	5 286 746.6	393 522.9	256.8	25.0

## **4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS**

### **4.1 Regional Geology**

Based on NOEGTS<sup>1</sup> mapping, the subsoils in the vicinity of the ONR Overhead Bridge site consist of glaciolacustrine plane deposits generally composed of clay and silt. The site is located in a varved clay belt that stretches between New Liskeard and Cochrane.

Published literature indicates that the bedrock at this site consists of dolostone, limestone, sandstone or shale<sup>2</sup>.

<sup>1</sup> Northern Ontario Engineering Geology Terrain Study, Ontario Geological Society Map Reference Number 5021.

<sup>2</sup> Geological Highway Map, Northern Ontario, Ontario Geological Society Map Reference Number 2440.





## **4.2 Subsurface Conditions**

The preliminary foundation investigation consisted of drilling four boreholes advanced in the vicinity of ONR Overhead Bridge. The borehole locations, ground surface elevations and interpreted stratigraphic conditions at the site are shown on Drawing 1. The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are given on the Record of Borehole sheets contained in Appendix A. The results of the in situ tests (i.e. SPT “N”-values and undrained shear strengths from the field vanes) as presented on the Record of Borehole sheets and in Section 4 are uncorrected. The results of geotechnical laboratory testing are also presented in Appendix B. The stratigraphic boundaries shown on the Record of Borehole sheets and on the interpreted stratigraphic profile on Drawing 1 are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In summary, the subsoil conditions encountered at the site generally consist of embankment fill underlain by topsoil (where encountered), a deposit of varved silty clay and deposits of sandy silt to silty sand and gravel and cobbles (where encountered).

### **4.2.1 Fill**

An approximately 200 mm and 170 mm thick upper layer of asphalt was encountered from ground surface in Boreholes ONR-1 and ONR-4 at Elevation 256.8 m underlain by a layer of sand and gravel to gravelly sand fill approximately 1.1 m and 0.4 m thick, respectively. A lower layer of asphalt approximately 100 mm and 500 mm thick was encountered underlying the granular layer in Boreholes ONR-1 and ONR-4, respectively.

A 0.3 m thick layer of topsoil fill was encountered from ground surface in Borehole ONR-3 at Elevation 251.4 m.

Below the lower layer of asphalt in Boreholes ONR-1 and ONR-4, from ground surface in Borehole ONR-2 at Elevation 248.3 m and below the topsoil fill in Borehole ONR-3, a 2.0 m to 8.0 m thick deposit of fill comprised of sand to silty sand was encountered. Cobbles were noted within the fill in Boreholes ONR-1 and ONR-3.

Below the sand to silty sand fill, Boreholes ONR-1, ONR-3 and ONR-4 encountered silty clay fill at depths of 9.1 m (Elevation 247.7 m), 2.3 m (Elevation 249.1 m) and 9.1 m (Elevation 247.7 m), respectively, with respective layer thicknesses of 3.1 m, 0.7 m and 1.9 m. Organic seams and/or pockets were noted within the majority of the silty clay fill samples.

SPT “N”-values measured in the sand to silty sand fill deposit range between 7 blows and 45 blows per 0.3 m of penetration and two values of 57 blows per 0.2 m of penetration, indicating a loose to very dense relative density. Measured “N”-values in the silty clay fill layer range between 0 blows (weight of hammer) and 7 blows per 0.3 m of penetration, suggesting a very soft to stiff consistency.

The results of grain size distribution testing completed on six selected samples of the sand to silty sand portion of the fill deposit are shown on Figure B1 in Appendix B.

An Atterberg limits test was carried out on one sample of the cohesive fill and measured a liquid limit of 37 per cent, a plastic limit of 17 per cent and plasticity index of 20 per cent. These results, which are plotted on a plasticity chart on Figure B2 in Appendix B, indicate that the tested sample consists of a silty clay of intermediate plasticity. The results of grain size distribution testing completed on one sample of the silty clay fill are shown on Figure B3 in Appendix B.





The natural moisture content measured on samples of the sand to silty sand fill ranges from 3 per cent to 10 per cent. The natural moisture content measured on one sample of the silty clay fill is 23 per cent.

#### **4.2.2 Topsoil**

An approximately 0.1 m thick deposit of clayey topsoil was encountered below the fill in Borehole ONR-2 at Elevation 246.0 m and an approximately 0.6 m thick deposit of silty topsoil was encountered below the fill in Borehole ONR-4 at Elevation 245.8 m.

#### **4.2.3 Silty Clay**

A deposit of silty clay, 5.8 m to 11.1 m thick was encountered below the fill in Boreholes ONR-1 and ONR-3 and below the topsoil in Boreholes ONR-2 and ONR-4. The silty clay deposit was noted to be varved, typically with approximately 10 mm thick clayey silt laminae and approximately 20 mm thick clay laminae. The surface of this deposit was encountered at depths between 2.4 m and 12.2 m below ground surface between Elevation 248.4 m and 244.6 m.

The SPT “N”-values measured in the silty clay deposit range from 0 blows (i.e. weight of hammer) to 8 blows per 0.3 m of penetration and in situ field vane testing carried out in the silty clay deposit measured undrained shear strengths ranging from 19 kPa to 65 kPa, with a sensitivity between 4 and 15. The in situ vane test results, together with the SPT ‘N’-values, indicate that the silty clay to clay deposit generally has a soft to stiff consistency.

Atterberg limits testing was carried out on seven samples of the varved silty clay and the measured liquid limits range from 40 per cent to 49 per cent, plastic limits range from 20 per cent to 22 per cent and plasticity indices range from 19 per cent to 28 per cent. The results, which are plotted on a plasticity chart on Figure B4-1 in Appendix B, indicate that the tested samples of the overall deposit consist of silty clay of intermediate plasticity.

Atterberg limits tests were also carried out on six samples of the deposit separated into the clayey silt laminae and the clay laminae and the test results are shown on Figure B4-2. The test results on the clay laminae indicate liquid limits ranging from 63 per cent to 77 per cent, plastic limits ranging from 26 per cent to 28 per cent and plasticity indices ranging from 37 per cent to 49 per cent. For the clayey silt laminae, the liquid limits range from 33 per cent to 35 per cent, the plastic limits range from 19 per cent to 22 per cent and the plasticity indices range from 13 per cent to 16 per cent. The test results confirm that the ‘silty’ varves are classified as clayey silt of low plasticity and the clayey varves are classified as clay of high plasticity.

The results of a grain size distribution test completed on a sample of clayey silt lamina are shown on Figure B5 in Appendix B.

Two laboratory consolidation (oedometer) tests were carried out on specimens of the silty clay stratum obtained from Boreholes ONR-2 and ONR-4 and the test results are shown on Figures B6 and B7, respectively. The preconsolidation stresses were estimated from the Void Ratio versus logarithmic Pressure plots using the Casagrande method as well as from the Total Work versus Pressure plots. The relevant consolidation test results are summarized below.





Borehole/ Sample Number	Elevation (m)	$\sigma_{vo}'$ (kPa)	$\sigma_p'$ (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	$e_o$	$C_c$	$C_r$	$c_v^*$ (cm <sup>2</sup> /s)
ONR-2/7	242.0	90	90	0	1.0	1.575	0.38	0.050	$9.0 \times 10^{-4}$
ONR-4/15	240.0	250	250	0	1.0	1.024	0.20	0.035	$4.2 \times 10^{-3}$

Note: \*For approximate stress range of  $70 \leq \sigma_v' \leq 140$  kPa (ONR-2/7) and  $140 \leq \sigma_v' \leq 560$  kPa (ONR-4/15)

where:  
 $\sigma_{vo}'$  effective overburden stress in kPa  
 $\sigma_p'$  preconsolidation stress in kPa  
OCR overconsolidation ratio  
 $e_o$  initial void ratio  
 $C_c$  compression index (based on void ratio)  
 $C_r$  recompression index (based on void ratio)  
 $c_v$  coefficient of consolidation in cm<sup>2</sup>/s in the normally consolidated range

The natural moisture content measured on samples of the overall varved silty clay deposit (combined clayey silt and clay laminae) range from 35 per cent to 54 per cent. The nature water content measured on samples of individual clayey silt laminae range from 31 per cent to 42 per cent and clay laminae range from 52 per cent to 87 per cent.

The summary results of selected field and laboratory test results are presented on Figure 1. The values presented are uncorrected.

#### 4.2.4 Sandy Silt to Silty Sand

A deposit of sandy silt to silty sand was encountered below the silty clay deposit in Boreholes ONR-2 to ONR-4. The surface of this deposit was encountered between the depths of 12.0 m and 22.7 m below ground surface (between Elevation 237.7 m and 234.1 m).

The measured SPT "N"-values within the sandy silt to silty sand deposit range from 0 blows (i.e. weight of rods) to 15 blows per 0.3 m of penetration, indicating a very loose to compact relative density.

The results of grain size distribution testing completed on three samples of the sandy silt to silty sand deposit are shown on Figure B8 in Appendix B.

Atterberg limits testing was carried out on two samples of the silty sand portion of this deposit and measured are plastic limits of 13 per cent, liquid limits of 10 per cent and plasticity indices of 3 per cent. These results, which are plotted on a plasticity chart on Figure B9 in Appendix B, indicate that the samples are classified as a silt of slight plasticity.

The natural moisture content measured on three samples of the deposit range from 9 per cent to 27 per cent.

#### 4.2.5 Gravel and Cobbles

A 0.5 m thick deposit of gravel and cobbles was encountered in Borehole ONR-1 below the silty clay deposit at a depth of 18.0 m below ground surface (Elevation 238.8 m).

One SPT "N"-value in the deposit is 70 blows per 0.2 m of penetration, indicating a very dense relative density.





#### **4.2.6 Refusal**

Borehole ONR-1 was terminated upon reaching split-spoon refusal and refusal to further casing advancement at a depth of 18.5 m below ground surface (Elevation 238.3 m). Auger refusal was encountered in Boreholes ONR-2 and ONR-3 at depths of 15.2 m and 16.4 m below ground surface, respectively (Elevation 233.1 m and 235.0 m). In Borehole ONR-4, a DCPT was advanced from the bottom of the sampled borehole at 23.5 m to refusal at 25.0 m below existing ground surface (Elevation 231.8 m). These refusal depths, while they do not confirm bedrock elevations, may be inferred to indicate proximity to the bedrock surface.

#### **4.2.7 Groundwater Conditions**

The water level in Borehole ONR-2 upon completion of drilling was measured at 3.9 m below ground surface (Elevation 244.4 m) on July 20, 2012. A piezometer was installed in Borehole ONR-3, sealed within the sandy silt deposit, to monitor the groundwater levels and the measured groundwater level in the piezometer was 7.0 m below ground surface (Elevation 244.4 m) on August 1, 2012, approximately one week after installation. The water levels in Boreholes ONR-1 and ONR-4 were not measured upon completion of drilling.

Groundwater levels are subject to seasonal fluctuations and to fluctuations after precipitation events and snowmelt.

### **5.0 CLOSURE**

The field drilling program was supervised by Mr. Ed Savard. This Preliminary Foundation Investigation Report was prepared by Mr. André Bom, P.Eng. Mr. Jorge Costa, P.Eng., a Designated MTO Foundations Contact and Principal with Golder, carried out an independent quality control review of this report.





## Report Signature Page

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# PART B

PRELIMINARY FOUNDATION DESIGN REPORT  
REPLACEMENT OF ONR OVERHEAD BRIDGE, SITE 47-037  
HIGHWAY 11, EARLTON, ONTARIO  
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GWP 5265-01-00





## 6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides preliminary foundation design recommendations for the proposed replacement of the ONR Overhead Bridge on Highway 11. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the preliminary design of the structure foundations. Further investigation and analysis will be required during detail design.

Where comments are made on construction, they are provided to highlight those aspects that could affect the future detail design of the project. Those requiring information on construction aspects should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

### 6.1 General

The existing nine-span ONR Overhead Bridge was constructed in 1961 and is supported by deep foundations inferred to be founded on or in proximity to the bedrock surface based on available information. The existing concrete girder bridge, which is 135 m long and 11 m wide, crosses both the ONR and a local roadway (Government Road). The topography of the bridge site is generally flat, such that approximately 9 m high earth fill approach embankments were constructed to access the bridge over the ONR tracks. The existing highway grade is at about Elevation 257 m at the south and north bridge abutments, corresponding to about 7 m above Government Road (Elevation 250 m) and about 8 m above the ONR grade (Elevation 249 m). The approach embankments were constructed with earth fill stability berms, consisting of an upper berm up to 6 m high and up to about 17 m wide and a lower berm up to 3 m high and up to about 10 m wide beyond the toe of the upper berm. The width of the berms decreases with distance away from the bridge. We understand that the structure is nearing the end of its service life and therefore replacement will be required.

The four boreholes advanced at the site for the current foundation investigation encountered embankment fill materials underlain by a thin layer of topsoil (where encountered) and a deposit of soft to stiff varved silty clay approximately 5.8 m thick near the existing south abutment (Borehole ONR-1) and approximately 11.1 m thick near the north abutment (Borehole ONR-4). At Boreholes ONR-2 and ONR-3, advanced in the immediate vicinity of the piers, the silty clay deposit is approximately 9.6 m and 10.7 m thick, respectively. Below the silty clay, the boreholes encountered a deposit of sandy silt to silty sand or a deposit of gravel and cobbles.

### 6.2 Site History

Based on available information, including reports and drawings obtained from MTO's GEOCRES system, the following has been deduced about the site and existing structure:

- GEOCRES 31M-032 by Trow, Soderman and Associates (1958): Foundation Investigation and Design was completed in 1958 for the existing structure. The following information is included in the report:
  - In the 1930s, the south approach embankment immediately adjacent to the railway “suddenly” failed after it was constructed to a height of about 8 m with side slopes 1.5H:1V. Following the failure, bridge construction was abandoned and the fill was redistributed over the site.





- Prior to construction of the existing bridge, the undrained shear strength of the upper approximately 1 m thick zone of the silty clay deposit reportedly was greater than 50 kPa and the remainder of the deposit had yielded field vane shear strengths between 15 kPa and 50 kPa.
- Two alternatives were recommended for reconstruction of the new embankments: side slopes of 3.5H:1V or a berm 3 m high and 13.5 m wide with side slopes of 2H:1V, which was ultimately selected for construction. The previously redistributed fill from the 1930s failure was integrated into the embankment as portions of the toe berm where applicable.
- Settlement of the approach embankments was estimated to be about 2.4 m and require 85 years for 50 per cent consolidation to occur. To enhance drainage from the clay deposit and the near surface zone, recommendations for drainage were given in the report for incorporating sand trenches below the embankments, however, it is unknown if the sand trenches were constructed.
- GEOCRESS 31M-019 by the Department of Highways (1961): Foundation Design was completed in April 1961 following the failure of the south approach embankment side slopes in September 1959 immediately or soon after the embankment was constructed to full height (9 m). The following information is included in the report:
  - The 1959 failure occurred following a period of heavy rain.
  - Reportedly, failure occurred partly due to the presence of the redistributed 1930s silty clay fill. A plan and profile of the embankment geometry pre- and post-failure is included in the report.
  - The report recommended upper and lower stability berms each 6 m and 3 m high, respectively, with a width of 20 m and 10 m, respectively. A summary of the calculated Factors of Safety (FoS) from the results of embankment stability analysis are included in the report and indicate a FoS of 1.67 for deep seated failure based on incorporating the recommended upper and lower berms and based on an undrained shear strength ( $s_u$ ) of 20 kPa for the native varved silty clay below the embankment fill.
  - A total of approximately thirty (30) "Movement Check Points" were recommended for installation in the upper and lower berms as well as near the toe of the embankment. The proposed check points were to consist of a 125 mm outer diameter and 2.4 m long steel pipe installed inside a 2.1 m deep auger hole backfilled with sand. At the top of the steel pipe, a wooden plug was installed with a reference nail for survey. Reportedly, movement readings were measured during the reconstruction of the fill and "no movements of significance were observed".
- The rehabilitation drawing for the existing structure dated March 1982 depicts settlement of the north approach embankment of about 350 mm immediately behind the abutment.
- A publication by Milligan et al. (1962), presents various case studies of Canadian varved clays including a summary of the failure at this Highway 11 ONR Overhead site and the recommended toe berm geometry.

The current site is located in a varved clay belt that stretches between New Liskeard and Cochrane. At another Highway 11 ONR crossing located approximately 40 km south of the current site near New Liskeard, north of Highway 65 and south of Wabi River, an embankment failure occurred during construction in 1963. The embankment suddenly failed when constructed to a height of 6 m. The New Liskeard ONR site is underlain by a deposit of much greater thickness of varved clay than at the ONR Overhead site near Earleton (in excess of 30 m). A technical paper by Lo and Stermac (1965) was published following the New Liskeard ONR failure in





1963, building on the concepts from the 1959 failure at the current site, at Earlton, discussed above. The existing ONR crossing at the New Liskeard site is at grade and was reportedly reconstructed after failure to its former grade by adding a berm about half the height of the embankment.

In summary, published information for the Earlton and New Liskeard Highway 11 embankment construction associated with the bridge crossings over the ONR indicates that large embankments constructed over the varved clay:

- undergo large settlement and continue settling over a very long period of time;
- are unstable above heights of about 5 m or 6 m without staged construction methods which allow for the dissipation of pore pressures;
- require large toe berms or flat side slopes; and
- are sensitive to small changes in embankment geometry (i.e. settlement and stability mitigation measures are required).

### **6.3 Review of Preliminary Alignment Alternatives**

For the proposed replacement structure, various preliminary alignment and bridge span alternatives have been considered by URS, including:

- a new 140 m long bridge on the existing structure alignment, with a detour crossing the ONR either at grade or overhead;
- shortening the bridge to a length of either 72 m or 92 m by locating the abutments (and therefore approach embankments) closer to the railway and Government Road and using a detour during construction;
- staged replacement, with a slight shift in the centreline; and
- a new bridge on a new alignment some distance from the existing alignment.

The selection of the preferred alternative for the replacement structure must take into account the presence of the 6 m to 11 m thick deposit soft to stiff varved silty clay. Mitigation of stability and settlement will be required for the new embankments on a new alignment. Alternatively, if the existing alignment is maintained, mitigation will be required if a change to the existing embankment geometry (i.e. widening or a grade raise) is proposed. Further, if a staged replacement is considered, the new widening and raising of the existing embankments will negatively impact the stability and settlement of the existing embankments, while at the same time traffic flow is to be maintained on the existing embankments during construction.

Mitigation of instability may include:

- increasing the size of the existing toe berms, if maintaining the existing alignment;
- incorporating large multilevel toe berms into the embankment geometry or constructing the side slopes at shallow angles if on a new alignment;





- using lightweight fill (i.e. EPS) in lieu of granular fill in the embankment mass to offset the load increase from the widening/raising of the existing embankments or constructing new embankments on a new alignments; and/or
- using staged construction methods incorporating wick drains and/or preloading if on a new alignment.

If the new alignment is located within the footprint of the existing toe berms, sub-excavation of the toe berms may not be possible due to the potential impact on the stability of the existing embankment.

Mitigation of settlement may include:

- using lightweight fill in lieu of granular fill for construction of portions of the raisings/widening of the existing embankments or new embankments;
- constructing the new embankments in stages and allowing for preloading of the foundation deposits if on a new alignment; and/or
- installing wick drains into the clay deposit to increase the rate of settlement of the deposit under the embankments on a new alignment.

Settlement mitigation of the existing approach embankments and railway embankment may be required depending on the proximity of the new approach embankments to the existing embankments.

A summary of the advantages, disadvantages and risks/consequences from a preliminary foundations design perspective for each bridge replacement alternative is presented in Table 1. From a foundations perspective, we recommend that a new bridge of approximately the same length as the existing bridge (i.e. 140 m long) be constructed on the existing alignment, to take advantage of the presently “preloaded” foundation footprint and to reduce the need for extensive measures for the mitigation of total and differential settlement and stability that would be required for other alternatives. For this alternative, traffic flow should be routed to an at-grade detour crossing the ONR during construction of the new structure.

It is understood that the preferred bridge replacement alternative will require widening and a grade raise of the existing embankments to meet current highway design and railway clearance requirements. Therefore, it is recommended that the existing embankment geometry be maintained to the extent possible with minimal widening or grade raise in order to minimize the need for (and cost of) stability and settlement mitigation measures, as even small changes in geometry could have a significant impact on both stability and settlement of the approach embankments.

Based on preliminary assessments, shortening the new bridge to 72 m (or less) on the existing alignment is not feasible from a foundations perspective due to the following:

- There has been a history of previous embankment failures at this site and there is evidence that the clay deposit below the existing embankments has been partially remoulded. Given the history of poor performance and the high risk of variability in the subsurface conditions at the site, there is a risk that constructing abutments and high embankments (i.e. 10 m) in close proximity to the rail alignment may affect the operation of the rail line due to unexpected movements, settlements or the like.
- The currently available subsurface data from the preliminary foundation investigation is limited to four boreholes advanced through the existing embankment fill and existing front slope toe berms where the cohesive deposit has undergone some degree of consolidation. It is likely that additional boreholes





advanced beyond the limits of the existing fills (and closer to the ONR rail alignment) could reveal lower shear strengths than those measured to date and used for the stability analysis. If the clay deposit exhibits lower shear strengths in these areas, larger amounts of EPS and/or larger toe berms may be required to maintain stability.

- Given that we (and MTO) are not aware of a precedent for constructing an embankment with 9 m to 10 m of EPS fill in Ontario, it is not recommended to attempt a design that pushes the limits of experience at a site with a history of previous failures and soft and variable subsurface conditions that is adjacent to a rail line that will be adversely affected by unexpected ground movements.

The bridge replacement alternative of a new 92 m long bridge constructed on the same alignment as the existing bridge would require a substantial volume of lightweight fill (i.e. EPS) to be incorporated into the construction of the extended approach embankments, but is considered technically feasible from a foundations perspective. Based on preliminary estimates of EPS volume, the 92 m long structure would require at least 5,700 m<sup>3</sup> extra EPS to be incorporated into the extended portion of the approach embankment relative to the 140 m long structure. Given that the construction cost savings of the 92 m span bridge in comparison to the 140 m span bridge is on the same order of magnitude as the cost premium associated with the additional EPS requirements for the 92 m bridge, it was determined by the Project Team that there is no significant cost advantage of the 92 m long structure relative to the 140 m long structure. Given that the 92 m structure presents inherent risks from a settlement and stability perspective that cannot be fully assessed based on the limited borehole investigations undertaken for this study, the 140 m span structure has been recommended.

A new bridge on a new alignment is considered technically feasible from a foundation perspective, provided that the approach embankments for the new alignment are constructed a sufficient distance from the existing embankments (i.e. greater than 100 m centerline to centerline) to minimize settlement and stability issues associated with the existing embankments. It should be noted that under this scenario, there are still likely to be settlement and stability concerns associated with the construction of the new embankments considering that the soil conditions in the new embankment areas are anticipated to be similar to the conditions in the vicinity of the existing bridge and embankments.

A new bridge on a new alignment that is located adjacent to the existing alignment is not considered technically feasible due to the large magnitude of settlement that could potentially occur under the existing embankments during construction as a result of the adjacent embankment loadings. An acceptable offset between the new alignment and existing alignment would need to be confirmed during detail design if this option is considered further, but it is anticipated that it would be on the order of approximately 100 m (as noted above) based on the preliminary analysis that has been conducted for this study.

The alignment shift (staged replacement) alternative is not considered technically feasible due to stability concerns with expanding the existing embankments during construction and potential negative impacts on the safety of live traffic on the existing Highway 11 during construction.

## 6.4 Approach Embankments

The following sections present the results of stability and settlement analysis for the immediate approach embankments for the preferred replacement alternative, consisting of a new bridge of the same approximate





length (i.e. 140 m) as the existing bridge constructed on the existing Highway 11 alignment. Recommendations for stability and settlement mitigation measures, as required, are also provided.

Constructing the new bridge on the existing alignment with essentially the same approach embankment footprint, takes advantage of the existing stability berms. Further, constructing the bridge on the existing alignment takes advantage of the “preloaded” condition of the cohesive foundation soils from the existing embankment loading. Based on the thickness of asphalt and granular padding encountered at Boreholes ONR-1 and ONR-4 at the south and north approaches, respectively, the magnitude of settlement that has occurred since the completion of embankment construction in 1961 is estimated to be at least 1.1 m and 1.4 m, respectively.

We understand from URS that in order to meet current highway design and railway clearance standards, modification to the existing embankment geometry is as follows:

- a 0.4 m grade raise at the existing/new abutment locations;
- a higher grade raise beyond about 20 m from the abutments to correct the vertical “sag”, up to about 2 m; and
- an embankment widening of about 3 m on both sides of the existing embankments to accommodate standard lane widths and new 1.5 m wide shoulders.

