

Foundation Investigation and Design Report

Highway 9 Settlement West of
Highway 400, Township of King,
ON

WO 2013-11019



Prepared for:
Ministry of Transportation Ontario

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Geocres No. 31D-568

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FOUNDATION INVESTIGATION REPORT

For

WO 2013-11019

Highway 9 Settlement 1.4 km west of Highway 400
Township of King, Central Region

1.0 Introduction

Stantec Consulting Ltd. (Stantec) was retained by the Ministry of Transportation of Ontario (MTO) to provide foundation recommendations for rehabilitation of a Highway 9 section that has experienced settlement. The site is located approximately 1.4 km west of Highway 400 in the Township of King.

This Foundation Investigation Report has been prepared specifically and solely for the proposed rehabilitation of the investigated settlement area on Highway 9.

Project Number: WO 2013-11019

Project Location: Highway 9 approximately 1.4 km west of Highway 400

The work was carried out under MTO Foundations Engineering Retainer Agreement Number 2011-E-0021 Order 003 (WO 2013-11019) with Stantec Consulting Ltd.

2.0 Site Description and Geology

Site Location

The site location of the settled highway section is shown on the Key Plan inset to Drawing No. 1. The site is located on Highway 9 approximately 1.4 km west of Highway 400 between Holancin Road and Rupke Road.

General Site Description

General site photographs showing existing site features are provided in Appendix A.

At site the location, Highway 9 is oriented in the east-west direction. At this location, Highway 9 has two lanes in each direction with a left-turn lane in the middle and approximately 3.5 m wide gravel shoulders. The settled section of Highway 9 is approximately 50 m long within the eastbound lanes with the deepest settlement near the middle of the settled zone. A Corrugated Steel Culvert (CSP) is located approximately 7 m east of the deepest settlement location.

Historical data indicates that irrigation pipes were being used to facilitate the transfer of water from the south side (canal) of Highway 9 to the north to irrigate agricultural lands. The current flow of water in the culvert appears to be from south to north.

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Physiographic Description

The site is located within a physiographic region known as the Schomberg Clay Plains at the northern foothills of the Oak Ridges Moraine (Chapman and Putnam, 1984). This region contains deep deposits of stratified clay and silt. The schomberg sediments are typically varved clays with annual layers of silt (summer) and clay (winter) having variable thicknesses. This physiographic region is also known to contain high organic content soils commonly described as muck.

Locally, this site is located at the southwestern tip of a broad valley extending southwestward from Cook's Bay at the southern end of Lake Simcoe approximately 25 km. The valley, once a shallow extension of the lake, forms approximately 8000 ha. of marsh through which Holland River meanders through to Lake Simcoe. Peat deposit in the order of 2.5 m thickness has been encountered during the construction of Highway 400. The peat is considerably deeper towards Lake Simcoe; however, local variations in the peat thickness may be anticipated.

The surrounding area is generally flat (poorly drained) with predominantly agricultural field to the north and marshland to the south side of Highway 9. Drainage is generally toward the east and northeast toward Cook's Bay (Lake Simcoe). In the vicinity of the site, flow is towards the Holland Drainage Canal.

3.0 Investigation Procedures

3.1 REVIEW OF EXISTING INFORMATION

During of the current investigation, we have acquired and reviewed several Geocres reports within the general area of the project, two of these being the most relevant in terms of their proximity to the project site. These reports are Geocres Nos. 31D-364 and 31D-373. Summary of the subsurface conditions based on these reports are provided below.

Geocres 31D-364:

- Foundation investigation work (1997) was carried for the proposed bridge widening on the south side of Highway 9 over the Holland Drainage Canal, approximately 450 m east of the settled section of Highway 9.
- Copy of the borehole locations & soil strata drawing is provided in Appendix A.
- The subsurface conditions immediately west of the bridge (i.e., based on boreholes closest to the site) can be summarized as follows (to bottom):
 - 100 to 150 mm of topsoil
 - 1.7 to 2.9 m of fill (sandy silt and clayey silt)
 - Approximately 700 mm firm to stiff amorphous granular peat. The peat has SPT N-values of 5 to 8 blows per 0.3 m. A field vane shear test indicated an undrained shear strength of approximately 125 kPa suggesting a very stiff consistency.
 - Stiff organic silt (marl) (unknown thickness as was observed near the bottom of the borehole)
 - Approximately 12 m firm to very stiff silty clay (upper silty clay)
 - Approximately 2.5 m stiff sandy silty clay till with some gravel (upper silty clay till)