Other geometry modifications were reviewed during preliminary design to accommodate/consider girder types, highway design speeds and shoulder widths, however, the above modifications are considered the minimum possible to meet the current highway standards. Given that small changes in embankment geometry can lead to significant negative impacts on embankment stability and settlement due to the presence of the cohesive deposit, the stability and settlement analysis assumes that the above geometry changes are the only ones that will be made to the current structures. Due to the limited extent of the subsurface investigation, only the approach embankments within 20 m of the proposed abutments have been analyzed. The profiles and cross-sections of the proposed embankment geometry used in the stability and settlement analysis for the alternatives described above have been provided by URS.

For the purpose of the preliminary stability and settlement analyses for the proposed approach embankments, the following assumptions have been made:

- The existing embankment fill will remain in place except where new fill is required for the widening and grade raise as well as for abutment construction.
- Any toe berm extensions will be constructed out of similar fill (i.e. cohesionless soils).
- New granular fill will replace the existing fill immediately behind the new abutments and below the approach slabs. Side slopes for the new granular fill are 2H:1V.
- The piezometric conditions used in the analyses are based on the water level measured in the piezometer in Borehole ONR-3 on September 27, 2012 at Elevation 248.4 m. Where the existing ground surface is lower than Elevation 248.4 m, the piezometric condition is assumed to be at the existing ground surface.





### 6.4.1 Approach Embankment Stability

Analyses were performed on the critical sections of the proposed approach embankments for the replacement structure on the existing alignment to assess the stability for the proposed embankment height, proposed highway geometry and soil stratigraphy. The critical embankment sections at this site are the side slopes, however, the front slopes (toward the railway) have also been checked. The geometry of the existing and proposed approach embankments and existing ground surface included in the analyses are based on the information provided by URS.

#### 6.4.1.1 Methodology

Limit equilibrium slope stability analyses were performed using the commercially available program GeoStudio 2007 (Version 7.19), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the Factor of Safety (FoS) of numerous potential failure surfaces was computed in order to establish the minimum FoS. The FoS is defined as the ratio of the forces tending to resist failure to the driving forces tending to cause failure. A target minimum FoS of 1.3 is normally adopted for the design of embankment slopes under static conditions at the end of construction. This FoS 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 FoS was achieved at the critical locations. In general, circular slip surfaces were analysed in the design. Analyses were carried out for total stress (i.e. undrained) conditions assuming the addition of new fill for the grade raise/widening will mobilize the undrained shear strength of the cohesive deposit or that the embankment may still be in a partially undrained state as settlement is still occurring (i.e. pore pressures not fully dissipated). In addition, effective stress (drained) analyses were conducted for the existing embankments to assess long-term conditions applying a FoS of 1.3.

#### 6.4.1.2 Parameter Selection

For the cohesionless fill, the topsoil layer, the silty clay fill and native cohesionless soil layers, effective stress parameters were employed in the analysis assuming drained conditions and the shear strength parameters were estimated from empirical correlations using the results of the in situ SPT “N”-values. The correlations proposed by Peck et al. (1974) were employed and the results were tempered by engineering judgment based on precedent experience in similar soils.

For the cohesive stratum, total stress parameters were employed in the analysis. The total stress parameters (i.e. undrained shear strength –  $s_u$ ) for the cohesive soil were assessed based primarily on the results of the in situ field vane tests. Bjerrum’s (1973) correction factor as a function of the plasticity index of the soil was employed to estimate the average mobilized undrained shear strength from the results of the in situ field vane tests; a correction factor of 5 per cent was applied. Further, a correction (reduction) factor of 15 per cent has been applied to the undrained shear strength of the clay deposit due to the clayey silt/clay varves noted within the cohesive deposit to account for the angle of minimum shearing resistance (Milligan and Lo, 1967). The profiles of undrained shear strength versus elevation, together with the selected design lines, based on the field and laboratory data below the approach embankments are presented on Figure 2 and at the piers on Figure 3.





The associated strengths and unit weight values assigned to the different soil types at the site are presented below.

Material Type			Unit Weight (kN/m <sup>3</sup> )	Undrained Shear Strength, s <sub>u</sub> (kPa)	Effective Friction Angle, $\phi$ ( <sup>o</sup> )
New Granular Fill			21	--	35
Lightweight Fill (i.e. EPS)			0.5	15	--
Existing Granular Fill and Berm Extension Fill (discussed in Section 6.4.1.3)			20	--	30
Existing Silty Clay Fill			18	--	26
Silty Topsoil			12	1	27
Silty Clay*	Beneath Existing Roadway (Full Embankment Height)	Below Elev. 243 m	18	36 (Figure 2)	23 <sup>o</sup> (c' = 0)
	Transition below upper berm		17	26 (Assumed)	
	Below lower berm and away from embankment	Above Elev. 244 m	16	18 (Figure 3)	
		Elev. 244 m - 240 m		16 (Figure 3)	
		Below Elev. 240 m		20 (Figure 3)	
Sandy Silt to Silty Sand			19	-	30
Gravel and Cobbles			19	-	40

### 6.4.1.3 Results of Analysis and Mitigation

Stability analyses were completed for the existing embankments assuming both undrained and drained conditions. The calculated FoS for the existing south and north approach embankments are discussed separately below.

- **South Approach Embankment:** The calculated FoS for the existing side slope and for the front slope, in undrained conditions is 1.2 and 1.4 as shown on Figures 4 and 5, respectively. The calculated FoS for both the existing side slope and for the front slope in drained conditions is greater than 1.3.
- **North Approach Embankment:** The calculated FoS for the existing side slope and for the front slope in undrained conditions is about unity and 1.2 as shown on Figures 6 and 7, respectively. The calculated FoS for both the existing side slope and for the front slopes in drained conditions is greater than 1.3. Based on these results, it is considered that the existing north approach side slope is “marginally” stable and that if the geometry is modified even slightly by raising and/or widening, the embankment side slopes would become unstable in the undrained condition. Although the FoS is essentially at unity for the undrained condition as shown on Figure 6, the analyses are applicable for the end-of-construction rapid loading conditions, yet depending on the staging on construction some portion of the drainage condition would apply as the embankment is constructed and therefore the FoS is greater than unity.

Based on the results of the slope stability analyses of the existing south and north approach embankments in undrained conditions described above, mitigation of embankment stability is required regardless of any geometry





changes as the FoS is less than 1.3. In addition, further mitigation measures will be required to the existing embankment geometry to allow for the currently proposed grade raise and widening. A comparison of alternatives to mitigate embankment instability for the new 140 m long bridge constructed on the existing alignment based on advantages, disadvantages, risks and relative costs is presented in Table 2. A combination of widening the existing stability toe berms and re-constructing the upper portion of the embankment with EPS fill is considered the most practical and technically feasible alternative from a foundations perspective.

The stability analyses for the South and North Approach Embankments within 20 m of the abutments were completed by varying the width of the stability berms and the thickness of the EPS fill until an adequate FoS was achieved for each approach embankment. Further, consideration was given in the analysis to maintaining the current berm footprints should additional right-of-way not be available to accommodate larger berms. The minimum recommended width of the stability berms and thickness of EPS fill to achieve a FoS of 1.3 or greater for each approach is presented below.

Approach Embankment	Minimum EPS Thickness	Width of Toe Berms *		Height of Side Slope Toe Berms	Stability Figure
		Side	Front		
South	2 m	20 m	Unchanged from existing (due to EPS placement); upper berm may be regraded/ lowered to match grade of lower berm	2 m	8 (Side Slope) 9 (Front Slope)
North	3 m	30 m (extends about 8 m beyond existing berm footprint)			10a (Side Slope)
	4 m (to maintain existing berm footprint)	21 m (to maintain existing berm footprint)			10b (Side Slope) 11 (Front Slope)

\* The existing side and front slope berm geometry is shown on Figures 4 to 7.

The preliminary assessment of the stability of the approach embankments within 20 m of the abutments discussed above should be reviewed and confirmed based on the subsoil conditions encountered within the proposed approach embankment footprints during Detail Design. Section 6.8 provides preliminary discussion on the embankment stability and mitigation and the requirement to obtain additional subsurface information beyond 20 m from the abutments.

#### 6.4.2 Approach Embankment Settlement

Settlement of the approach embankments can be expected to occur as a result of the loading on the compressible foundation soils from the proposed grade raise and widening relative to the existing embankment geometry. Settlement may also occur due to compression of the new and existing embankment fill itself. However, if the new embankment fill is comprised of granular materials and lightweight EPS fill, the settlement of the new fill is expected to be negligible. Further, settlement of the existing cohesionless fill material (i.e. sand to silty sand) is expected to be negligible due to the relative density of this material, and settlement of the native





cohesionless soil below the silty clay deposit resulting from the proposed embankment geometry is also expected to be negligible.

#### 6.4.2.1 Methodology

Settlement analyses for the foundation soils below the raised and widened approach embankments were carried out using hand calculations. The model geometry and stratigraphy are shown on Figures 4 and 5, as used for the stability analysis for the South Approach and North Approach side slopes, respectively.

#### 6.4.2.2 Settlement Criteria

Based on MTO's "Embankment Settlement Criteria for Design" Final Draft dated March 2, 2010, the following post-construction settlement and differential settlement criteria are considered acceptable to occur within 20 years post-paving for the bridge approach embankments.

Location	Distance from Transition Point (i.e. Abutment) (m)	Total Post-Construction Settlement (mm)
Transition/Taper to Bridge Abutments	0 to 20	25
	20 to 50	50
	50 to 75	75

These criteria have been used for determining whether mitigation measures are required to limit post-construction settlement of the approach embankments.

#### 6.4.2.3 Parameter Selection

The immediate compression of the existing fill and native cohesionless deposits was assessed by estimating an elastic modulus of deformation based on the SPT 'N'-values and empirical correlations found in literature by Bowles (1984) and Kulhawy and Mayne (1990) and engineering judgment from experience with similar soils in this region of Ontario.

The consolidation settlement of the soft to stiff silty clay deposit below the approach embankments was assessed using the results of the in situ field vane tests and the laboratory consolidation tests to estimate the deformation parameters for this soil. In addition, the results of the laboratory index testing were also employed to estimate deformation parameters using empirical correlations proposed in literature by Terzaghi and Peck (1967), Kulhawy and Mayne (1990), and Koppula (1986). Based on the results of the two laboratory consolidation tests, the silty clay deposit at this site is normally consolidated.

The following simplified stratigraphy, unit weights and deformation parameters have been employed in the settlement analysis of the proposed approach embankments.





Soil Deposit	Bulk Unit Weight (kN/m <sup>3</sup> )	Elastic Modulus (MPa)	Approximate Thickness (m)	
			South Approach (Borehole ONR-1)	North Approach (Borehole ONR-4)
Existing Cohesive Fill (Silty Clay)	18	1	3.1	1.9
Silty Topsoil	12	1	N/A	0.6
Soft to Stiff Silty Clay to Clay	16 to 18	See below	5.8	11.1

The following consolidation parameters were estimated for the silty clay deposit based on the results of laboratory consolidation tests performed on a specimen of the silty clay obtained from Borehole ONR-4 at the existing North Approach and from Borehole ONR-2 at the existing South Pier. The plot of preconsolidation pressure and in situ vertical effective stress versus elevation is provided on Figure 2 (below the embankments) and Figure 3 (at the piers).

Location	Approximate Elevation (m)	$\sigma_{vo}'$ (kPa)	$\sigma_p'$ (kPa)	OCR	$e_o$	$C_c$
South Approach (ONR-1)	244.6 to 238.8	(see Figure 2)		1.0	1.0	0.20
North Approach (ONR-4)	245.2 to 234.1					
Pier (ONR-2)	245.9 to 236.3	(See Figure 3)		1.0	1.6	0.38

Based on the results of the consolidation tests, a coefficient of consolidation,  $c_v$  (n/c), equal to  $9.0 \times 10^{-4}$  cm<sup>2</sup>/s is used for the calculation of the rate of primary settlement for the normally consolidated silty clay at this site.

In addition to primary consolidation within the silty clay, secondary compression may also occur. Secondary compression is referred to as creep settlement and occurs over a long period of time, after full dissipation of excess pore pressure under a constant stress. The following relationship has been employed for estimating the magnitude of creep settlement over the life of the embankment following the completion of primary settlement at each location:

$$S_c = HC_{\alpha\epsilon} \log\left(\frac{t}{t_{EOP}}\right)$$

where :

- $S_c$  = secondary consolidation (creep) settlement (mm)
- $C_{\alpha\epsilon}$  = modified secondary compression index as estimated from laboratory consolidation tests
- $H$  = initial thickness of compressible clay deposit (mm)
- $t$  = post-construction period of interest (20 years)
- $t_{EOP}$  = time to reach end of primary consolidation (years)

#### 6.4.2.4 Results of Settlement Analysis

A summary of the settlement analysis for a grade raise of approximately 0.4 m at the South and North Approach Embankments adjacent to the abutments is presented below.





Approach	Elastic Settlement of the Silty Clay Fill	Primary Consolidation of the Silty Clay Stratum	Secondary Consolidation of the Silty Clay Stratum
South	60 mm	30 mm	30 mm per log cycle
North	40 mm	50 mm	60 mm per log cycle

Based on a  $c_v$  (n/c), equal to  $9.0 \times 10^{-4} \text{ cm}^2/\text{s}$ , the estimated time for 90% primary consolidation to occur at the South and North Approaches is approximately 4 years and 9 years, respectively.

A comparison of the alternative settlement mitigation options for the new bridge on the existing alignment based on advantages, disadvantages, risks/consequences and relative costs is provided in Table 2 following the text of this report. The most technically feasible alternative to mitigate settlement at this site is the use of EPS fill for embankment re-construction. It is recommended that a sufficient thickness of EPS be provided within the new approach embankments such that a net “unloading” of the foundation soils occur, even with the proposed grade raise/widening. A net unloading is required since the embankments are settling under the current loading, even without considering the addition of new fill. The preliminary recommended minimum thickness of EPS to provide a net unloading and to limit the post-construction settlements to within the MTO settlement criteria outlined in Section 6.4.2.2 is 2 m and 3 m at the South and North Approaches, respectively. Further, note that the existing slope toe berms will also need to be widened and the upper tier berm lowered for stability considerations as presented in Section 6.4.1.3.

The EPS blocks should be covered with a polyethylene sheet and protected with a minimum 125 mm thick reinforced concrete slab on top of the blocks and an adequate pavement structure such that the total thickness of cover over the EPS is not less than 1 m. Further, the EPS should extend laterally across the embankment section, stepped to avoid conflict with the guide rail posts (where applicable) and configured such that the EPS is provided with a minimum of 1 m soil cover on the embankment side slopes.

The preliminary assessment of the settlement of the approach embankments within 20 m of the abutments discussed above should be reviewed and confirmed based on the subsoil conditions encountered within the proposed approach embankment footprints during Detail Design. Section 6.8 provides a preliminary discussion on embankment settlement and mitigation, and the requirement to obtain additional subsurface information beyond 20 m from the abutments.

## 6.5 Foundation Options

Due to the presence of the relatively thick deposit of soft to stiff cohesive soil at this site and the potential for large settlements, only deep foundation options have been considered for support of the replacement structure on the existing alignment. A summary of the advantages and disadvantages associated with deep foundation alternatives for the replacement structure on the existing alignment is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, relative costs and risks/consequences is provided in Table 3 following the text of this report.

- **Driven steel H-piles:** Steel H-piles driven to bedrock are feasible for support of the abutments and wing walls.





- **Driven steel pipe (tube) piles:** Steel tube (pipe) piles driven to bedrock could also be considered as a deep foundation option for support of the abutments and wing walls.
- **Caissons:** Caissons socketted into the bedrock are also considered to be feasible as a deep foundation option at this site. However, caissons are not generally constructed in northern Ontario due to constructability issues associated with large-diameter drill holes into the typically strong to very strong bedrock and other construction considerations, such as groundwater conditions, use of liner and tremie concrete placement.

From a foundations perspective, steel H-piles driven to bedrock are considered the most practical and straightforward as well as economical in terms of initial construction costs.

The selection of steel H-piles as the preferred foundation alternative should consider the generation of excess pore pressure in the weak clayey soils supporting the existing approach embankments, which should be reviewed in more detail during Detail Design.

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

### 6.6.1 Founding Elevations

The abutments, piers and wing walls may be supported on steel H-piles or steel pipe (tube) piles driven to bedrock. The following pile tip elevations may be used for preliminary design purposes:

Foundation Element (Borehole)	Recommended Preliminary Design Pile Tip Elevation	Estimated Underside of Pile Cap Elevation	Estimated Approx. Pile Length
South Abutment (ONR-1)	238 m	252 m	14 m
Pier #1 (ONR-1 & ONR-2)	235 m	248 m	13 m
Pier #2 (ONR-2)	233 m	245 m	12 m
Pier #3 (ONR-2 and ONR-3)	234 m	244 m	10 m
Pier #4 (ONR-3)	235 m	249 m	14 m
North Abutment (ONR-4)	231 m	252 m	21 m

The proposed pile tip elevations will have to be re-evaluated and modified as necessary during detail design, based on future additional subsurface investigation at the proposed abutment and pier locations. The pile caps should be constructed at a minimum depth of 2.3 m for frost protection purposes, per OPSD 3090.100 (Foundation Frost Penetration Depths for Northern Ontario).

For the installation of steel H-piles or steel pipe piles, consideration must be given to the presence of cobbles as encountered in one of the boreholes, and potential presence of boulders, overlying the bedrock. In this regard, steel H-piles are preferred over steel pipe piles as steel H-piles are considered to have a lesser potential of “hanging up” or being deflected away from their vertical or battered orientation during installation. The piles should be reinforced at the tip with driving shoes or flange plates in accordance with OPSD 3001.100 (Pile Driving Shoe) to reduce the potential for damage to the piles during driving, in accordance with OPSS 903 (Deep Foundations). In very dense and/or bouldery soils, as may be encountered at this site, driving shoes (such as Titus Standard “H” Bearing Pile Points) are preferred over driving shoes comprised of flange plates. If steel pipe





piles are used, driving shoes should be in accordance with OPSD 3001.100 Type II (Steel Tube Pile Driving Shoe).

### 6.6.2 Axial Geotechnical Resistance

For HP 310x110 piles driven to bedrock, a factored axial geotechnical resistance at Ultimate Limit States (ULS) of 2,000 kN per pile may be used for design. The geotechnical resistance for piles driven to bedrock at Serviceability Limit States (SLS) for 25 mm of settlement will be greater or equal to the factored axial resistance at ULS, since the bedrock is considered to be an unyielding material; as such, ULS conditions will govern for this foundation type.

The preliminary geotechnical resistances/reactions and pile size will have to be re-evaluated and modified as necessary during detail design in consideration of the additional subsurface investigation at the foundation elements. In addition, the axial resistance for closed-end, concrete-filled, 324 mm (12 ¾ in.) diameter steel pipe piles having a minimum wall thickness of 9.5 mm (3/8 in.) will need to be confirmed during Detail Design.

As the settlement of the re-constructed approach embankments to the new structure will need to be mitigated with a net unloading of the foundation soils, it is anticipated that negligible downdrag loads on the piles will occur. If a net unloading is not achieved (i.e. if a different settlement mitigation alternative is chosen, such as preloading), settlement of the foundation soils will occur resulting in negative skin friction and downdrag loads on the piles, the magnitude of which should be assessed during Detail Design.

The pile termination or set criteria will be dependent on the pile driving hammer type and the selected pile type. The set criteria can be established through a variety of methods including empirical correlations, such as the use of the Hiley Formula, 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.

## 6.7 Construction Considerations

The following subsections identify potential construction issues that should be considered at this preliminary stage of the design as they may impact the planning for Detail Design. Where applicable, Non-Standard Special Provisions (NSSP) should be developed during Detail Design for incorporation into the Contract Documents.

### 6.7.1 Excavation and Temporary Roadway Protection

The foundation excavations for pile caps will extend through the existing fill and below the groundwater level. Due to the potential for generation of excess pore pressure within the weak clayey soils during pile driving, it is recommended that the existing embankments be lowered to at least the underside of the proposed abutments such that the “lowered embankment” is 5 m or less prior to the start of pile installation at the site. Open-cut excavations into the fill material and silty clay deposit should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. Existing fill and the silty clay deposit would be classified as Type 3 soil, according to the OHSA. Temporary excavations (i.e. those that are open for a relatively short time period) should be made with side slopes no steeper than 1H:1V.





### 6.7.2 Obstructions

The soils immediately overlying the bedrock surface at one borehole location consists of coarse gravel and cobbles and in places may potentially contain boulders, which could affect the installation of deep foundations. Further observations for the presence of cobbles and/or boulders are recommended in the next stage of investigation in support of the Detail Design. If conditions warrant, an NSSP should be included in the Contract Documents developed during the detail design stage to identify to the contractor the possible presence of cobbles and/or boulders within the overburden soils.

## 6.8 Recommendations for Further Work During Detail Design

Due to the highly complex nature of the foundation soils and the history of poor performance of the embankments at the site, additional investigation and assessment will be required at the Detail Design stage of the preferred bridge replacement option to confirm the subsurface conditions and the preliminary recommendations provided herein. The additional investigation and assessment, including boreholes and Cone Penetration Tests (CPTs), is required at the approach embankment areas and at the preferred structure foundation elements, as recommended below.

At approach embankment areas:

- Additional boreholes and CPTs in an alternating pattern to assess the depth and extent of the existing fill materials and underlying varved silty clay deposit within the footprint of the existing/new approach embankments from 20 m to at least 200 m behind the abutments due to the need for, what is considered for this site, a relatively large grade raise to improve the vertical curve (i.e. sag) in the highway vertical alignment. The boreholes and CPTs should be spaced no further than 25 m apart along the centreline of the highway alignment and at least one pair of immediately adjacent boreholes/CPTs should be carried out on each side of the bridge.
- Additional boreholes and CPTs in an alternating pattern to assess the existing berm fill materials and underlying varved silty clay deposits below and adjacent to the side slope and front berm areas. The boreholes and CPTs should be spaced no further than 25 m apart in the front slope berm areas at locations between the piers, if applicable, and staggered on the upper/lower side slope berms at 25 m spacing to the end of the berms, as applicable. Additional boreholes and CPTs (minimum 6 on each side of the bridge) in an alternating pattern should be located at least 15 m away from the toe(s) of the existing front and side slope berms in order to assess the strength of the clay deposits beyond the influence of the existing embankment fills.
- Further assessment of the stability and details of the preferred mitigation alternatives for the embankment side slopes and front slopes, including a detailed survey of the berm geometries for the stability analyses.
- Further assessment of the estimated magnitude of settlement and details of the preferred mitigation alternatives for the proposed approach embankments.
- Detailed assessment of the required extent and thickness of light-weight EPS fill and the applicability of and duration of preloading, and/or combinations of mitigation measures such as preloading and





incorporation of lightweight fill into the embankment design (i.e beyond 20 m from the abutments, as the embankment grade decreases).

At each foundation element:

- Additional boreholes within the footprint of the proposed abutment and pier foundation elements, including penetrating 3 m into “refusal” or bedrock to confirm the design tip elevation for driven piles.
- Comments on the presence of cobbles and/or boulders overlying bedrock as encountered during borehole drilling, to assess the need for an NSSP to warn the contractor of the presence of such obstructions as they may affect excavations and the installation of deep foundations.
- Assessment of potential for and magnitude of downdrag loads on the piles depending on the estimated settlement of the cohesive soil strata.
- Assessment of the potential for the generation of excess pore pressures within the weak clayey foundation soils during pile driving, which at the Detail Design stage may require lowering of the existing approach embankments in the abutment areas during pile driving.
- Assessment of the need for a foundation monitoring program to measure excess pore pressure and settlement.

As noted in Section 6.3, the potential for and implications of constructing the new bridge on a different alignment were discussed with MTO/URS. For such an alternative alignment to be considered further, it will be necessary to advance boreholes and CPTs to assess the subsurface conditions for the replacement bridge on the new alignment (i.e. to be positioned 100 m or greater east of the existing centreline). If the cohesive deposits are less extensive and/or have a higher undrained shear strength on the new alignment, realigning the highway and relocating the crossing could be a potential replacement alternative. However, based the relatively flat topography in the vicinity of the site and the nature of the regionally deposited cohesive soil between New Liskeard and Cochrane, there may not be improved subsurface conditions in such close proximity east of the existing structure.

## **7.0 CLOSURE**



This Preliminary Foundation Design Report was prepared by Mr. André Bom, P.Eng., a geotechnical engineer and technical input provided by Ms. Sarah E.M. Coyne, P.Eng. and Mr. P. Dittrich, senior geotechnical engineers and Associate/Principal, respectively. Mr. Jorge M. A. Costa, P.Eng., a Principal and Designated MTO Foundations Contact for Golder, conducted an independent review of this report.





## Report Signature Page

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AB/JMAC/kp

<http://capws.golder.com/sites/capws2/p111910025groundhogriverandonrbridges/reports/onr/final/11-1191-0025-1 final rpt 13may10 prelim fdr onr.docx>





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  - OPSD 3090.100 Foundation, Frost Penetration Depths for Northern Ontario
- Ontario Provincial Standard Specifications
- OPSS 903 Construction Specification for Deep Foundations





Ontario Occupational Health and Safety Act:

Ontario Regulation 213/91 Construction Projects

Ontario Water Resources Act

Ontario Regulation 903/90 Wells; O.Reg 468/10 Amendment to Ontario Regulation 903





Table 1: Comparison of Bridge Replacement Alternatives

Options	Rank	Advantages	Disadvantages	Risks/Consequences
New Bridge on Existing Alignment and same approximate length as existing, i.e. 140 m (with an at grade detour)	1	<ul style="list-style-type: none"> <li>Existing embankments and toe berms are currently stable.</li> <li>Existing embankment fills have “preloaded” the foundation soils.</li> <li>Less volume of fill required.</li> </ul>	<ul style="list-style-type: none"> <li>EPS required, but volume is less than for 92 m long bridge.</li> <li>Additional spans/pier foundations required.</li> </ul>	<ul style="list-style-type: none"> <li>At grade detour will have to be constructed sufficiently away from existing embankments to allow for widening of existing embankments.</li> <li>Potential for differential settlement laterally between existing embankments and widened shoulders.</li> <li>While stability and settlement is mitigated by EPS, there is still the potential for differential settlement laterally.</li> </ul>
New Shorter Bridge (i.e. 92 m) on Existing Alignment (with an at grade detour)	2	<ul style="list-style-type: none"> <li>Fewer pier foundations required.</li> </ul>	<ul style="list-style-type: none"> <li>Foundation soils have not been “preloaded” in the areas between the existing and future abutment front slope and the railway and therefore mitigation of greater settlement required than for 140 m long bridge.</li> <li>Mitigation of settlement for existing railway and local roadway embankments required.</li> <li>Much greater EPS volume required than for 140 m long bridge.</li> <li>Greater volume of earth fill required than for the 140 m long bridge.</li> </ul>	<ul style="list-style-type: none"> <li>At grade detour will have to be constructed sufficiently away from existing embankments to allow for widening of existing embankments.</li> <li>Potential for settlement of existing railway and local roadway due to loadings from new adjacent embankments.</li> <li>Potential for differential settlement across the new roadway at location of extension of new fill.</li> <li>Risk of encountering softer and/or thicker clay soils below the proposed new closer approach embankments (i.e. in front of the existing abutments) negatively impacting stability and settlement of the embankments.</li> </ul>





Table 1: Comparison of Bridge Replacement Alternatives

Options	Rank	Advantages	Disadvantages	Risks/Consequences
New Bridge on New Alignment (with traffic maintained on existing bridge during construction)	3	<ul style="list-style-type: none"> <li>New alignment will not have to deal with a site with past failures and potentially remoulded soils.</li> </ul>	<ul style="list-style-type: none"> <li>Toe berms or shallow side slopes in conjunction with staged construction will be required to mitigate stability for the new approach embankments.</li> <li>Large volume of new fill required.</li> <li>Significant mitigation of settlement and time rate of settlement of the new embankments will be required.</li> <li>Settlement mitigation of the existing structure and embankments will also be required depending on the spacing of the two embankments.</li> <li>Potentially high cost for stability settlement mitigation measures such as for the use of wick drains, EPS.</li> </ul>	<ul style="list-style-type: none"> <li>Potential impact to the existing structure and embankments if the new structure is constructed adjacent to or near the existing bridges.</li> <li>Delay to schedule due to embankment preloading/staged construction.</li> </ul>
New Shorter Bridge (i.e. 72 m or shorter) on Existing Alignment (with an at grade detour)	NF	<ul style="list-style-type: none"> <li>Fewer pier foundations required.</li> </ul>	<ul style="list-style-type: none"> <li>Front slope toe berms would encroach onto local roadway and railway. Proximity of new fill to railway and local roadway induce settlement of the clay stratum under these embankments.</li> </ul>	<ul style="list-style-type: none"> <li>Risk of encountering softer and/or thicker clay soils below the proposed new closer approach embankments (i.e. in front of the existing abutments) negatively impacting stability and settlement of the embankments.</li> </ul>
New Bridge Constructed Using Staged Replacement (with slight shift in the centreline)	NF	<ul style="list-style-type: none"> <li>Reduced volume of fill compared with new alignment.</li> </ul>	<ul style="list-style-type: none"> <li>Large differential settlement of the approach embankment will occur as a result of the widening and the existing embankments and will settle and cause the existing foundations to experience additional downdrag loads.</li> <li>Staged construction and preloading would be required.</li> </ul>	<ul style="list-style-type: none"> <li>Potential of differential settlement between existing embankments and new embankment section, and between new embankments and new structure both during construction affecting traffic and in long term..</li> </ul>

NF: Considered not technically, cost or schedule feasible to mitigate stability and/or settlement.





**Table 2: Comparison of Stability and Settlement Mitigation Alternatives for New 140 m Long Bridge on Existing Alignment**

Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Stability Berms and Lightweight Fill (EPS)	1	<ul style="list-style-type: none"> <li>Overall stability takes advantage of existing large berms thereby reducing size of new berms.</li> <li>EPS reduces load on compressible subsoils thereby reducing post-construction settlement of foundation subsoils.</li> <li>Straightforward installation of EPS blocks within embankment fill.</li> <li>No time delay.</li> <li>Eliminates downdrag loading on piles provided a net unloading occurs.</li> <li>Buoyancy of EPS not a concern as the groundwater level is about 9 m below the top of the embankment (near original ground surface at the abutments).</li> </ul>	<ul style="list-style-type: none"> <li>Existing stability berms require lengthening or change in geometry for the proposed grade raise/widening.</li> <li>High cost of EPS material.</li> <li>Requires concrete slab and minimum of 1 m of conventional soil cover to mitigate potential for differential icing thereby restricting volume of lightweight fill to core area.</li> </ul>	<ul style="list-style-type: none"> <li>EPS cost is up to an order of magnitude higher than other fill materials.</li> <li>Additional property may be required in order to increase lengths of stability berms.</li> </ul>	<ul style="list-style-type: none"> <li>Increased lengths of stability berms at north approach may not be practical due to property restrictions, additional EPS required.</li> <li>Creep settlement will still occur.</li> </ul>
Change in Bridge Geometry (move abutments back and/or lower the approach embankment grade)	NF	<ul style="list-style-type: none"> <li>Lowering grade would reduce consolidation settlement of subsoils and improve stability.</li> </ul>	<ul style="list-style-type: none"> <li>Consolidation settlement of subsoils would still occur due to difference between existing and new fill a further distance away from existing abutments (not analyzed).</li> </ul>	<ul style="list-style-type: none"> <li>Longer bridge span would be more expensive.</li> </ul>	<ul style="list-style-type: none"> <li>Long stability berms and EPS still required to mitigate stability and settlement.</li> <li>Creep settlement will still occur.</li> </ul>





**Table 2: Comparison of Stability and Settlement Mitigation Alternatives for New 140 m Long Bridge on Existing Alignment**

Mitigation Option	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Preloading	NF	<ul style="list-style-type: none"> <li>Reduces post-construction settlement.</li> </ul>	<ul style="list-style-type: none"> <li>Strength gain of cohesive foundation soils from preloading would not be sufficient to reduce stability berm lengths.</li> <li>Time delay in schedule to allow for preloading (or staged preloading).</li> <li>Instrumentation and monitoring program would be required.</li> </ul>	<ul style="list-style-type: none"> <li>Increased overall costs due to extra time required in schedule.</li> <li>Cost of instrumentation and monitoring program.</li> </ul>	<ul style="list-style-type: none"> <li>Longer stability berms required without the use of EPS at north approach.</li> <li>Creep settlement will still occur.</li> </ul>
Sub-excavation of Silty Clay to Clay Deposit	NF	<ul style="list-style-type: none"> <li>Reduces post-construction settlement and improves stability.</li> </ul>	<ul style="list-style-type: none"> <li>Would require sub-excavation of existing embankments and below the existing berms.</li> <li>Requires excavation below the ground water level.</li> <li>Extensive shoring/ dewatering may be required.</li> <li>Requires site for disposal of excavated material.</li> </ul>	<ul style="list-style-type: none"> <li>Cost of excavation/ shoring.</li> <li>Cost of disposal of excess material and replacement backfill.</li> </ul>	<ul style="list-style-type: none"> <li>High risk of post-construction settlement of fill material placed below the ground water level.</li> <li>Long term settlement of rock fill will occur.</li> </ul>
Wick Drains	NF	<ul style="list-style-type: none"> <li>Reduces time for primary consolidation to occur.</li> </ul>	<ul style="list-style-type: none"> <li>May require sub-excavation of existing embankments.</li> <li>Increases magnitude of creep settlement.</li> <li>Must be used in conjunction with preloading/staged construction.</li> </ul>	<ul style="list-style-type: none"> <li>Additional cost of foundation investigation and design, installation of wick drains and instrumentation and monitoring program.</li> </ul>	<ul style="list-style-type: none"> <li>Creep settlement will occur.</li> </ul>

NF: Considered not technically, cost or schedule feasible to mitigate stability or settlement.





**Table 3: Comparison of Foundation Alternatives for New Bridge on Existing Alignment (Same Length)**

Foundation Type	Rank	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Driven Steel H-Piles	1	<ul style="list-style-type: none"> <li>■ Straightforward construction.</li> <li>■ Can be installed in stages.</li> </ul>	<ul style="list-style-type: none"> <li>■ Potential for “hanging up” on cobbles and potential boulders overlying bedrock prior to reaching design tip elevation.</li> <li>■ Potential for generating EPP in clayey subsoils but likely less than for driven steel tube piles, so that the effects on existing embankments would be reduced.</li> </ul>	<ul style="list-style-type: none"> <li>■ Relative costs lower than for caissons.</li> </ul>	<ul style="list-style-type: none"> <li>■ Potential need to lower existing embankment prior to start of piling to reduce development of excess pore pressures in the cohesive deposit during piling.</li> </ul>
Driven Steel Tube Piles	2	<ul style="list-style-type: none"> <li>■ Straightforward construction.</li> <li>■ Can be installed in stages</li> </ul>	<ul style="list-style-type: none"> <li>■ Potential for “hanging up” on cobbles and potential boulders overlying bedrock prior to reaching design tip elevation.</li> <li>■ Not normally accepted by MTO for integral abutment design.</li> <li>■ Potential for generating EPP in clayey subsoils, which could affect stability of existing/new embankments.</li> </ul>	<ul style="list-style-type: none"> <li>■ Relative costs lower than caissons.</li> </ul>	<ul style="list-style-type: none"> <li>■ Potential need to lower existing embankment prior to start of piling to reduce development of excess pore pressures in the cohesive deposit during piling.</li> <li>■ Greater potential (compared to H-piles) of deflection due to the presence of cobbles and potential boulders overlying bedrock.</li> </ul>
Caissons	3	<ul style="list-style-type: none"> <li>■ Higher axial resistance compared to steel H-piles or tube piles.</li> <li>■ Possible elimination of pile cap and associated excavation.</li> <li>■ Can be installed in stages.</li> </ul>	<ul style="list-style-type: none"> <li>■ Potential for encountering difficulties penetrating the cobbles and possibly boulders compared to piles as caissons would have to extend into the bedrock.</li> <li>■ Not suitable for integral abutment design.</li> <li>■ Requires rock drilling/large socket for seating caissons into bedrock.</li> </ul>	<ul style="list-style-type: none"> <li>■ Relative costs much higher than for steel H-piles, although fewer units are required.</li> </ul>	<ul style="list-style-type: none"> <li>■ Likely able to reach the design tip depth into bedrock.</li> <li>■ Potential for construction problems associated with groundwater inflow into caisson during installation.</li> </ul>



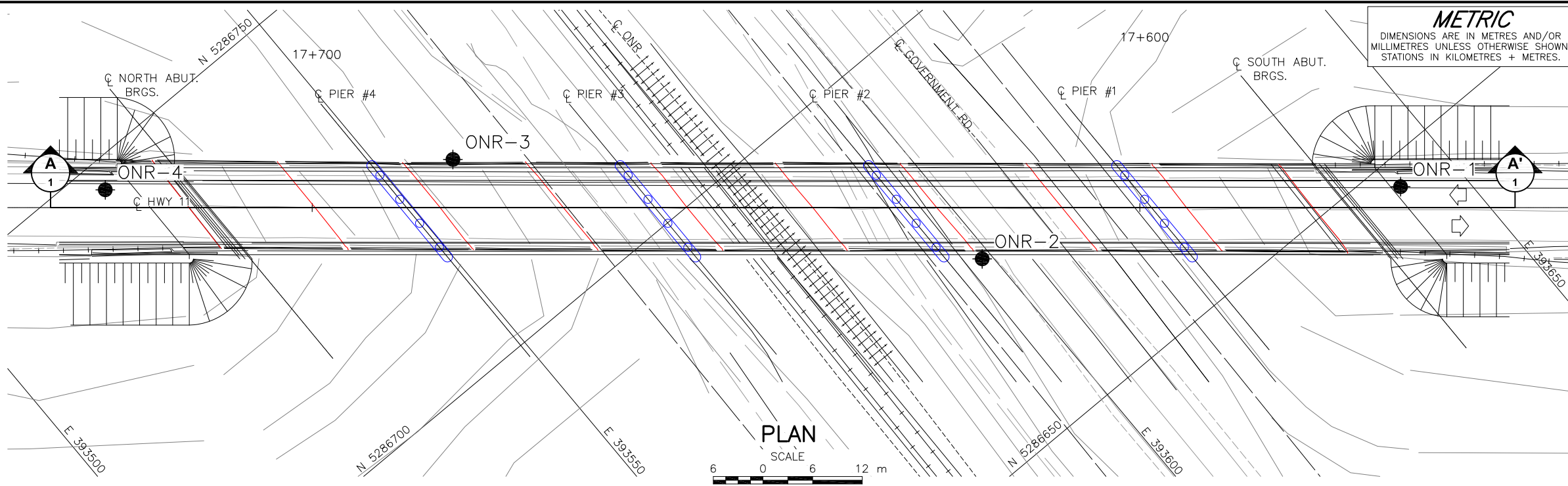


**PRELIMINARY FOUNDATION REPORT, REPLACEMENT OF ONR OVERHEAD  
HIGHWAY 11, SITE NO. 47-037, GWP 5265-01-00**

**Table 3: Comparison of Foundation Alternatives for New Bridge on Existing Alignment (Same Length)**

Shallow Foundations	NF	<ul style="list-style-type: none"><li>■ Can be constructed in stages.</li></ul>	<ul style="list-style-type: none"><li>■ Much lower resistances than are available for deep foundations.</li><li>■ Large foundation footprint required.</li><li>■ Settlement will occur due to consolidation of clay stratum.</li><li>■ Need for excavating to foundation level below the groundwater level.</li></ul>	<ul style="list-style-type: none"><li>■ Lower cost than for deep foundations.</li></ul>	<ul style="list-style-type: none"><li>■ Differential settlement between foundations and adjacent approach embankment.</li><li>■ Inadequate geotechnical axial resistance.</li></ul>
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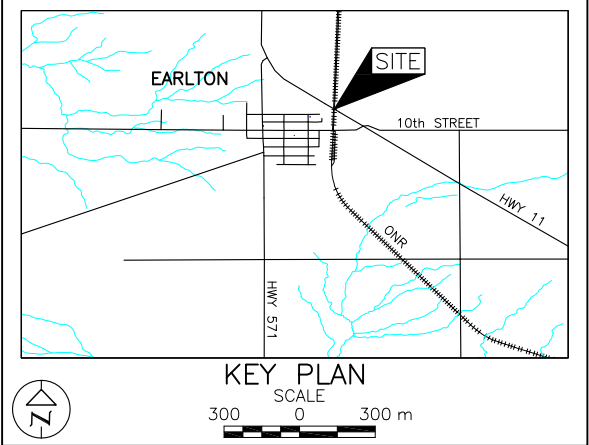


CONT No.  
GWP No. 5265-01-00

HIGHWAY 11  
ONR OVERHEAD BRIDGE  
BOREHOLE LOCATIONS AND  
SOIL STRATA

**Golder Associates Ltd.**  
SUDBURY, ONTARIO, CANADA

SHEET



LEGEND

Borehole

Seal

Piezometer

N  
Standard Penetration Test Value

16  
Blows/0.3m unless otherwise stated  
(Std. Pen. Test, 475 j/blow)

WL in piezometer, measured on AUG 1, 2012

WL upon completion of drilling

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
ONR-1	256.8	5286646.6	393643.3
ONR-2	248.3	5286672.3	393598.9
ONR-3	251.4	5286722.5	393557.5
ONR-4	256.8	5286746.6	393522.9

NOTES

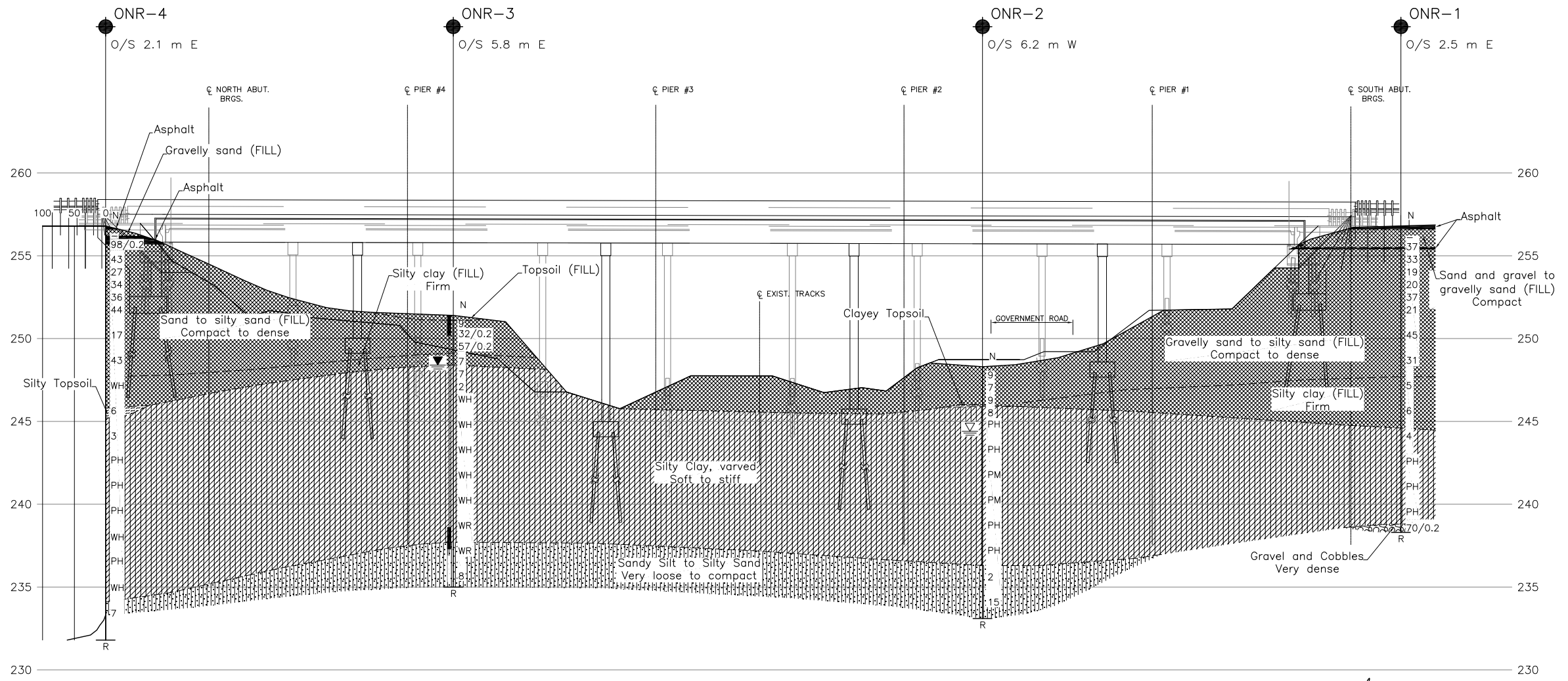
This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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.

REFERENCE

Base plans provided in digital format by URS, drawing file nos. ACAD-ONR\_Alt4\_120727.dwg received JULY 27, 2012.

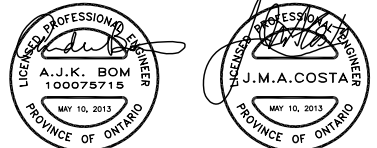


A-A'  
1

PROFILE ALONG CENTRELINE  
HIGHWAY 11

HORIZONTAL SCALE  
0 6 12 m

VERTICAL SCALE  
3 0 3 6 m



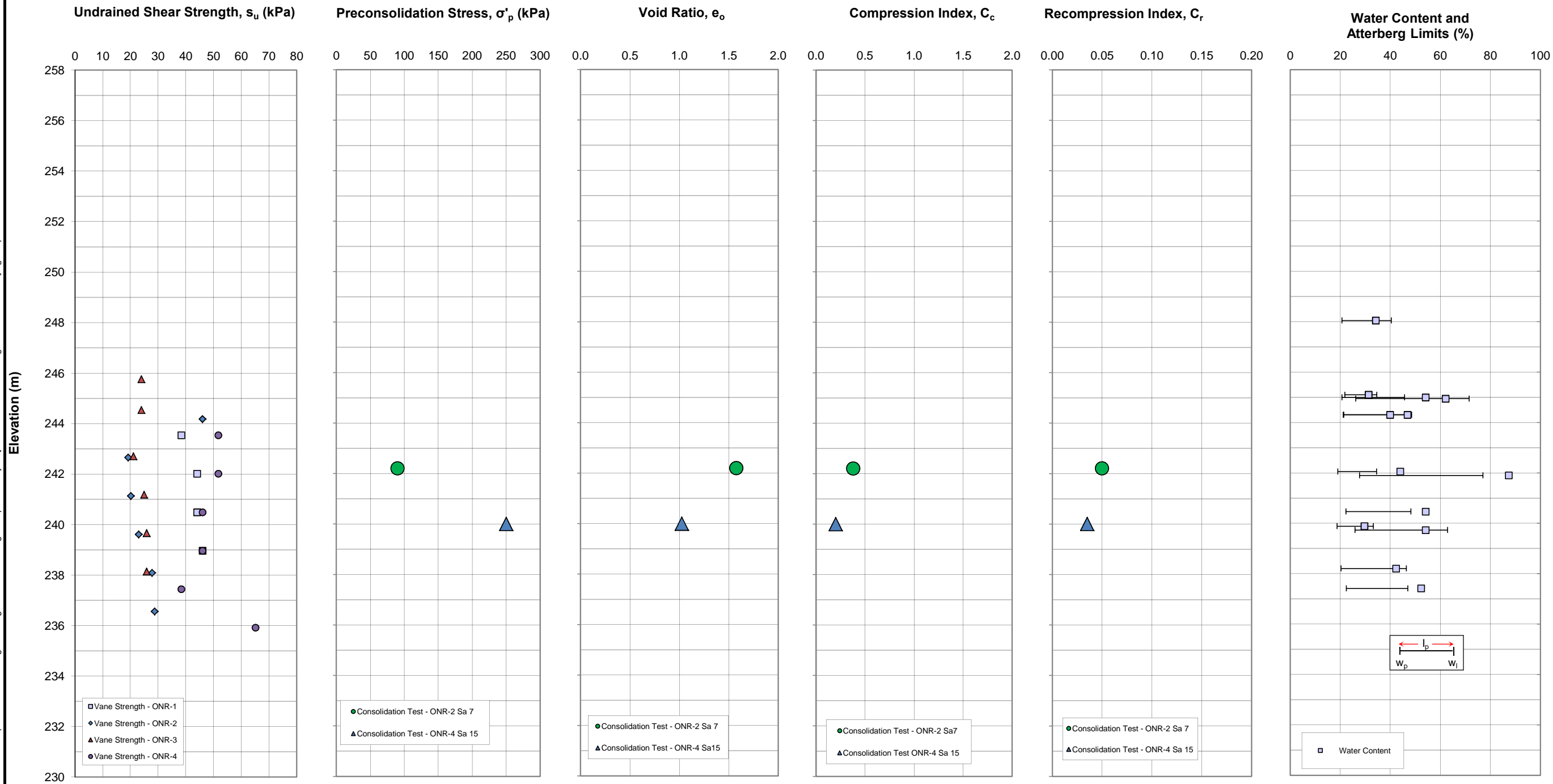
NO.	DATE	BY	REVISION
Geocres No. 31M-99			
HWY. 11		PROJECT NO. 11-1191-0025	DIST.
SUBM'D.	CHKD. AB	DATE: MAY 2013	SITE: 47-037
DRAWN: J.J.L.	CHKD.	APPD: JMAC	DWG. 1