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- Approximately 3.5 m very stiff silty clay (lower silty clay)
 - Approximately 3.5 m hard clay to silt till (lower till)
 - Unknown thickness of dense to very dense sand and silt deposit.
- Shallow boreholes remained dry while artesian groundwater conditions were encountered in the sand and silt deposits

Geocres 31D-373:

- The foundation investigation work (1999) was carried out for the proposed widening of Highway 9 from West Canal Bank Road to 0.5 km west of Weston Road (up to approximately 130 m west of the site), i.e., widening to the south of the then two lane Highway 9 rural configuration to obtain a five lane rural configuration.
- The closest borehole from this 1999 study is located about 50 m east of the current study area.
- Copy of the Site Key Plan is attached in Appendix A.
- Report indicates that the canal was built in the early 60s (then approximately 45 m south of the Highway 9 centreline).
- Surficial geology information suggested the prevalence of organic peat and muck deposits at the surface overlying a combination of lacustrine sand and clay.
- Construction history of Highway 9 in the area indicated that deep excavations of up to 7.6 m below the then proposed Highway 9 grade were predicted to remove a locally deep pocket of peat. A deep pocket of peat was also noted immediately to the east of the project limits, i.e., within the general area the current assignment.
- The construction of the canal involved excavation of existing material and its subsequent disposal in the tract of land between the canal and Highway 9.
- The subsurface conditions of the eastern section of the study area (based on boreholes closest to the site) can be summarized as follows:
 - Up to 1.5 m thick sandy silt with organics fill
 - Up to 4.4 m thick soft to firm, fibrous to amorphous peat with typical thickness of 1.5 to 2 m; pertinent geotechnical properties of the peat as documented in this report are summarized in Section 4.2.6.
 - A sequence of lacustrine type deposits consisting in turn of approximately 0.5 m of sand or sandy silt, 1 m of firm silty clay and compact silty sand extending to the maximum depth of investigation (9.7 m).
- Groundwater was estimated to be approximately 1 m below the existing grade to the south of the highway; this level was inferred to coincide with the water level in the adjacent drainage canal.

3.2 FIELD INVESTIGATION

Prior to carrying out the investigation, Stantec made arrangements to obtain utility clearances for the proposed borehole locations.

A field investigation with 3 boreholes was carried out on September 11 and 12, 2013. The boreholes were designated BH13-1 through BH13-3 and their locations along with soil strata along the settled Highway 9 section are shown on the Borehole Location Plan Drawing No.1 in Appendix A.

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The boreholes were advanced within the eastbound driving lane of Highway 9, approximately 2.5 m from the edge of asphalt pavement. Borehole BH13-2 was advanced at the observed deepest settlement location while Boreholes BH13-1 and BH13-3 were advanced near the western and eastern limits, respectively, of the settled section of Highway 9. Borehole BH13-2 is located approximately 7 m west of the CSP.

All boreholes were advanced using a truck mounted D90 drill rig equipped with hollow & solid stem augers and soil sampling equipment.

Groundwater levels were inferred from the water levels within the open boreholes after the completion of drilling.

3.3 LOCATION AND ELEVATION SURVEY

Elevation and location survey of the boreholes was performed by Stantec personnel. The ground surface elevation at each borehole was surveyed with reference to a Geodetic Benchmark (BM) provided by MTO. The BM was a 1.8 m round iron bar with brass cap located on north side of Highway 9, approximately 565 m east of Rupke Road. The BM monument was stamped 00820108167. The geodetic elevation of the BM was 220.485 m.

Table 3.1 summarizes the location and elevation information for the boreholes drilled at the culvert site.

Table 3.1: Borehole Information Summary

| | Boreholes | | |
|-------------------------------|-----------|-----------|-----------|
| | BH13-1 | BH13-2 | BH13-3 |
| MTM Zone 10 Coordinates | | | |
| Northing | 4876174.0 | 4876181.4 | 4876188.5 |
| Easting | 295919.3 | 295944.3 | 295968.3 |
| Ground Surface Elevation (m) | 220.0 | 219.9 | 220.0 |
| Total Depth Drilled (m) | 6.9 | 11.4 | 6.7 |
| End of Borehole Elevation (m) | 213.1 | 208.5 | 213.3 |
| Number of Soil Samples | 8 | 15 | 8 |
| Depth Cored (m) | - | - | - |

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3.4 LABORATORY TESTING

All samples were subjected to a detailed visual examination by a Geotechnical Engineer. The following geotechnical laboratory tests were carried out:

| <u>Test</u> | <u>No. of Tests</u> |
|--|---------------------|
| Moisture Content | 35 |
| Grain Size Analysis - Sieve | 5 |
| Grain Size Analysis – Sieve & Hydrometer | 2 |
| Grain Size Analysis – Hydrometer | 1 |
| Atterberg Limits | 1 |
| Organic Matter | 4 |

Samples remaining after testing will be stored for one year after issuance of the final report. After the storage period, the samples will be discarded.