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SUMMARY PLOT OF SELECTED FIELD AND LABORATORY  
RESULTS FOR COHESIVE DEPOSIT  
Highway 11 ONR Overhead Bridge

FIGURE 1



Date: April 2013  
Project No: 11-1191-0025

Prepared By: MH  
Checked By: AB

Golder Associates

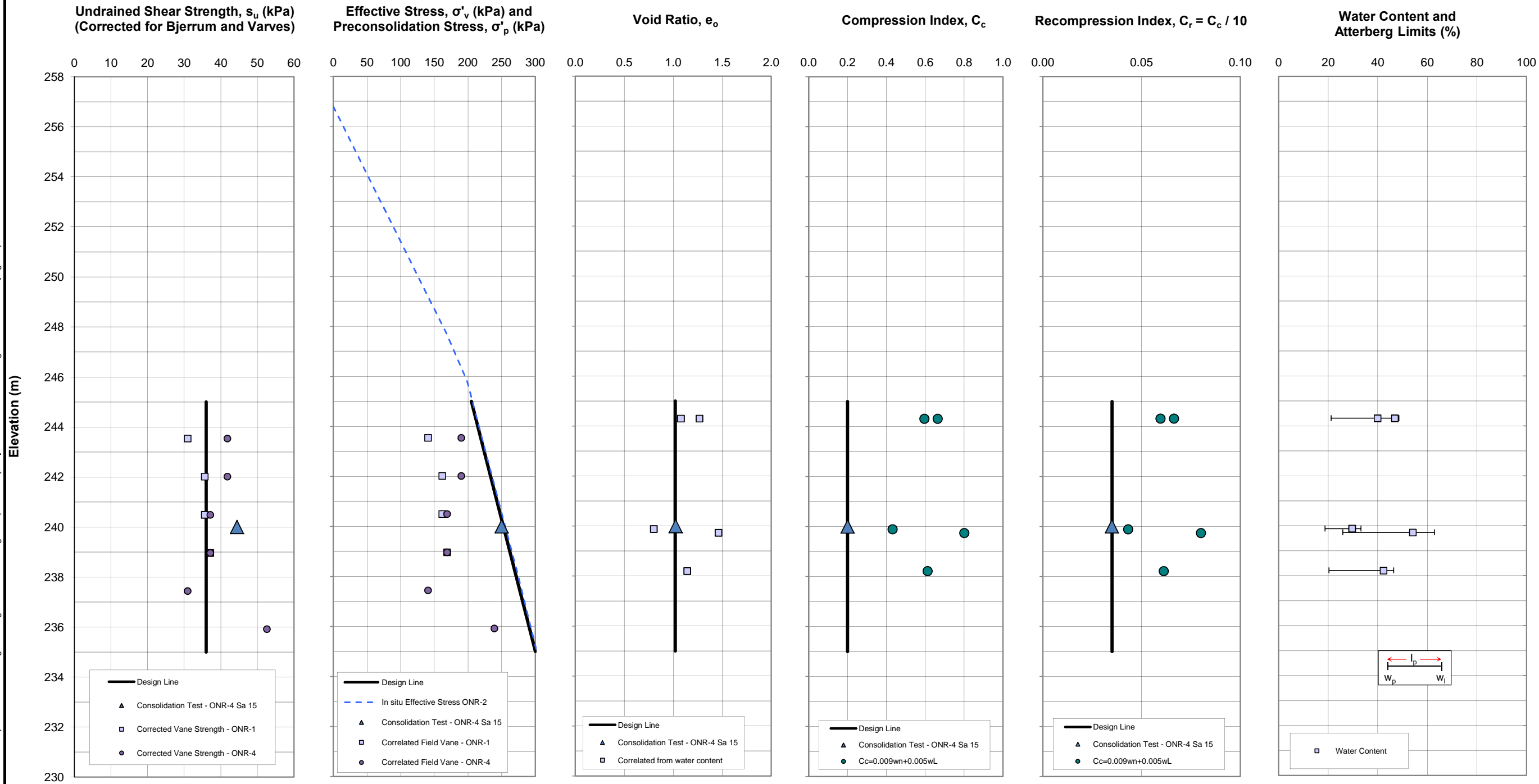




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SUMMARY PLOT OF ENGINEERING PARAMETERS FOR  
COHESIVE DEPOSIT BELOW APPROACH EMBANKMENTS  
(Boreholes ONR-1 and ONR-4)

FIGURE 2

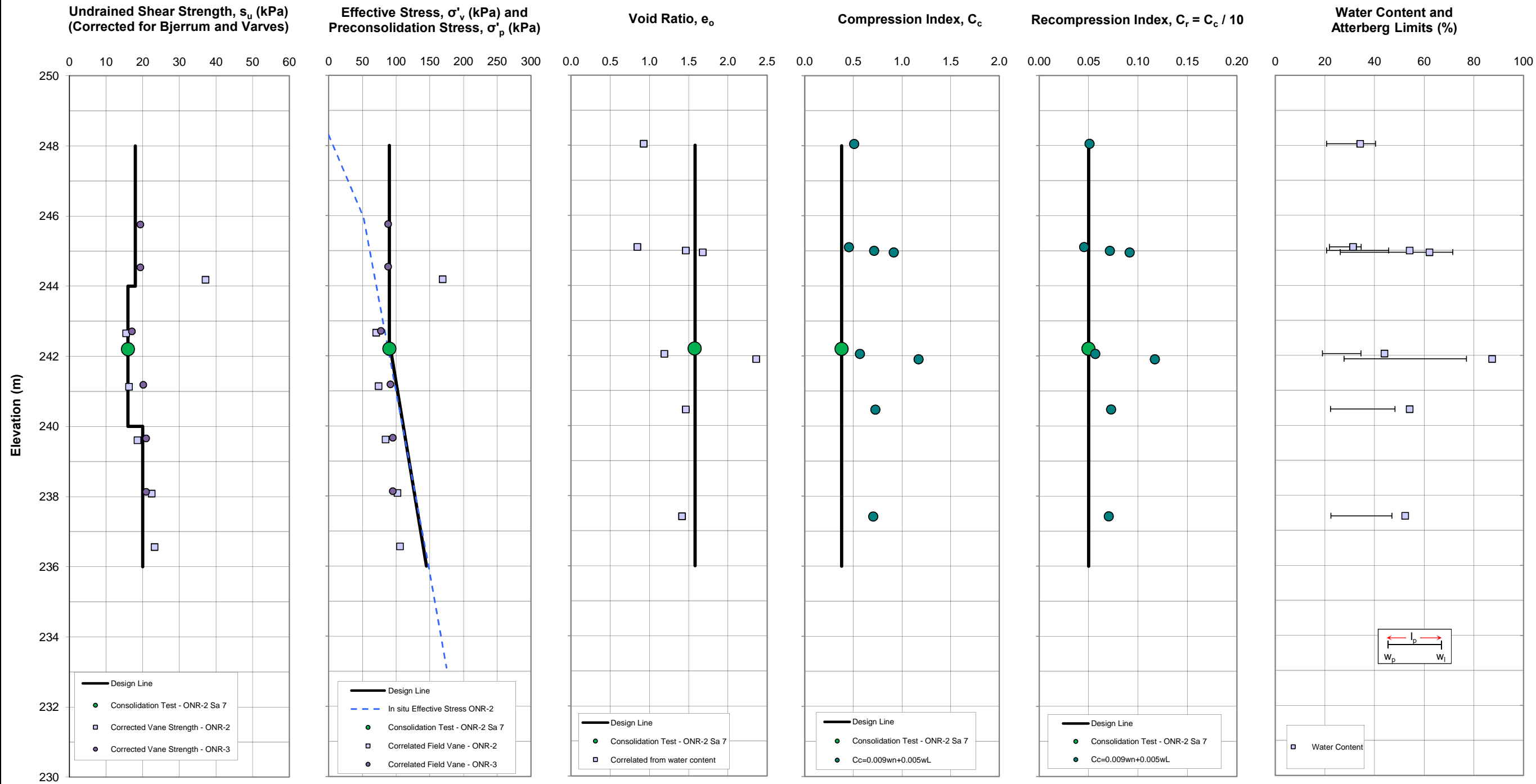




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SUMMARY PLOT OF ENGINEERING PARAMETERS FOR  
COHESIVE DEPOSIT AT PIERS  
(Boreholes ONR-2 and ONR-3)

FIGURE 3



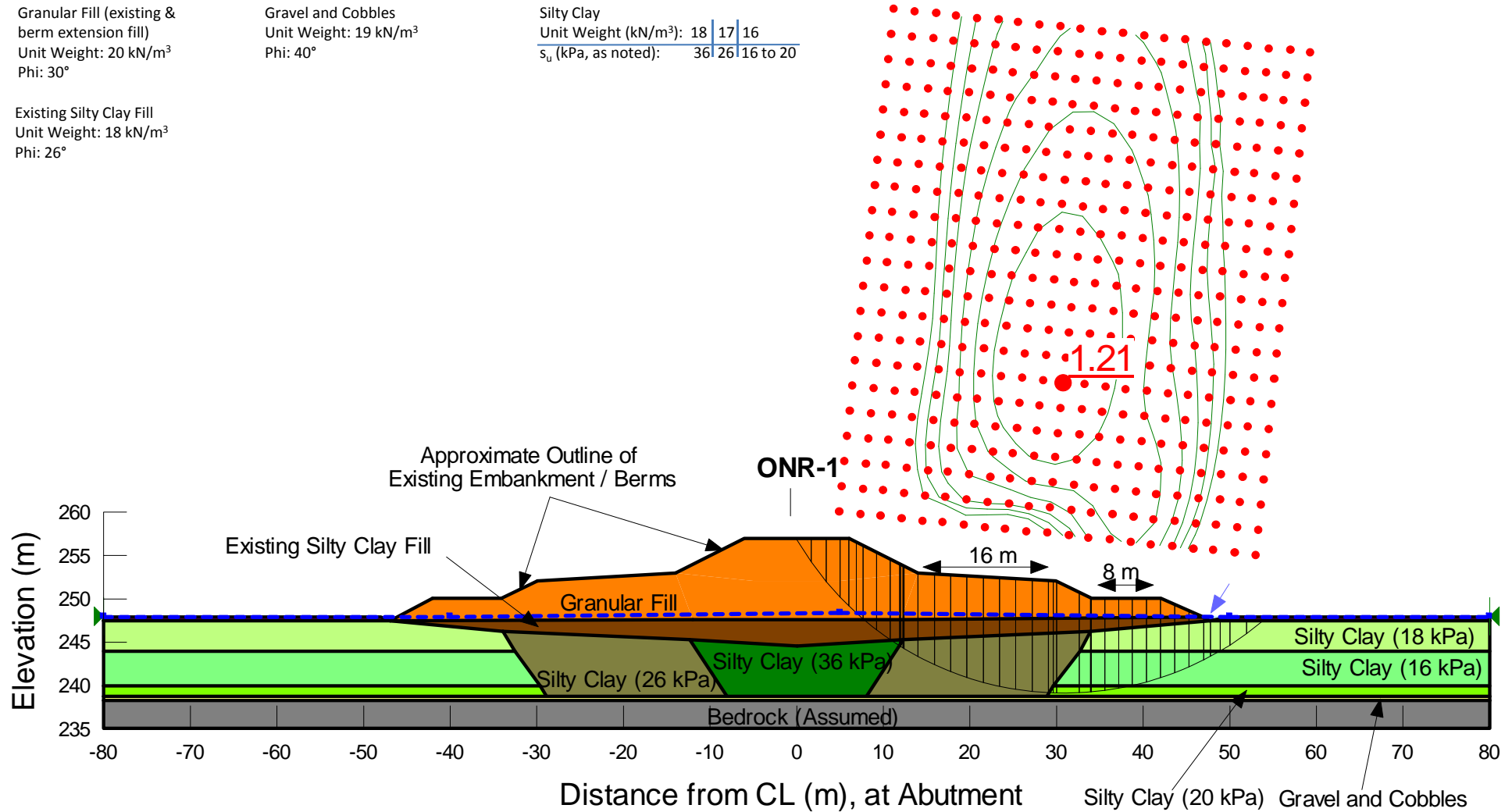


Granular Fill (existing & berm extension fill)  
Unit Weight: 20 kN/m<sup>3</sup>  
Phi: 30°

Gravel and Cobbles  
Unit Weight: 19 kN/m<sup>3</sup>  
Phi: 40°

Silty Clay  
Unit Weight (kN/m<sup>3</sup>): 18 | 17 | 16  
 $s_u$  (kPa, as noted): 36 | 26 | 16 to 20

Existing Silty Clay Fill  
Unit Weight: 18 kN/m<sup>3</sup>  
Phi: 26°




PROJECT		Highway 11 ONR Overhead Bridge			
TITLE		Stability Analysis Existing South Approach Embankment Side Slope			
		PROJECT No. 11-1191-0025		FILE No.	
		DESIGN	AB	APR. 2013	SCALE AS SHOWN
		CADD	--		REV.
		CHECK			
		REVIEW	JMAC	APR. 2013	

Figure 4



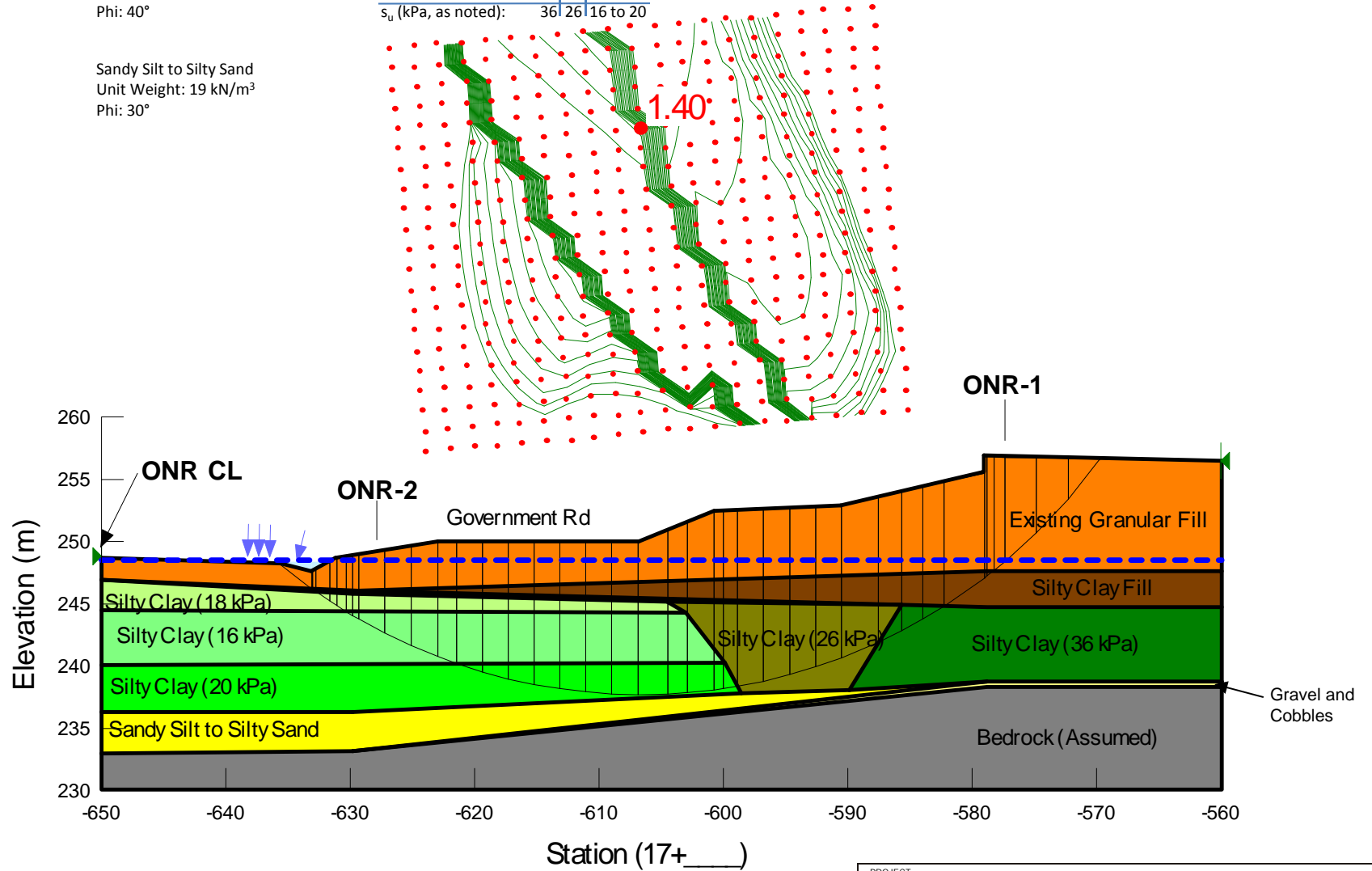
Granular Fill (existing & berm extension fill)  
Unit Weight: 20 kN/m<sup>3</sup>  
Phi: 30°

Existing Silty Clay Fill  
Unit Weight: 18 kN/m<sup>3</sup>  
Phi: 26°

Gravel and Cobbles  
Unit Weight: 19 kN/m<sup>3</sup>  
Phi: 40°

Sandy Silt to Silty Sand  
Unit Weight: 19 kN/m<sup>3</sup>  
Phi: 30°

Silty Clay  
Unit Weight (kN/m<sup>3</sup>): 18 | 17 | 16  
 $s_u$  (kPa, as noted): 36 | 26 | 16 to 20




PROJECT		Highway 11 ONR Overhead Bridge			
TITLE		Stability Analysis Existing South Approach Embankment Front Slope			
		PROJECT No. 11-1191-0025		FILE No.	
		DESIGN	AB	APR. 2013	SCALE AS SHOWN
		CADD	--		REV.
		CHECK			
		REVIEW	JMAC	APR. 2013	

Figure 5



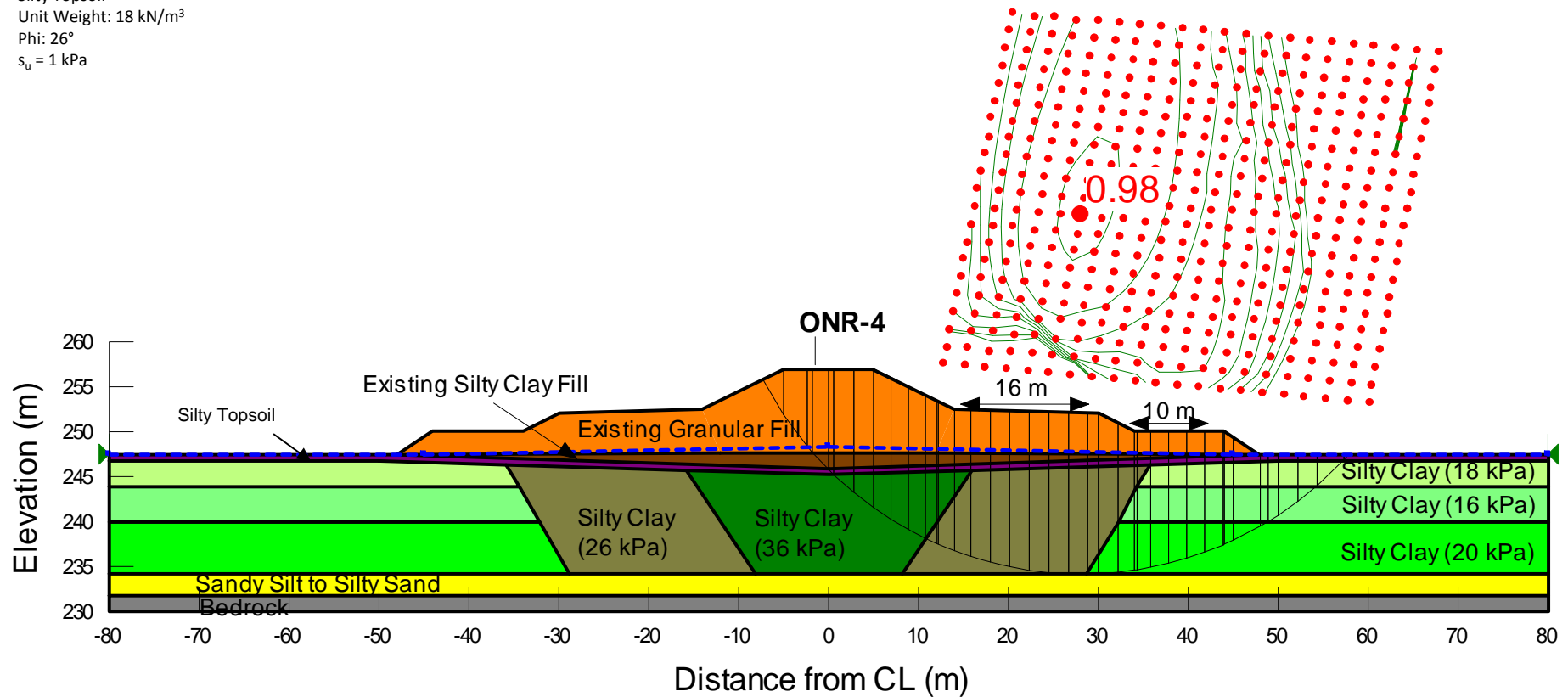
Granular Fill (existing & berm extension fill)  
Unit Weight: 20 kN/m<sup>3</sup>  
Phi: 30°

Sandy Silt to Silty Sand  
Unit Weight: 19 kN/m<sup>3</sup>  
Phi: 30°

Silty Clay  
Unit Weight (kN/m<sup>3</sup>): 18 | 17 | 16  
 $s_u$  (kPa, as noted): 36 | 26 | 16 to 20

Existing Silty Clay Fill  
Unit Weight: 18 kN/m<sup>3</sup>  
Phi: 26°

Silty Topsoil  
Unit Weight: 18 kN/m<sup>3</sup>  
Phi: 26°  
 $s_u$  = 1 kPa




PROJECT		Highway 11 ONR Overhead Bridge			
TITLE		Stability Analysis Existing North Approach Embankment Side Slope			
		PROJECT No. 11-1191-0025		FILE No.	
		DESIGN	AB	APR. 2013	SCALE AS SHOWN
		CADD	--		REV.
		CHECK			
		REVIEW	JMAC	APR. 2013	

Figure 6



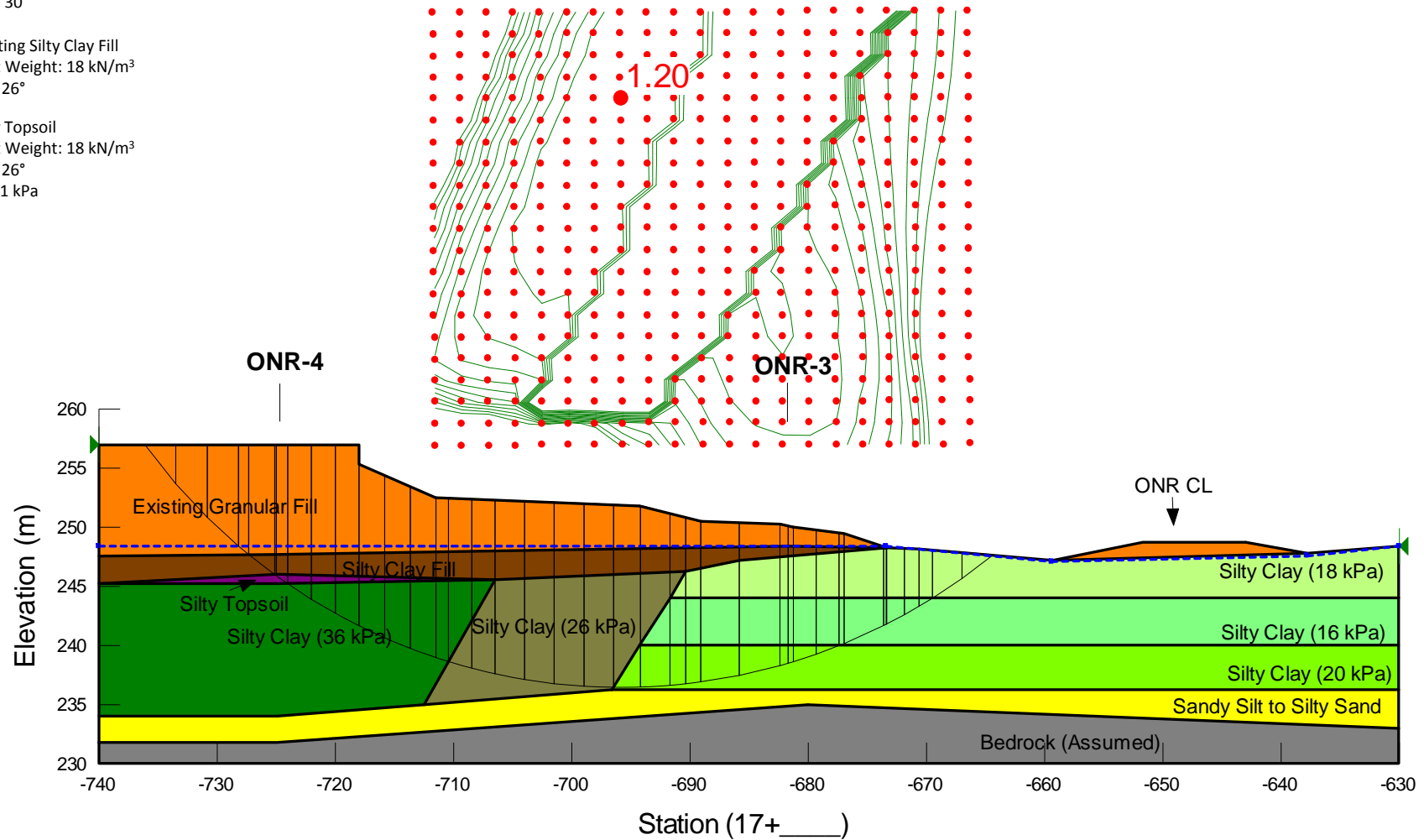
Granular Fill (existing & berm extension fill)  
Unit Weight: 20 kN/m<sup>3</sup>  
Phi: 30°


Sandy Silt to Silty Sand  
Unit Weight: 19 kN/m<sup>3</sup>  
Phi: 30°

Silty Clay  
Unit Weight (kN/m<sup>3</sup>): 18 | 17 | 16  
 $s_u$  (kPa, as noted): 36 | 26 | 16 to 20

Existing Silty Clay Fill  
Unit Weight: 18 kN/m<sup>3</sup>  
Phi: 26°

Silty Topsoil  
Unit Weight: 18 kN/m<sup>3</sup>  
Phi: 26°  
 $s_u$  = 1 kPa



PROJECT		Highway 11 ONR Overhead Bridge			
TITLE		Stability Analysis Existing North Approach Embankment Front Slope			
		PROJECT No. 11-1191-0025		FILE No.	
		DESIGN	AB	APR. 2013	SCALE AS SHOWN
		CADD	--		REV.
		CHECK			
		REVIEW	JMAC	APR. 2013	
					<b>Figure 7</b>



Pavement Structure  
Unit Weight: 22 kN/m<sup>3</sup>  
Phi: 35°

Granular Fill (existing &  
berm extension fill)  
Unit Weight: 20 kN/m<sup>3</sup>  
Phi: 30°

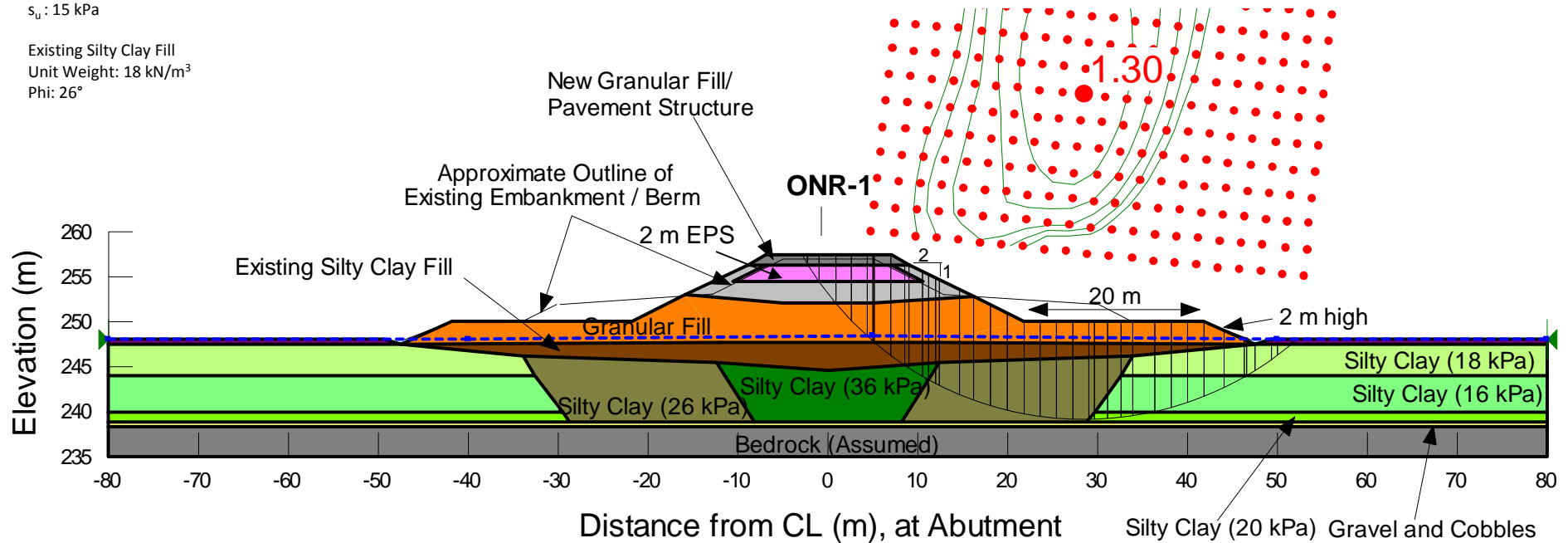
Silty Clay  
Unit Weight (kN/m<sup>3</sup>): 18 | 17 | 16  
 $s_u$  (kPa, as noted): 36 | 26 | 16 to 20


New Granular Fill  
Unit Weight: 21 kN/m<sup>3</sup>  
Phi: 35°

Gravel and Cobbles  
Unit Weight: 19 kN/m<sup>3</sup>  
Phi: 40°

Lightweight Fill (EPS)  
Unit Weight: 0.5 kN/m<sup>3</sup>  
 $s_u$ : 15 kPa

Existing Silty Clay Fill  
Unit Weight: 18 kN/m<sup>3</sup>  
Phi: 26°



PROJECT					Highway 11 ONR Overhead Bridge	
TITLE					Stability Analysis Proposed South Approach Embankment Side Slope	
		PROJECT No. 11-1191-0025			FILE No.	
		DESIGN	AB	APR. 2013	SCALE	AS SHOWN
		CADD	--		REV.	
		CHECK				
		REVIEW	JMAC	APR. 2013	Figure 8	



Pavement Structure  
Unit Weight: 22 kN/m<sup>3</sup>  
Phi: 35°

Lightweight Fill (EPS)  
Unit Weight: 0.5 kN/m<sup>3</sup>  
 $s_u$ : 15 kPa

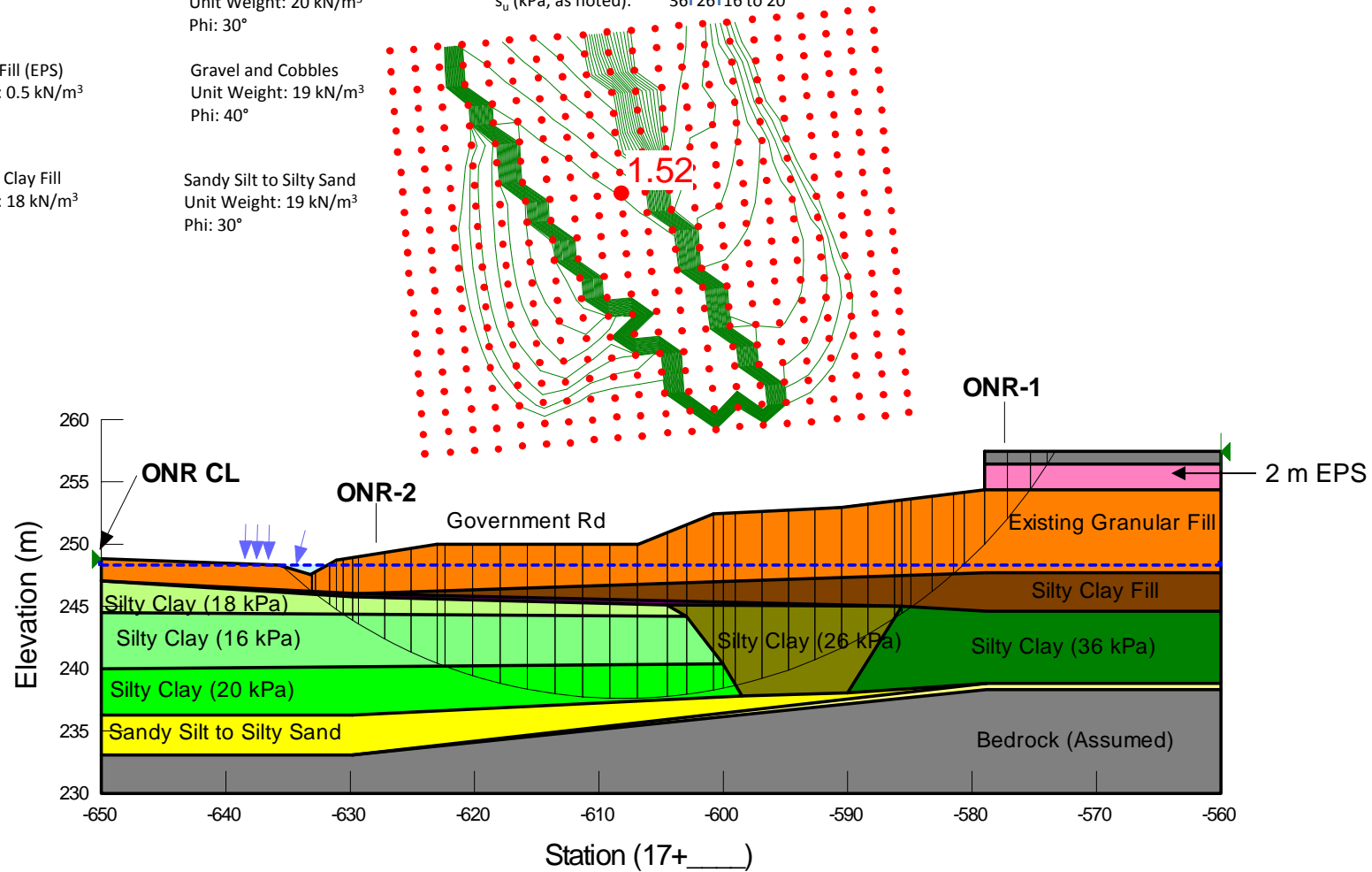
Existing Silty Clay Fill  
Unit Weight: 18 kN/m<sup>3</sup>  
Phi: 26°

Granular Fill (existing &  
berm extension fill)  
Unit Weight: 20 kN/m<sup>3</sup>  
Phi: 30°

Gravel and Cobbles  
Unit Weight: 19 kN/m<sup>3</sup>  
Phi: 40°

Sandy Silt to Silty Sand  
Unit Weight: 19 kN/m<sup>3</sup>  
Phi: 30°

Silty Clay  
Unit Weight (kN/m<sup>3</sup>): 18 | 17 | 16  
 $s_u$  (kPa, as noted): 36 | 26 | 16 to 20



PROJECT		Highway 11 ONR Overhead Bridge			
TITLE		Stability Analysis Proposed South Approach Embankment Front Slope			
		PROJECT No. 11-1191-0025		FILE No.	
DESIGN	AB	APR. 2013	SCALE	AS SHOWN	REV.
CADD	--				
CHECK					
REVIEW	JMAC	APR. 2013			



Figure 9



New Granular Fill  
Unit Weight: 21 kN/m<sup>3</sup>  
Phi: 35°

Granular Fill (existing &  
berm extension fill)  
Unit Weight: 20 kN/m<sup>3</sup>  
Phi: 30°

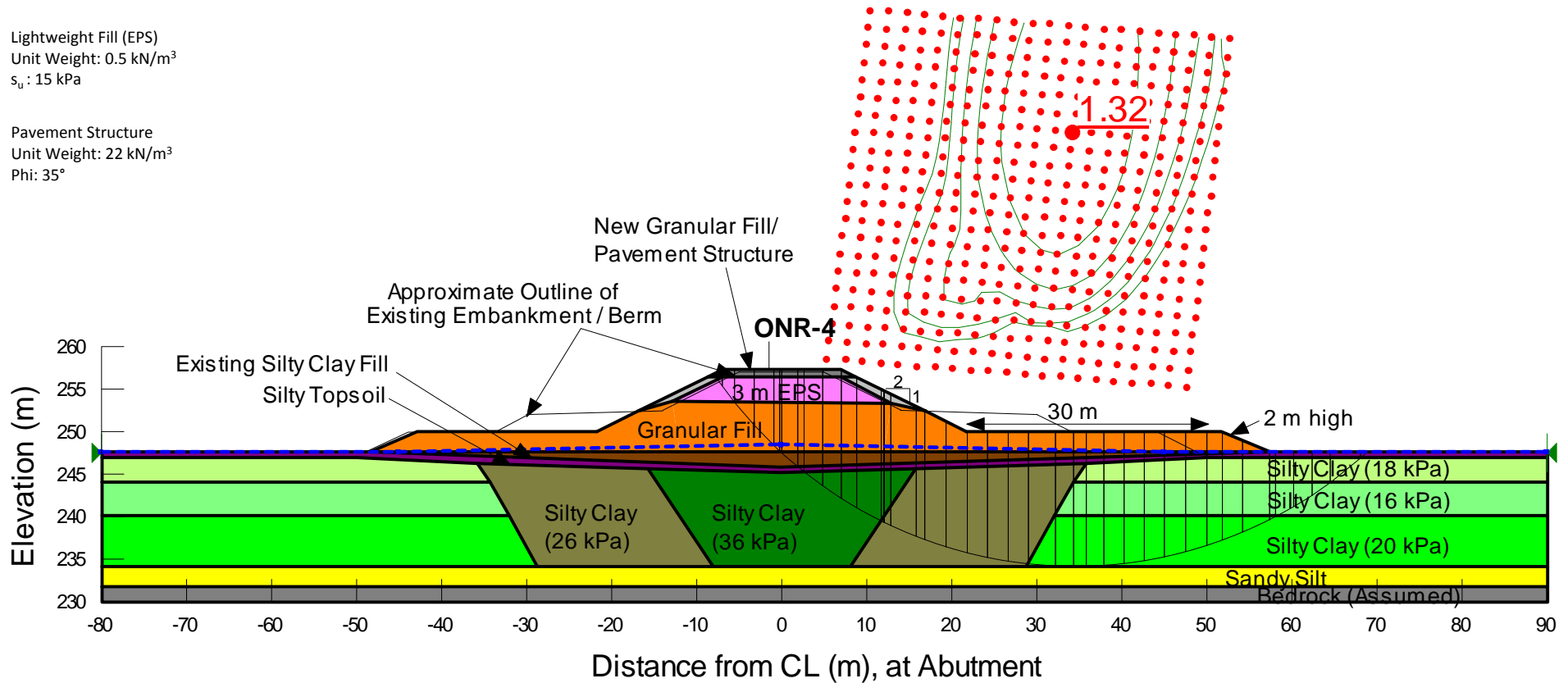
Silty Clay  
Unit Weight (kN/m<sup>3</sup>): 18 | 17 | 16  
 $s_u$  (kPa, as noted): 36 | 26 | 16 to 20

Existing Silty Clay Fill  
Unit Weight: 18 kN/m<sup>3</sup>  
Phi: 26°

Sandy Silt  
Unit Weight: 19 kN/m<sup>3</sup>  
Phi: 30°

Lightweight Fill (EPS)  
Unit Weight: 0.5 kN/m<sup>3</sup>  
 $s_u$ : 15 kPa

Pavement Structure  
Unit Weight: 22 kN/m<sup>3</sup>  
Phi: 35°



PROJECT		Highway 11 ONR Overhead Bridge	
TITLE		Stability Analysis Proposed North Approach Embankment Side Slope EPS 3 m Thick, 30 m Toe Berm	
PROJECT No. 11-1191-0025		FILE No.	
DESIGN	AB	APR. 2013	SCALE AS SHOWN REV.
CADD	--		
CHECK			
REVIEW	JMAC	APR. 2013	



Figure 10a



New Granular Fill  
Unit Weight: 21 kN/m<sup>3</sup>  
Phi: 35°

Granular Fill (existing &  
berm extension fill)  
Unit Weight: 20 kN/m<sup>3</sup>  
Phi: 30°

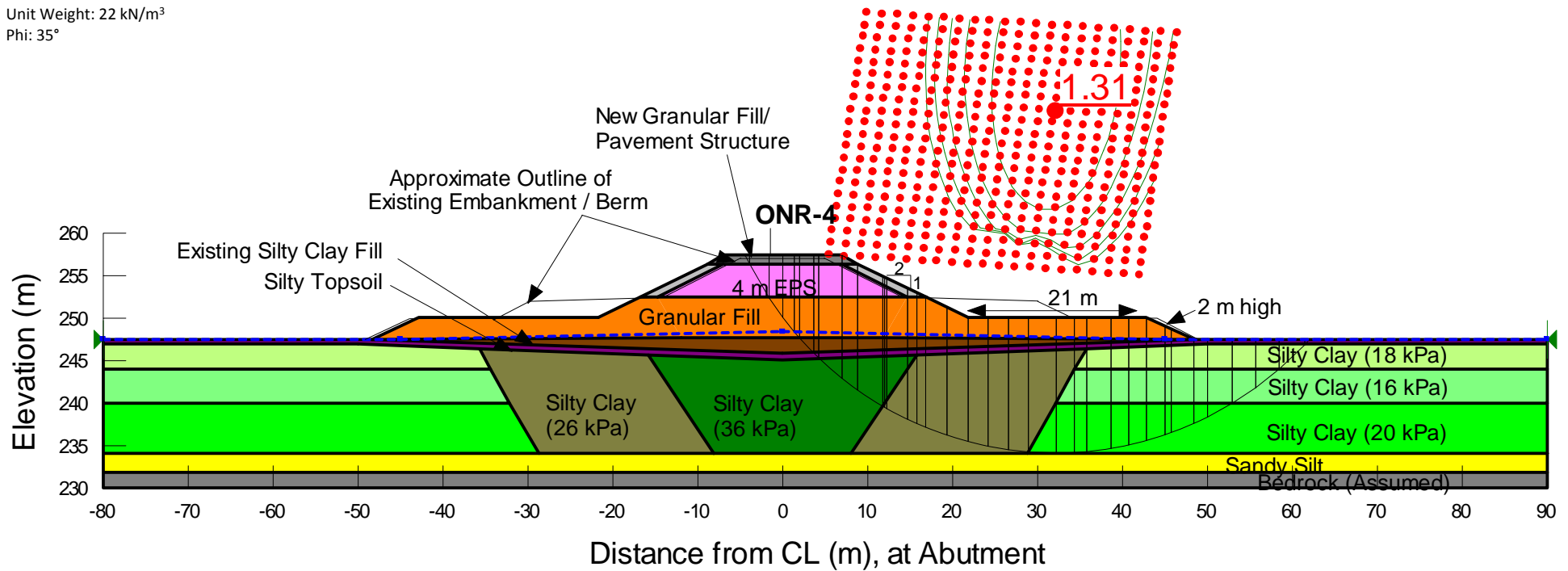
Silty Clay  
Unit Weight (kN/m<sup>3</sup>): 18 | 17 | 16  
 $s_u$  (kPa, as noted): 36 | 26 | 16 to 20

Existing Silty Clay Fill  
Unit Weight: 18 kN/m<sup>3</sup>  
Phi: 26°

Sandy Silt  
Unit Weight: 19 kN/m<sup>3</sup>  
Phi: 30°

Lightweight Fill (EPS)  
Unit Weight: 0.5 kN/m<sup>3</sup>  
 $s_u$ : 15 kPa

Pavement Structure  
Unit Weight: 22 kN/m<sup>3</sup>  
Phi: 35°



PROJECT		Highway 11 ONR Overhead Bridge			
TITLE		Stability Analysis Proposed North Approach Embankment Side Slope EPS 4 m Thick, 21 m Toe Berm			
		PROJECT No. 11-1191-0025	FILE No.		
DESIGN	AB	APR. 2013	SCALE	AS SHOWN	REV.
CADD	--				
CHECK					
REVIEW	JMAC	APR. 2013			



Figure 10b



New Granular Fill  
Unit Weight: 21 kN/m<sup>3</sup>  
Phi: 35°

Granular Fill (existing)  
Unit Weight: 20 kN/m<sup>3</sup>  
Phi: 30°

Silty Clay  
Unit Weight (kN/m<sup>3</sup>): 18 | 17 | 16  
 $s_u$  (kPa, as noted): 36 | 26 | 16 to 20

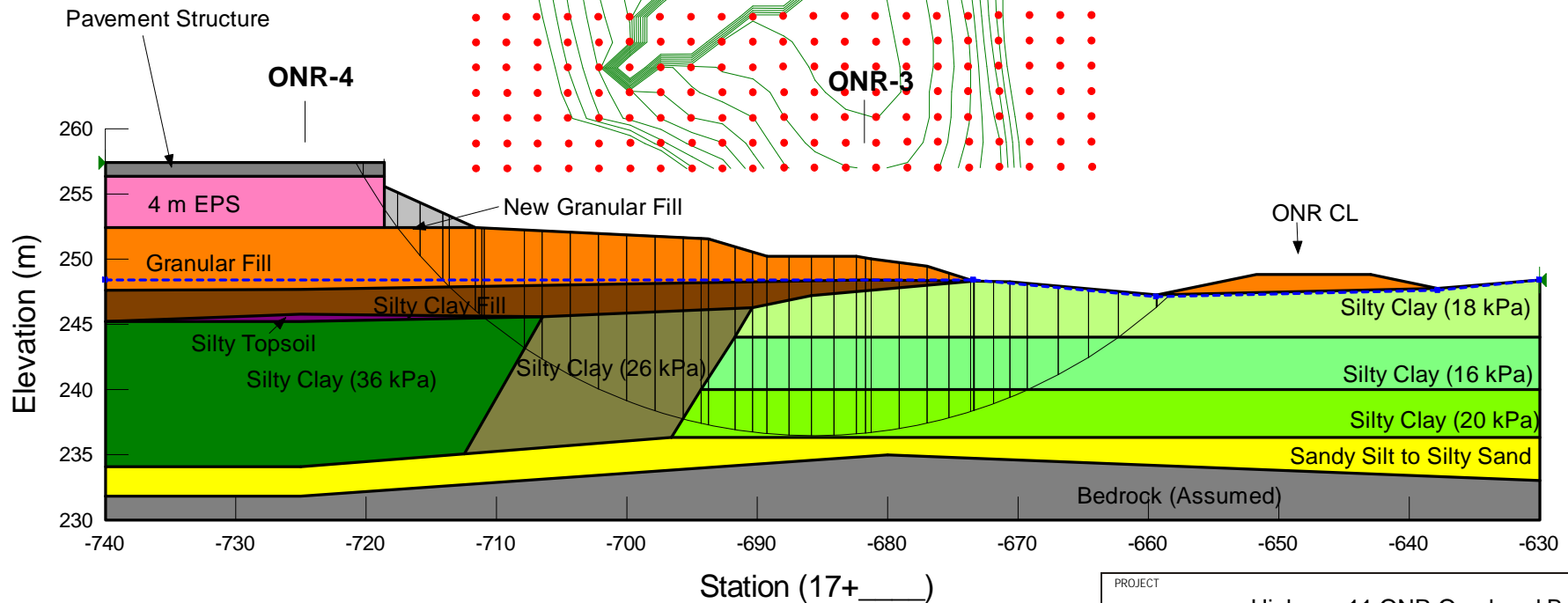
Existing Silty Clay Fill  
Unit Weight: 18 kN/m<sup>3</sup>  
Phi: 26°

Sandy Silt to Silty Sand  
Unit Weight: 19 kN/m<sup>3</sup>  
Phi: 30°

Lightweight Fill (EPS)  
Unit Weight: 0.5 kN/m<sup>3</sup>  
 $s_u$ : 15 kPa

Silty Topsoil  
Unit Weight: 18 kN/m<sup>3</sup>  
Phi: 26°  
 $s_u$  = 1 kPa

Pavement Structure  
Unit Weight: 22 kN/m<sup>3</sup>  
Phi: 35°



PROJECT		Highway 11 ONR Overhead Bridge			
TITLE		Stability Analysis Proposed North Approach Embankment Front Slope EPS 4 m thick			
		PROJECT No. 11-1191-0025	FILE No.		
DESIGN	AB	APR. 2013	SCALE	AS SHOWN	REV.
CADD	--		Figure 11		
CHECK					
REVIEW	JMAC	APR. 2013			







# APPENDIX A

## Record of Boreholes and Drillholes





## 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

### 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.)

#### 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.).

<b>PH:</b>	Sampler advanced by hydraulic pressure
<b>PM:</b>	Sampler advanced by manual pressure
<b>WH:</b>	Sampler advanced by static weight of hammer
<b>WR:</b>	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 cm<sup>2</sup> 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.

### III. SOIL DESCRIPTION

#### (a) Cohesionless Soils

Density Index	N
Relative Density	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

#### (b) Cohesive Soils 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

### 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 <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
$D_R$	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 <sub>4</sub>	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
$\gamma$	unit weight

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

### V. MINOR SOIL CONSTITUENTS

Percent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (cohesionless) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand





## LIST OF SYMBOLS

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

### I. GENERAL

$\pi$	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

### II. STRESS AND STRAIN

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\varepsilon$	linear strain
$\varepsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	Poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	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
$\gamma'$	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
$w_l$ or LL	liquid limit
$w_p$ or PL	plastic limit
$I_p$ or PI	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
$C_\alpha$	secondary compression index
$m_v$	coefficient of volume change
$C_v$	coefficient of consolidation (vertical direction)
$C_h$	coefficient of consolidation (horizontal direction)
$T_v$	time factor (vertical direction)
U	degree of consolidation
$\sigma'_p$	pre-consolidation stress
OCR	over-consolidation ratio = $\sigma'_p / \sigma'_{vo}$

#### (d) Shear Strength

$\tau_p, \tau_r$	peak and residual shear strength
$\phi'$	effective angle of internal friction
$\delta$	angle of interface friction
$\mu$	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

\* Density symbol is  $\rho$ . Unit weight symbol is  $\gamma$  where  $\gamma = \rho g$  (i.e. mass density multiplied by acceleration due to gravity)

Notes: 1  
2

$$\tau = c' + \sigma' \tan \phi'$$

$$\text{shear strength} = (\text{compressive strength})/2$$





## LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

### WEATHERINGS 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 and structure are preserved.

### BEDDING THICKNESS

<u>Description</u>	<u>Bedding Plane Spacing</u>
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
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	Less than 6 mm

### JOINT OR FOLIATION SPACING

<u>Description</u>	<u>Spacing</u>
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

### GRAIN SIZE

<u>Term</u>	<u>Size*</u>
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: \* Grains greater than 60 microns diameter are visible to the naked eye.

### CORE CONDITION

#### Total Core Recovery (TCR)

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 varied 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 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 abbreviation 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

JN Joint	PL Planar
FLT Fault	CU Curved
SH Shear	UN Undulating
VN Vein	IR Irregular
FR Fracture	K Slickensided
SY Stylolite	PO Polished
BD Bedding	SM Smooth
FO Foliation	SR Slightly Rough
CO Contact	RO Rough
AXJ Axial Joint	VR Very Rough
KV Karstic Void	
MB Mechanical Break	



MSUD-MTO 001 11-1191-0025.GPJ GAL-MISS.GDT 22/10/12 DATA INPUT:





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

MSUD-MTO 001 11-1191-0025.GPJ GAL-MISS.GDT 22/10/12 DATA INPUT:



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

SUD-MTO 001 11-1191-0025.GPJ GAL-MISS.GDT 22/10/12 DATA INPUT:



PROJECT <u>11-1191-0025</u>		<b>RECORD OF BOREHOLE No ONR-2</b>				2 OF 2 <b>METRIC</b>									
G.W.P. <u>5265-01-00</u>		LOCATION <u>N 5286672.3; E 393598.9</u>				ORIGINATED BY <u>EHS</u>									
DIST <u>          </u> HWY <u>11</u>		BOREHOLE TYPE <u>108 mm I.D. HOLLOW STEM AUGERS</u>				COMPILED BY <u>JJL</u>									
DATUM <u>GEODETIC</u>		DATE <u>JULY 20, 2012</u>				CHECKED BY <u>AB</u>									
SOIL PROFILE		SAMPLES				DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT		UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa 20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED		W <sub>p</sub>	W	W <sub>L</sub>	WATER CONTENT (%)	γ	GR SA SI CL
233.1	--- CONTINUED FROM PREVIOUS PAGE ---														
15.2	END OF BOREHOLE AUGER REFUSAL  Note:  1. Water level at a depth of 3.9 m below ground surface (Elev. 244.4 m) upon completion of drilling and rising slowly.														

SUD-MTO 001 11-1191-0025.GPJ GAL-MISS.GDT 22/10/12 DATA INPUT:



PROJECT		11-1191-0025		RECORD OF BOREHOLE No ONR-3		1 OF 2 METRIC														
G.W.P.		5265-01-00		LOCATION		N 5286722.5; E 393557.5														
DIST		HWY 11		BOREHOLE TYPE		108 mm I.D. HOLLOW STEM AUGERS														
DATUM		GEODETIC		DATE		JULY 23 and 24, 2012														
ORIGINATED BY		EHS		COMPILED BY		JJL														
CHECKED BY		AB																		
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa			WATER CONTENT (%)			γ			GR SA SI CL		
251.4	0.0	GROUND SURFACE							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED			W <sub>p</sub> — W — W <sub>L</sub> 20 40 60			kN/m <sup>3</sup>					
251.1	0.3	Topsoil (FILL) Brown Moist		1	SS	9		251												
		Sand, some gravel, some silt, some clay with cobbles at 1.1 m and 1.6 m depths (FILL) Brown to grey Moist		2	SS	32/0.2		250												15 54 18 13
				3	SS	57/0.2														
249.1	2.3	Silty clay, organic pockets (FILL) Firm Brown Moist		4	SS	7		249												
248.4	3.0	SILTY CLAY, varved Soft to firm Grey Wet		5	SS	7		248												
				6	SS	2		247												
				7	SS	WH														
								246												
				8	SS	WH		245												
								244												
				9	SS	WH		243												
								242												
				10	SS	WH		241												
								240												
				11	SS	WH		239												
								238												
				12	SS	WR		237												
237.7	13.7	Sandy SILT, with clay, trace gravel, clay seams Very loose to loose Grey Wet		13	SS	WR														4 29 37 30

SUD-MTO 001 11-1191-0025.GPJ GAL-MISS.GDT 22/10/12 DATA INPUT:

Continued Next Page

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



PROJECT		RECORD OF BOREHOLE No ONR-3				2 OF 2 METRIC											
11-1191-0025																	
G.W.P. 5265-01-00		LOCATION N 5286722.5; E 393557.5				ORIGINATED BY EHS											
DIST _____ HWY 11		BOREHOLE TYPE 108 mm I.D. HOLLOW STEM AUGERS				COMPILED BY JJL											
DATUM GEODETIC		DATE JULY 23 and 24, 2012				CHECKED BY AB											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	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 ---						20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W <sub>p</sub> W W <sub>L</sub> 20 40 60					
235.0	Sandy SILT, with clay, trace gravel, clay seams Very loose to loose Grey Wet		14	SS	8		236										
16.4	END OF BOREHOLE AUGER REFUSAL  Note:  1. Water level in piezometer at a depth of 7.0 m (Elev. 244.4 m) on August 1, 2012.  2. Water level in piezometer at a depth of 3.0 m below ground surface (Elev. 248.4 m) on September 27, 2012.						235										



PROJECT <u>11-1191-0025</u>		<b>RECORD OF BOREHOLE</b>		<b>No ONR-4</b>	1 OF 2	<b>METRIC</b>
G.W.P. <u>5265-01-00</u>	LOCATION <u>N 5286746.6; E 393522.9</u>	ORIGINATED BY <u>EHS</u>				
DIST <u>          </u> HWY <u>11</u>	BOREHOLE TYPE <u>108 mm I.D. HOLLOW STEM AUGERS</u>	COMPILED BY <u>JJL</u>				
DATUM <u>GEODETIC</u>	DATE <u>JULY 18 and 19, 2012</u>	CHECKED BY <u>AB</u>				

[illegible]

Continued Next Page

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

SUD-MTO 001 11-1191-0025.GPJ GAL-MISS.GDT 22/10/12 DATA INPUT:



PROJECT		11-1191-0025		RECORD OF BOREHOLE No ONR-4		2 OF 2 METRIC						
G.W.P.		5265-01-00		LOCATION		N 5286746.6; E 393522.9						
DIST		HWY 11		BOREHOLE TYPE		108 mm I.D. HOLLOW STEM AUGERS						
DATUM		GEODETIC		DATE		JULY 18 and 19, 2012						
ORIGINATED BY		EHS		COMPILED BY		JJL						
CHECKED BY		AB										
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER			TYPE	"N" VALUES					
--- CONTINUED FROM PREVIOUS PAGE ---												
	SILTY CLAY, varved Firm to stiff Grey Wet		14	TO	PH							
	Light grey clayey silt laminae 10 mm thick		15	TO	PH							
	Dark grey clay laminae 20 mm thick											
			16	SS	WH							
			17	TO	PH							
			18	SS	WH							
234.1												
22.7	Sandy SILT to Silty SAND, some gravel, trace to some clay		19	SS	7							
233.3	Loose Grey Wet											
23.5	START OF DCPT											
231.8												
25.0	END OF DCPT REFUSAL TO FURTHER PENETRATION (HAMMER BOUNCING) END OF BOREHOLE											
Note: 1. Water level not measured upon completion of drilling.												

SUD-MTO 001 11-1191-0025.GPJ GAL-MISS.GDT 22/10/12 DATA INPUT:

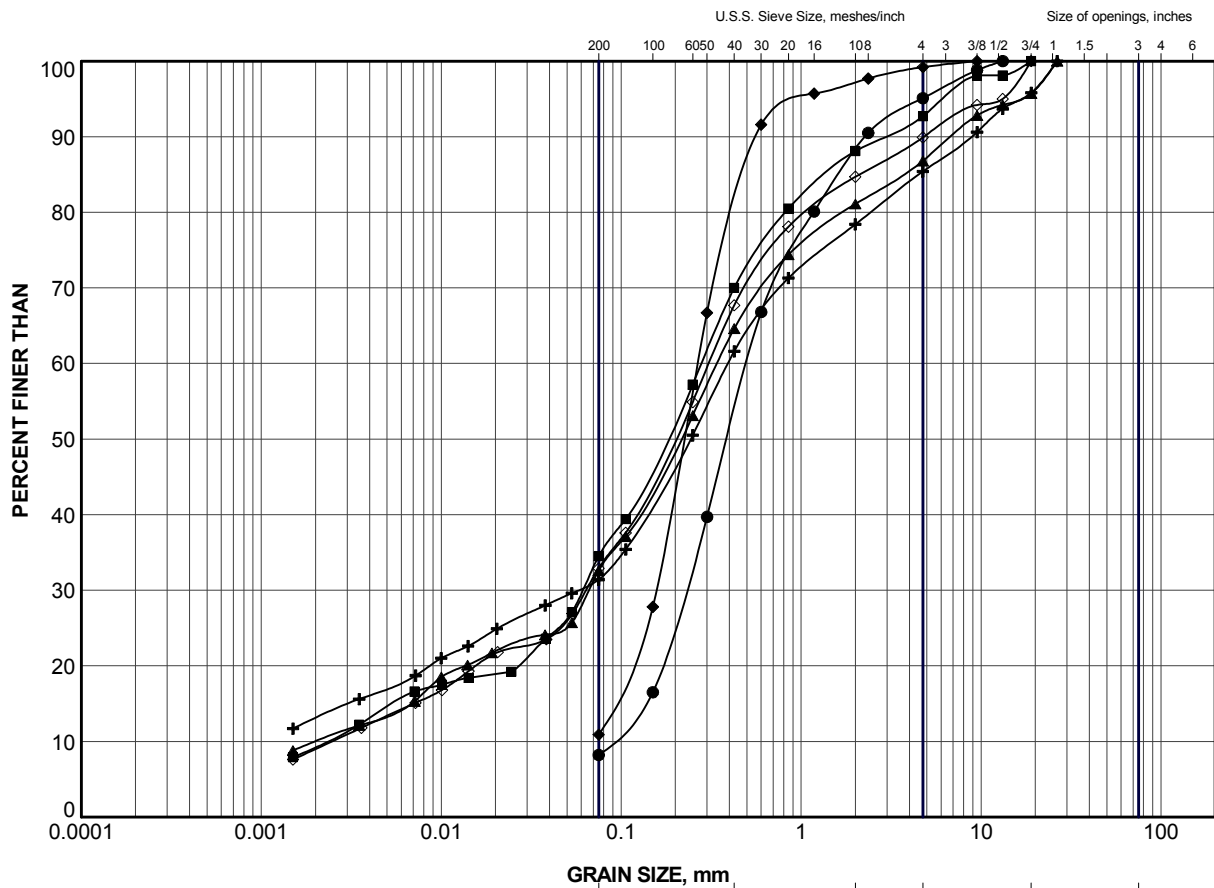




# APPENDIX B

## Laboratory Test Results






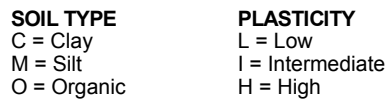
CLAY AND SILT	GRAVEL SIZE, mm					Cobble Size
	fine	medium	coarse	fine	coarse	
	SAND SIZE			GRAVEL SIZE		

### LEGEND


SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	ONR-1	4	254.2
■	ONR-1	7	251.9
▲	ONR-2	3	246.5
+	ONR-3	2	250.5
◆	ONR-4	3	255.0
◇	ONR-4	7	251.9

PROJECT					
HIGHWAY 11 ONR OVERHEAD BRIDGE					
TITLE					
GRAIN SIZE DISTRIBUTION SILTY SAND TO SAND (FILL)					
PROJECT No.		11-1191-0025		FILE No. 11-1191-0025.GPJ	
DRAWN	JJL	Oct 2012	SCALE	N/A	REV.
CHECK	AB	Oct 2012			
APPR	JMAC	Oct 2012			
 <b>Golder Associates</b> SUDBURY, ONTARIO			<b>FIGURE B1</b>		

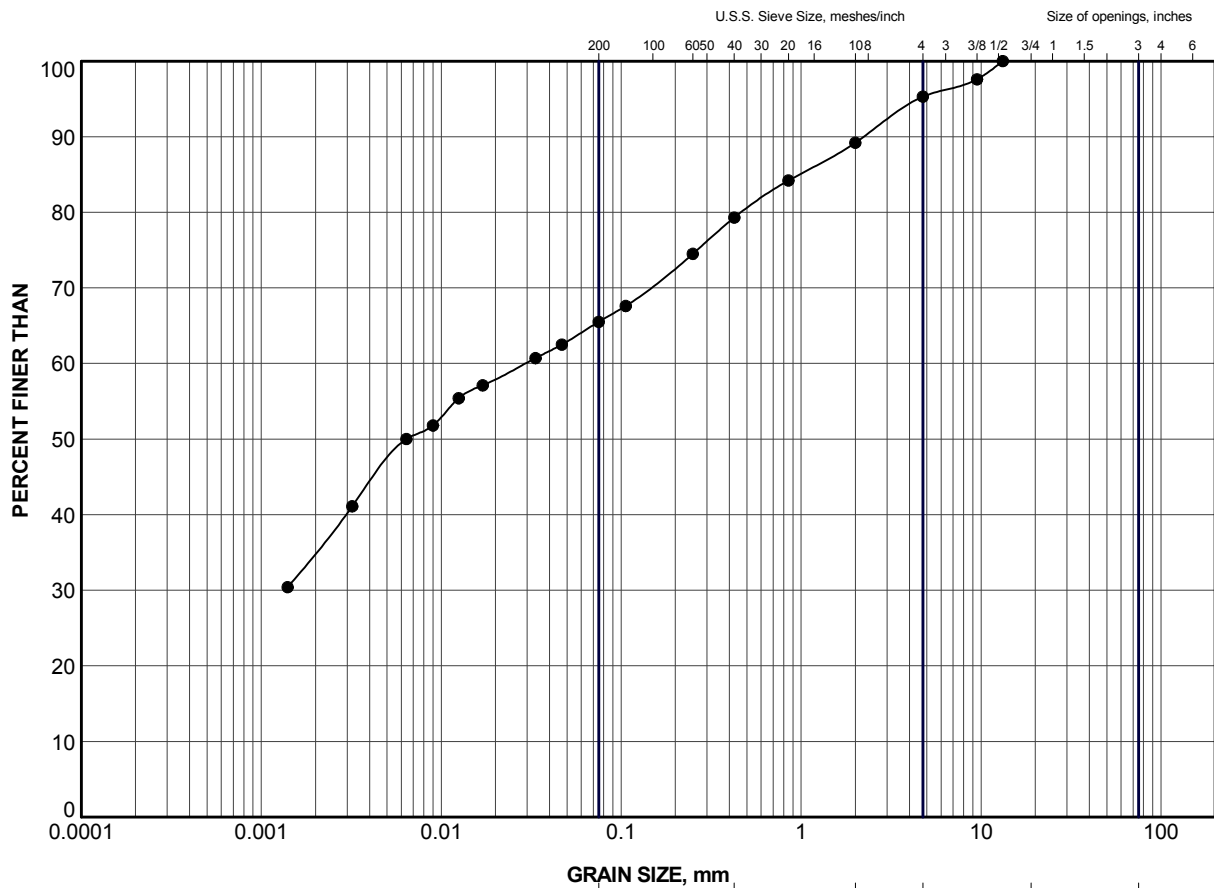




SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	ONR-1	10	37	17	20

PROJECT										HIGHWAY 11 ONR OVERHEAD BRIDGE																			
TITLE																				PLASTICITY CHART SILTY CLAY (FILL)									
 <b>Golder Associates</b> SUDBURY ONTARIO										PROJECT No.					11-1191-0025					FILE No.					11-1191-0025.GPJ				
										DRAWN		JUL		Oct 2012			SCALE		N/A			REV.							
										CHECK		AB		Oct 2012			<b>FIGURE B2</b>												
										APPR		JMAC		Oct 2012															





#### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	ONR-1	10	247.4

PROJECT

HIGHWAY 11  
ONR OVERHEAD BRIDGE

TITLE

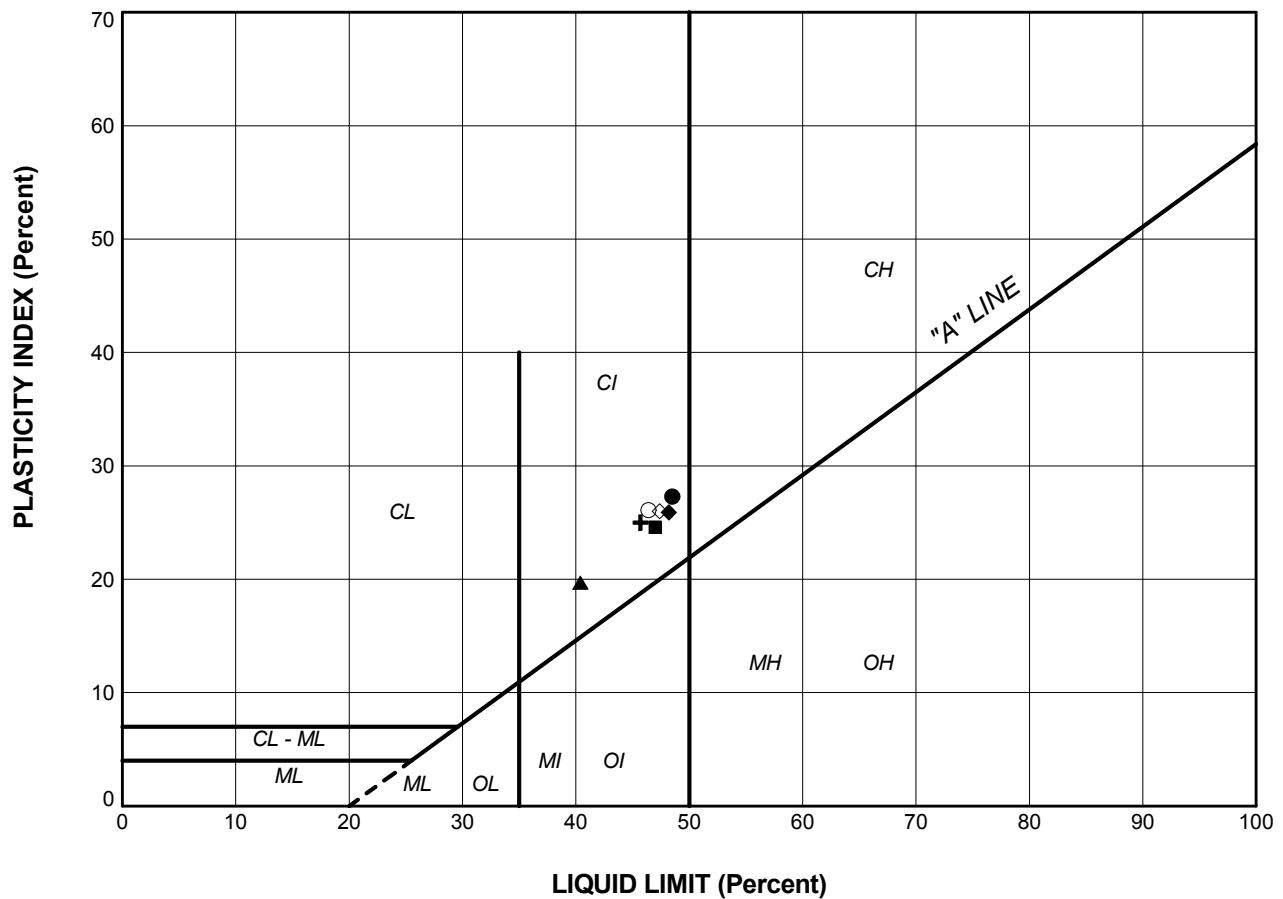
**GRAIN SIZE DISTRIBUTION**  
SILTY CLAY (FILL)



**Golder Associates**  
SUDBURY, ONTARIO

PROJECT No.		11-1191-0025	FILE No.		11-1191-0025.GPJ
DRAWN	JJL	Oct 2012	SCALE	N/A	REV.
CHECK	AB	Oct 2012	<b>FIGURE B3</b>		
APPR	JMAC	Oct 2012			



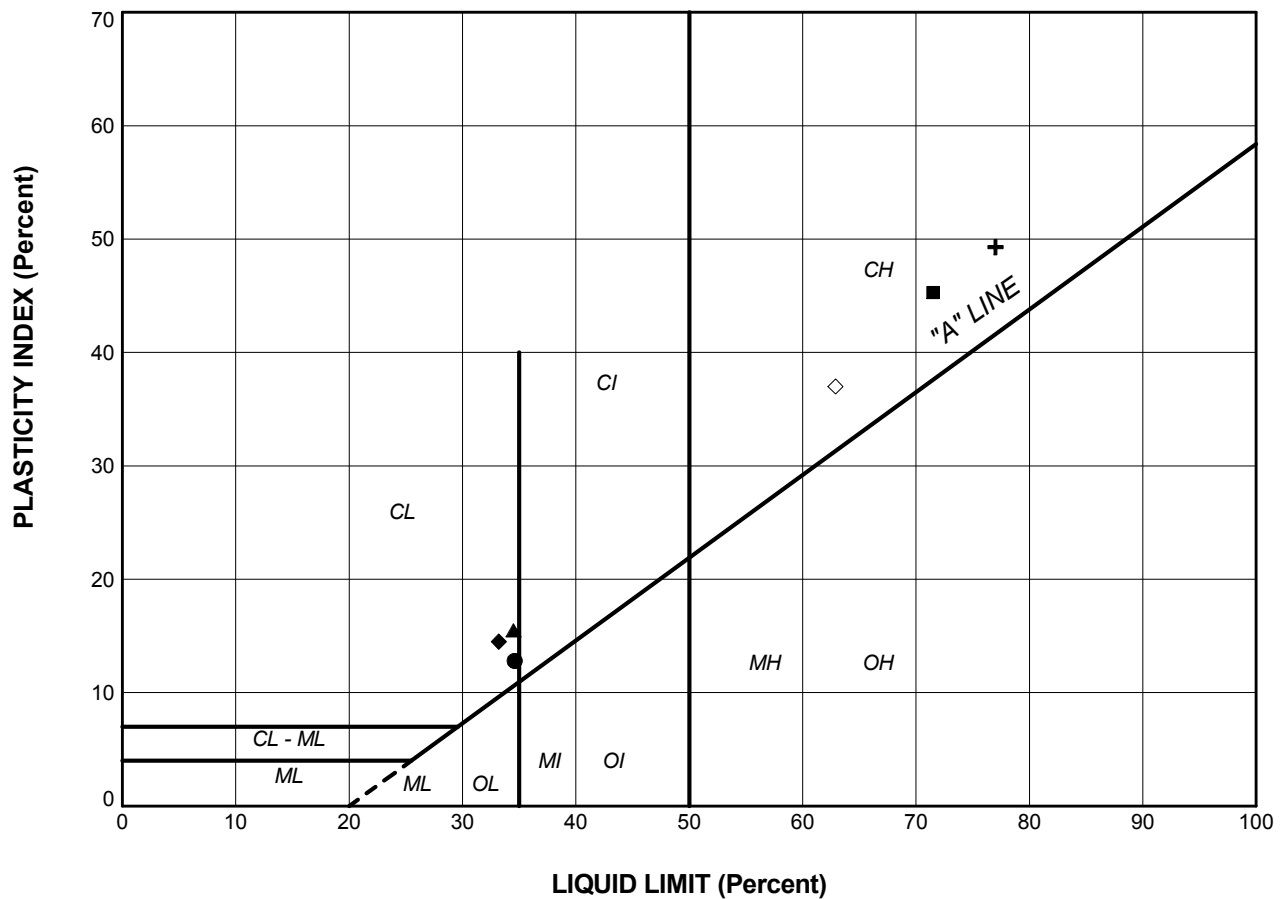


<b>LEGEND</b>					
SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	ONR-1	12	49	21	27
■	ONR-2	10	47	22	25
▲	ONR-3	5	40	21	20
+	ONR-3	8	46	21	25
◆	ONR-3	11	48	22	26
◇	ONR-4	12	47	21	26
○	ONR-4	16	46	20	26

PROJECT					
HIGHWAY 11 ONR OVERHEAD BRIDGE					
TITLE					
PLASTICITY CHART SILTY CLAY					
PROJECT No.		11-1191-0025		FILE No. 11-1191-0025.GPJ	
DRAWN	JJL	Oct 2012	SCALE	N/A	REV.
CHECK	AB	Oct 2012	<b>FIGURE B4-1</b>		
APPR	JMAC	Oct 2012			



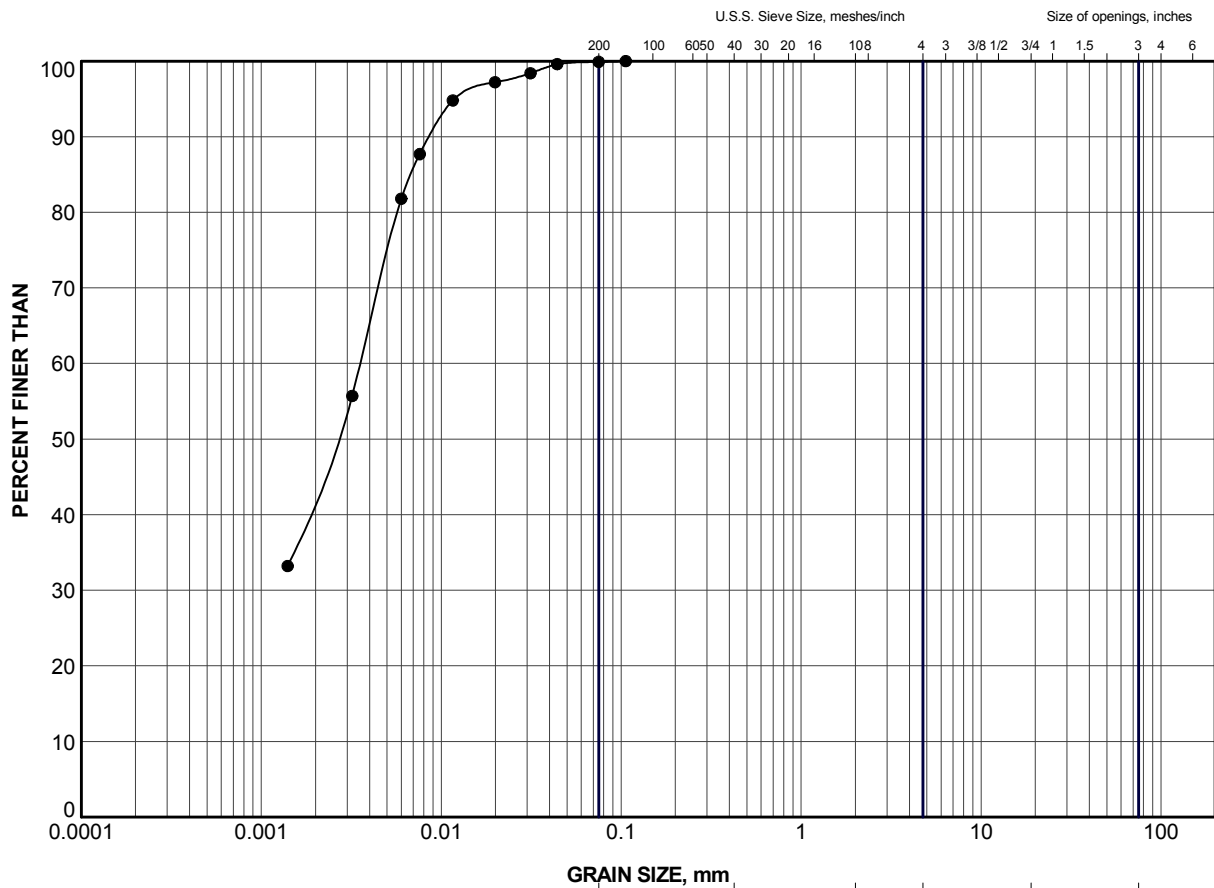




PROJECT					
HIGHWAY 11 ONR OVERHEAD BRIDGE					
TITLE					
PLASTICITY CHART CLAYEY SILT AND CLAY LAMINAE					
PROJECT No.		11-1191-0025		FILE No.	
DRAWN		JLL		Oct 2012	
CHECK		AB		Oct 2012	
APPR		JMAC		Oct 2012	
SCALE		N/A		REV.	
FIGURE B4-2					








GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

#### LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	ONR-2	7	242.2

PROJECT					
HIGHWAY 11 ONR OVERHEAD BRIDGE					
TITLE					
GRAIN SIZE DISTRIBUTION CLAYEY SILT LAMINA					
PROJECT No.		11-1191-0025		FILE No. 11-1191-0025.GPJ	
DRAWN	JJL	Oct 2012	SCALE	N/A	REV.
CHECK	AB	Oct 2012			
APPR	JMAC	Oct 2012			
 <b>Golder Associates</b> SUDBURY, ONTARIO			<b>FIGURE B5</b>		



**CONSOLIDATION TEST SUMMARY****FIGURE B6****Pg. 1 of 4****SAMPLE IDENTIFICATION**

Project Number	11-1191-0025	Sample Number	7
Borehole Number	ONR-2	Sample Depth, m	6.3

**TEST CONDITIONS**

Test Type	Standard	Load Duration, hr	24
Oedometer Number	1		
Date Started	July 25/12		
Date Completed	August 13/12		

**SAMPLE DIMENSIONS AND PROPERTIES - INITIAL**

Sample Height, cm	2.544	Unit Weight, kN/m <sup>3</sup>	16.40
Sample Diameter, cm	6.353	Dry Unit Weight, kN/m <sup>3</sup>	10.28
Area, cm <sup>2</sup>	31.70	Specific Gravity, assumed	2.70
Volume, cm <sup>3</sup>	80.64	Solids Height, cm	0.988
Water Content, %	59.51	Volume of Solids, cm <sup>3</sup>	31.32
Wet Mass, g	134.88	Volume of Voids, cm <sup>3</sup>	49.32
Dry Mass, g	84.56	Degree of Saturation, %	102.0

**TEST COMPUTATIONS**

Pressure kPa	Primary Consolidation	Corr. Height cm	Void Ratio	Average Height cm	t <sub>90</sub> sec	cv. cm <sup>2</sup> /s	mv m <sup>2</sup> /kN	k cm/s	Total Work kJ/m <sup>3</sup>
0	0.00	2.544	1.575	2.544					
9	0.03	2.540	1.572	2.542	217	0.0063	1.45E-04	9.00E-08	0.006
18	0.05	2.536	1.567	2.538	101	0.0135	2.11E-04	2.79E-07	0.031
35	0.10	2.526	1.557	2.531	240	0.0057	2.21E-04	1.23E-07	0.132
69	0.21	2.505	1.536	2.515	240	0.0056	2.42E-04	1.33E-07	0.565
143	0.88	2.417	1.447	2.461	1500	0.0009	4.69E-04	3.93E-08	4.292
285	1.75	2.242	1.269	2.330	2381	0.0005	4.84E-04	2.29E-08	19.778
571	0.96	2.146	1.173	2.194	866	0.0012	1.32E-04	1.52E-08	38.047
1140	0.69	2.077	1.102	2.112	375	0.0025	4.79E-05	1.18E-08	65.668
571	-0.07	2.084	1.110	2.081					
143	-0.30	2.114	1.140	2.099					
35	-0.32	2.146	1.172	2.130					
9	-0.32	2.178	1.205	2.162					

Note:

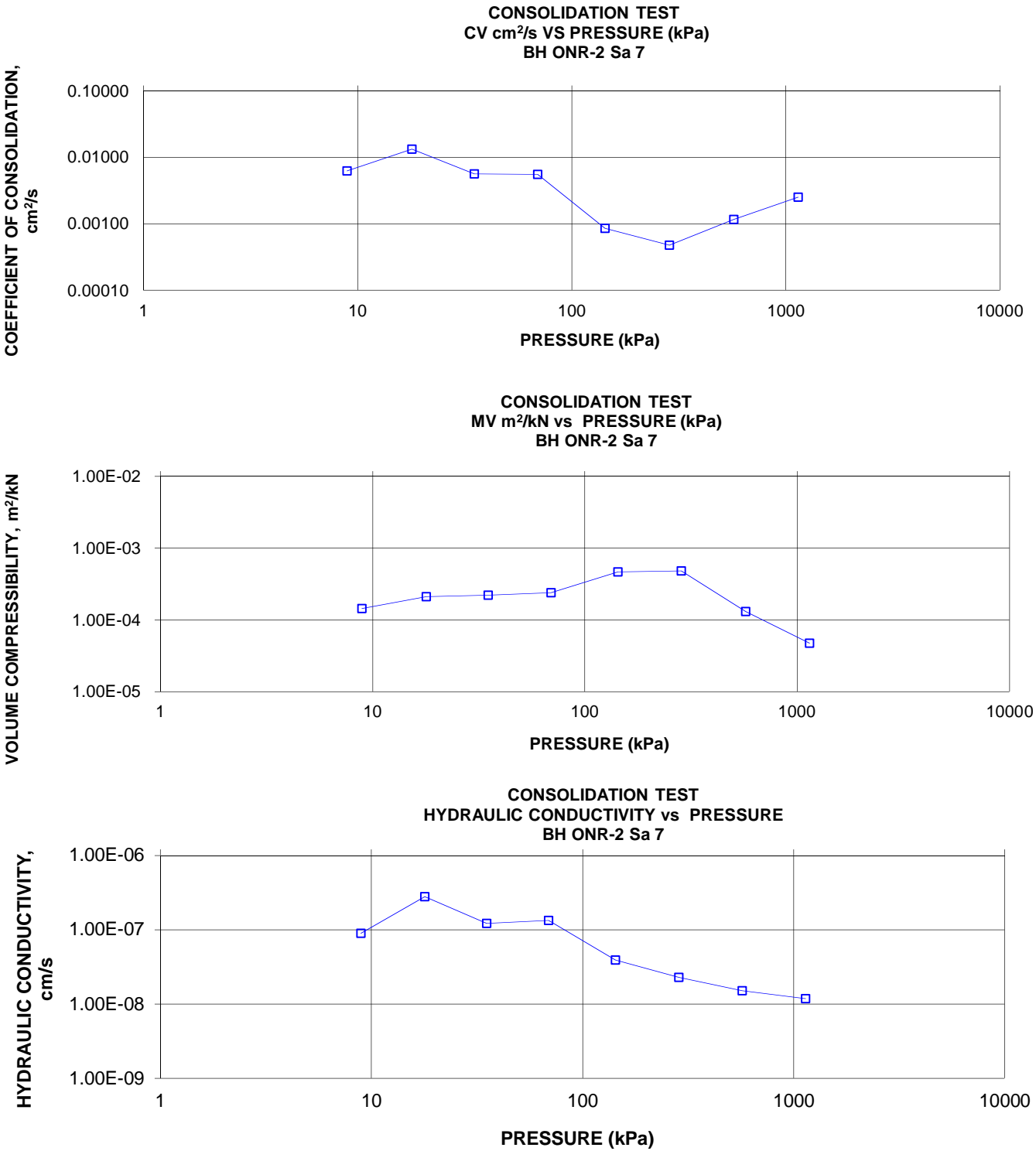
k calculated using  $\alpha$  based on t<sub>90</sub> values.**SAMPLE DIMENSIONS AND PROPERTIES - FINAL**

Sample Height, cm	2.178	Unit Weight, kN/m <sup>3</sup>	16.35
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m <sup>3</sup>	12.01
Area, cm <sup>2</sup>	31.70	Specific Gravity, assumed	2.70
Volume, cm <sup>3</sup>	69.05	Solids Height, cm	0.988
Water Content, %	36.10	Volume of Solids, cm <sup>3</sup>	31.32
Wet Mass, g	115.09	Volume of Voids, cm <sup>3</sup>	37.73
Dry Mass, g	84.56		

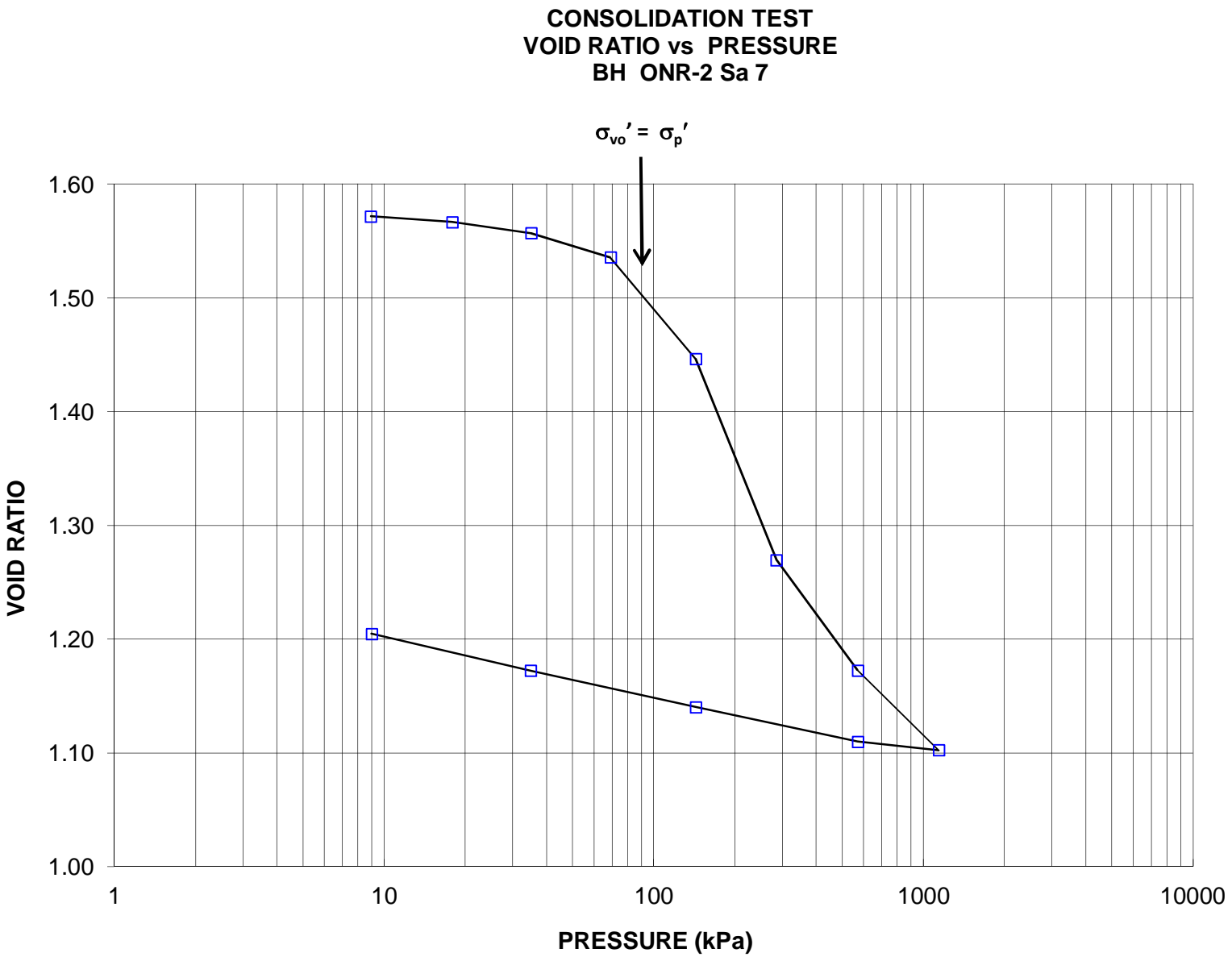


CONSOLIDATION TEST SUMMARY

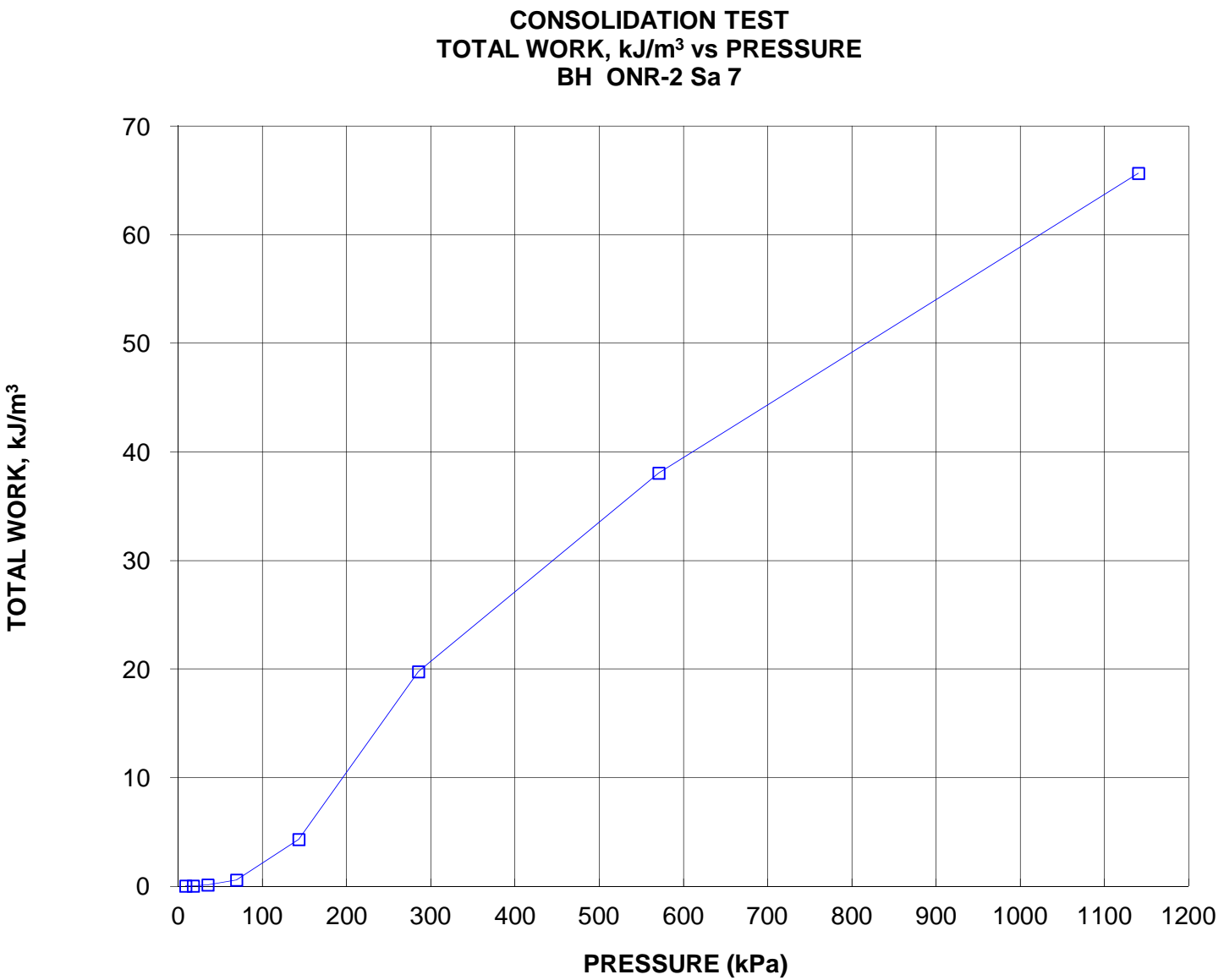
FIGURE B6  
Pg. 2 of 4









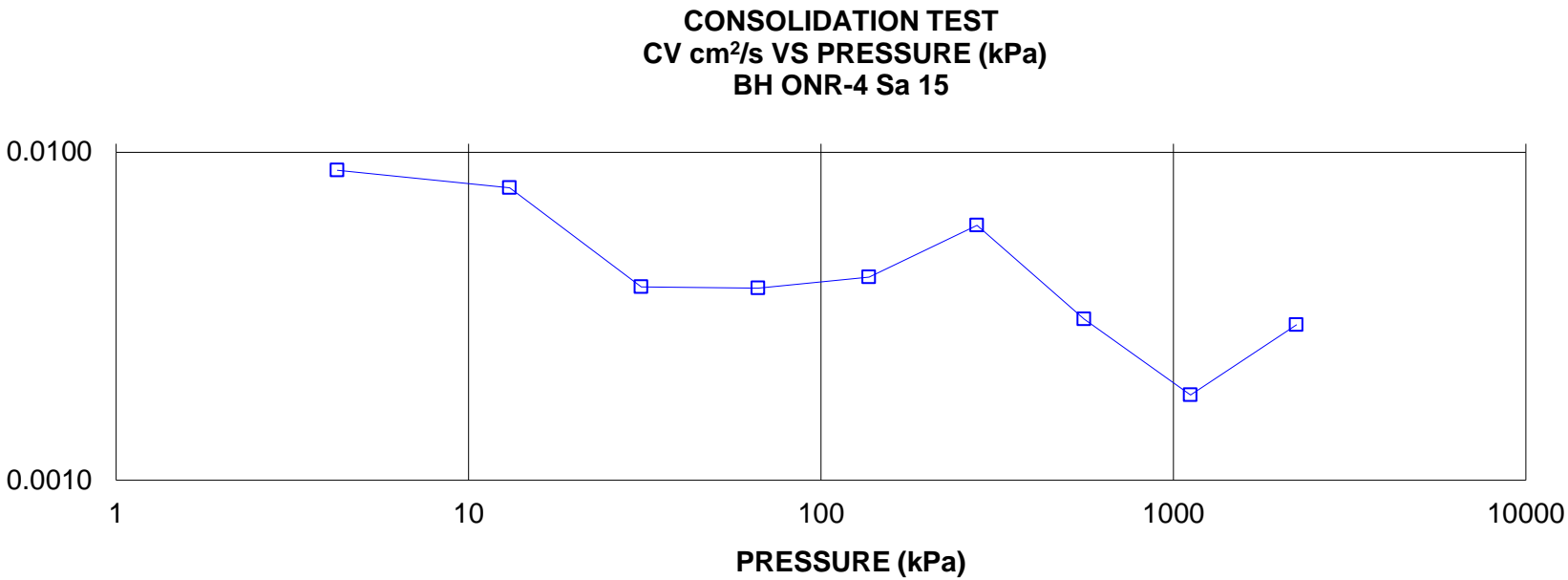




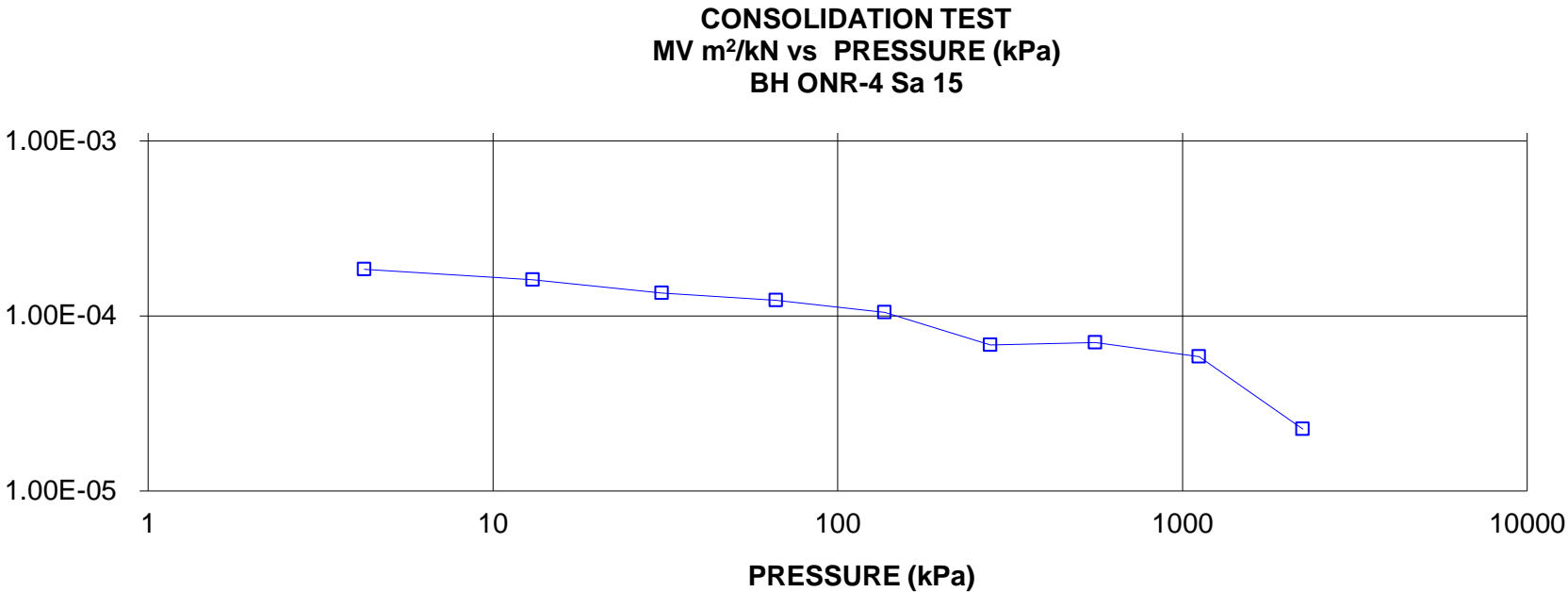
CONSOLIDATION TEST SUMMARY							FIGURE B7 Pg. 1 of 4		
SAMPLE IDENTIFICATION									
Project Number:		11-1191-0025				Sample Number:		15	
Borehole Number:		ONR-4				Sample Depth, m:		17.0	
TEST CONDITIONS									
Test Type		Standard				Load Duration, hr		24	
Oedometer Number		2							
Date Started		July 25/12							
Date Completed		August 13/12							
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL									
Sample Height, cm		2.526				Unit Weight, kN/m <sup>3</sup>		18.36	
Sample Diameter, cm		6.351				Dry Unit Weight, kN/m <sup>3</sup>		13.08	
Area, cm <sup>2</sup>		31.68				Specific Gravity, Assumed		2.70	
Volume, cm <sup>3</sup>		80.02				Solids Height, cm		1.248	
Water Content, %		40.39				Volume of Solids, cm <sup>3</sup>		39.53	
Wet Mass, g		149.84				Volume of Voids, cm <sup>3</sup>		40.49	
Dry Mass, g		106.73				Degree of Saturation, %		106.5	
TEST COMPUTATIONS									
Pressure	Primary	Corr.		Average					Total
kPa	Consolidation	Height	Void	Height	t <sub>90</sub>	cv.	mv	k	Work
		cm	Ratio	cm	sec	cm <sup>2</sup> /s	m <sup>2</sup> /kN	cm/s	kJ/m3
0	0	2.526	1.024	2.526					
4	0.02	2.524	1.023	2.525	154	0.0088	1.85E-04	1.59E-07	0.002
13	0.04	2.520	1.020	2.522	173	0.0078	1.61E-04	1.23E-07	0.014
31	0.06	2.514	1.015	2.517	346	0.0039	1.36E-04	5.17E-08	0.067
66	0.11	2.503	1.006	2.509	346	0.0039	1.23E-04	4.65E-08	0.280
137	0.19	2.485	0.991	2.494	317	0.0042	1.05E-04	4.27E-08	1.039
277	0.24	2.460	0.972	2.473	217	0.0060	6.84E-05	4.73E-08	3.221
558	0.50	2.410	0.932	2.435	406	0.0031	7.04E-05	2.14E-08	11.705
1117	0.83	2.328	0.866	2.369	653	0.0018	5.84E-05	1.04E-08	40.370
2235	0.64	2.264	0.814	2.296	375	0.0030	2.26E-05	6.60E-09	86.303
1117	-0.09	2.273	0.821	2.268					
277	-0.28	2.301	0.844	2.287					
66	-0.33	2.334	0.870	2.317					
13	-0.33	2.367	0.897	2.350					
4	-0.13	2.380	0.907	2.374					
Note: k calculated using cv based on t <sub>90</sub> values.									
SAMPLE DIMENSIONS AND PROPERTIES - FINAL									
Sample Height, cm		2.380				Unit Weight, kN/m <sup>3</sup>		18.31	
Sample Diameter, cm		6.35				Dry Unit Weight, kN/m <sup>3</sup>		13.88	
Area, cm <sup>2</sup>		31.68				Specific Gravity, Assumed		2.70	
Volume, cm <sup>3</sup>		75.40				Solids Height, cm		1.248	
Water Content, %		31.91				Volume of Solids, cm <sup>3</sup>		39.53	
Wet Mass, g		140.79				Volume of Voids, cm <sup>3</sup>		35.87	
Dry Mass, g		106.73							
Prepared By: SL				Golder Associates				Checked By: AB	



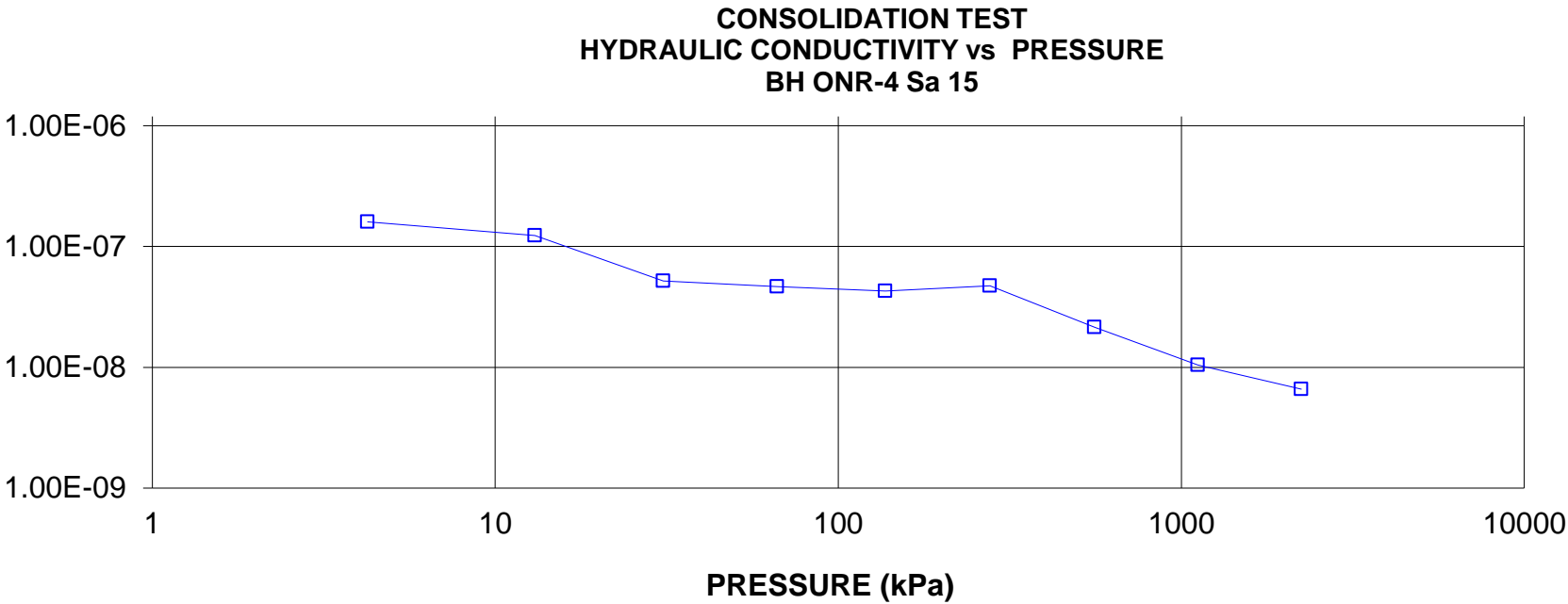
COEFFICIENT OF CONSOLIDATION,  
cm<sup>2</sup>/s



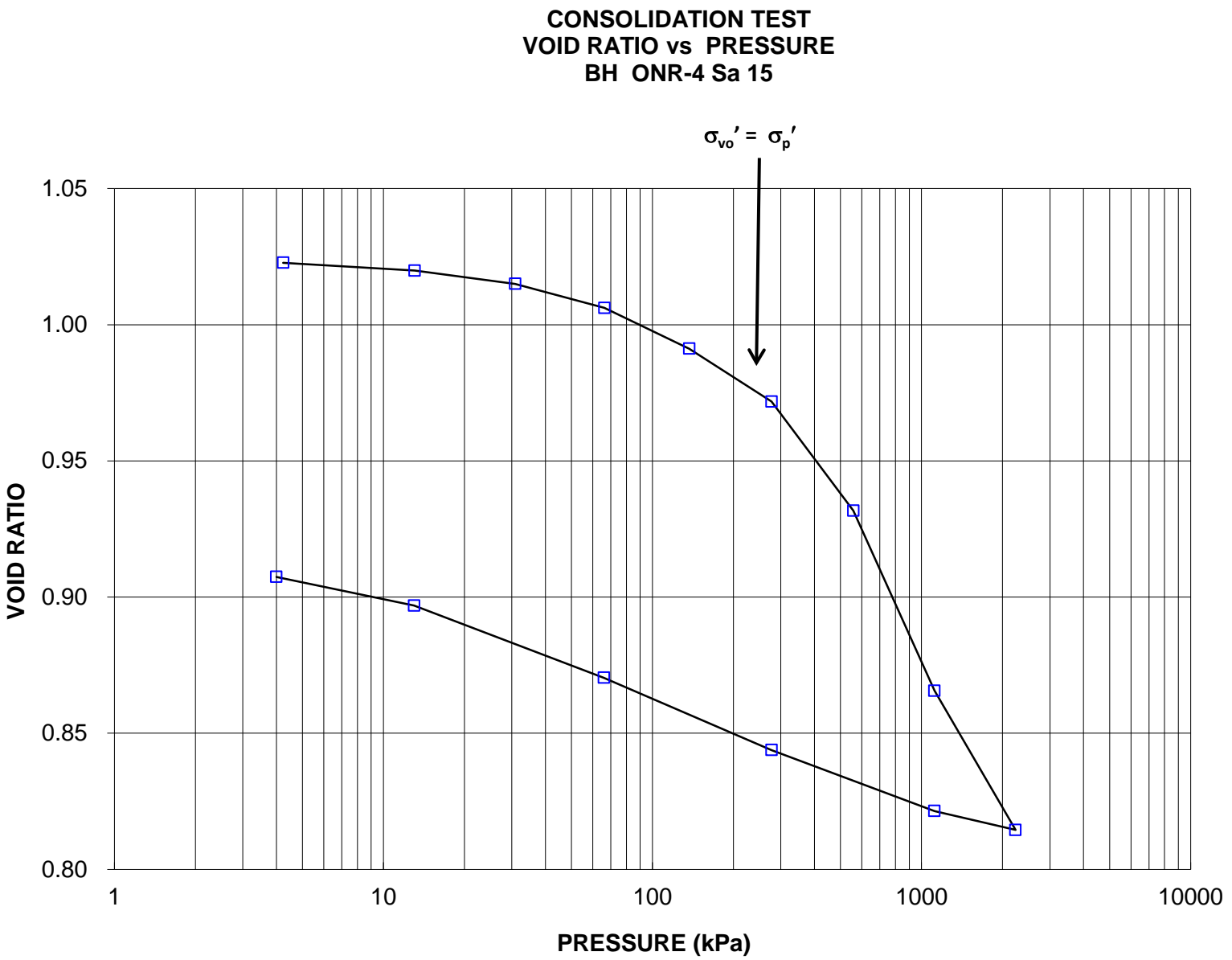
VOLUME COMPRESSIBILITY, m<sup>2</sup>/kN



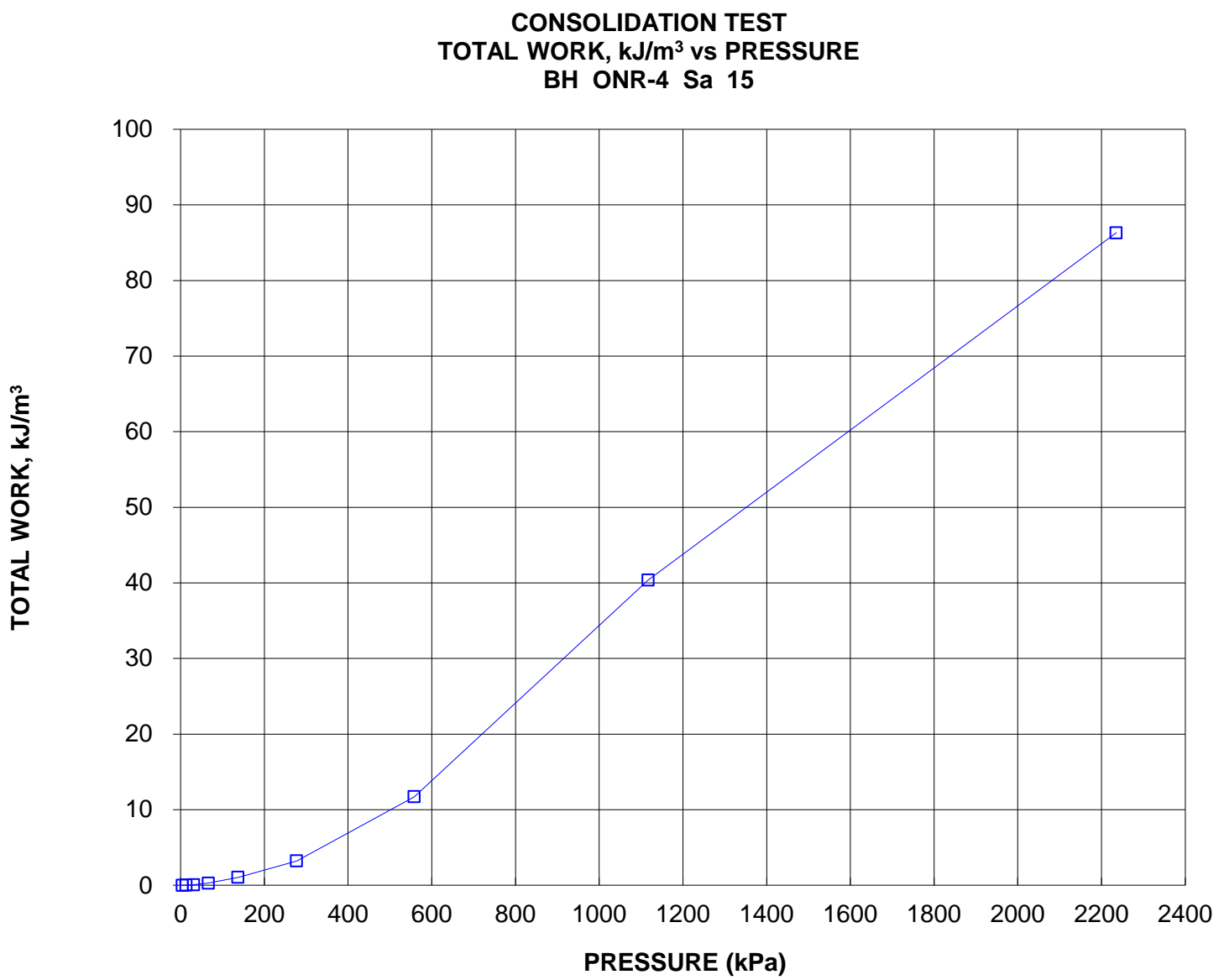
HYDRAULIC CONDUCTIVITY,  
cm/s



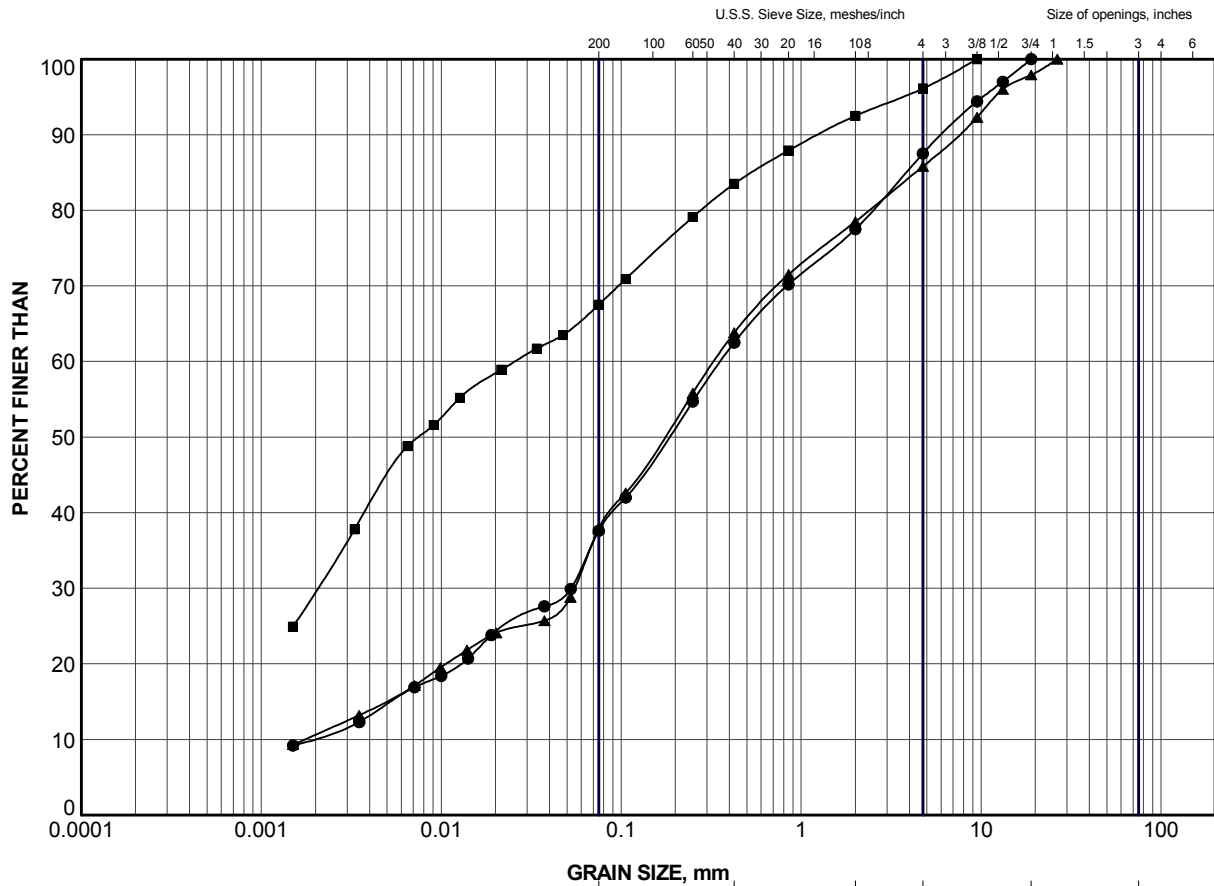












GRAIN SIZE, mm						
CLAY AND SILT	fine	medium	coarse	fine	coarse	Cobble Size
	SAND SIZE			GRAVEL SIZE		

#### LEGEND

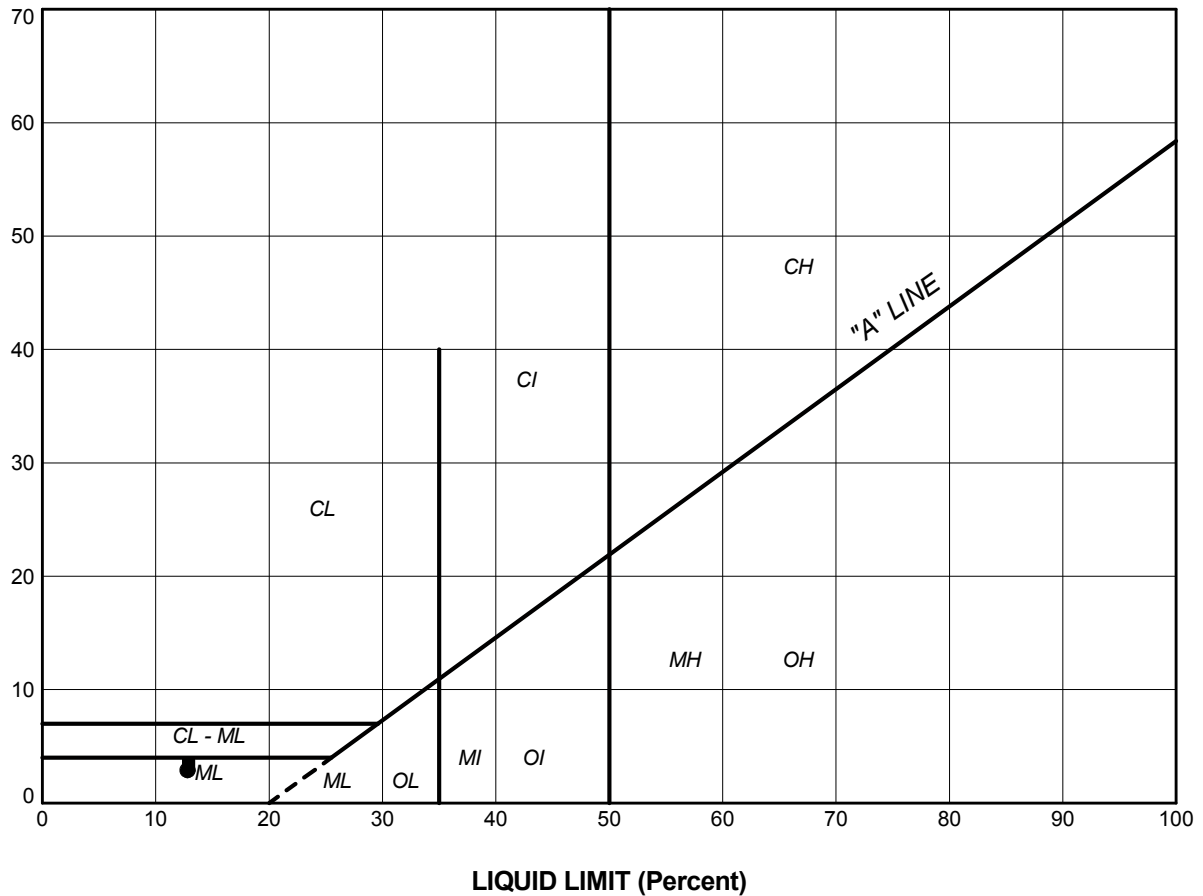
SYMBOL	BOREHOLE	SAMPLE	ELEV (m)
●	ONR-2	11	235.8
■	ONR-3	13	237.4
▲	ONR-4	19	233.6

PROJECT					
HIGHWAY 11 ONR OVERHEAD BRIDGE					
TITLE					
GRAIN SIZE DISTRIBUTION SILTY SAND TO SANDY SILT					
PROJECT No.		11-1191-0025		FILE No. 11-1191-0025.GPJ	
DRAWN	JJL	Oct 2012	SCALE	N/A	REV.
CHECK	AB	Oct 2012	FIGURE B8		
APPR	JMAC	Oct 2012			





PLASTICITY INDEX (Percent)



**SOIL TYPE**  
C = Clay  
M = Silt  
O = Organic

**PLASTICITY**  
L = Low  
I = Intermediate  
H = High

### LEGEND

SYMBOL	BOREHOLE	SAMPLE	LL(%)	PL(%)	PI
●	ONR-2	11	13	10	3
■	ONR-4	19	13	10	3

PROJECT					
HIGHWAY 11 ONR OVERHEAD BRIDGE					
TITLE					
PLASTICITY CHART SILTY SAND					
PROJECT No. 11-1191-0025			FILE No. 11-1191-0025.GPJ		
DRAWN	JJL	Oct 2012	SCALE	N/A	REV.
CHECK	AB	Oct 2012			
APPR	JMAC	Oct 2012			
			FIGURE B9		





At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

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Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

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