Subsequent to the drilling investigation, a survey of the settled Highway 9 profile was carried out along the eastbound edge of the pavement. The survey was conducted over approximately 60 m segment of the settled Highway 9, approximately 30 m both sides of the deepest settlement location. The survey points were spaced approximately 5 m parallel to the Highway 9 centreline. Figure 6 of Appendix D shows the measured settlement profile along the southern edge of the eastbound lane of Highway 9.

4.0 Subsurface Conditions

4.1 GENERAL

The subsurface conditions observed in the boreholes advanced within the settled highway segment are presented in detail on the Borehole Records provided in Appendix B. An explanation of the symbols and terms used to describe the Borehole Records is also provided.

In general, the subsurface stratigraphy consisted of asphalt pavement over granular road base over silty gravel with sand roadway fill material, followed by a silty sand / sandy silt layer underlain by peat.

A borehole location plan and a stratigraphic section of the soils encountered within the boreholes are provided in Drawing No. 1 of Appendix A.

4.2 OVERBURDEN

4.2.1 Pavement Structure

The existing pavement structure was measured within the boreholes advanced through the roadway. Table 4.1 shows the observed pavement structure at the borehole locations.

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Table 4.1: Pavement Thickness

| Borehole | Asphalt Thickness (mm) | Approximate Thickness of Granular Base and Subbase (mm) |
|----------|------------------------|---|
| BH13-1 | 380 | 1100 |
| BH13-2 | 390 | 1100 |
| BH13-3 | 390 | 210 |

The granular road base and subbase material generally consisted of brown silty sand with gravel.

The Standard Penetration Test (SPT) blow count (N-values) observed within the base and subbase material was 44 and 53 blows per 0.3 m suggesting a dense to very dense state.

Moisture content and grain size distribution tests carried out on representative samples of the road base and subbase material yielded the following results:

Gravel: 27 & 29%
Sand: 56 & 57%
Fines (silt & clay): 15 & 16%
Moisture content: 3 to 5%

Representative grain size distribution curve for the base and subbase material layer is provided in Figure No. 1a of Appendix C.

4.2.2 FILL: Roadway Material

Roadway fill material was encountered in all boreholes immediately beneath the road base and subbase materials. The roadway fill consisted of silty gravel with sand. The roadway fill was approximately 800, 800, and 900 mm thick, respectively, in Boreholes BH13-1, BH13-2 and BH13-3 and extended to respective base elevations of 217.7, 217.6 and 218.5 m.

The N-values observed within the roadway fill ranged from 11 to 25 blows per 0.3 m suggesting a compact state.

Moisture content and grain size distribution tests carried out on a representative sample of the roadway fill yielded the following results:

Gravel: 46%
Sand: 36%
Fines (silt & clay): 18%
Moisture content: 4 to 6%

A representative grain size distribution curve for the roadway fill material is provided in Figure No. 1b of Appendix C.

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4.2.3 FILL: Silty Sand (Subgrade Material)

A silty sand layer was encountered in all boreholes immediately beneath the roadway fill. This layer was approximately 1.1 to 3.2 m thick and extended to a base elevation of between 214.4 and 217.0 m.

The SPT N-values for the silty sand layer ranged from 1 to 22 blows per 0.3 m suggesting a very loose to compact state.

Moisture content and grain size distribution tests carried out on representative samples of the sandy silt / silty sand layer yielded the following results:

| | |
|----------------------|----------|
| Gravel: | 7 & 18% |
| Sand: | 51 & 52% |
| Fines (silt & clay): | 31 & 41% |
| Moisture Content: | 9 to 10% |

The Unified Soil Classification System (USCS) group symbol for this layer is SM (silty sand). The grain size distribution curve for the sandy silt material is provided in Figure No. 2 of Appendix C.

An approximately 200 mm thick layer of peat was encountered immediately beneath the silty sand layer (approximate elevation 214.4 m) in BH13-2; its moisture content was 36%. A thin seam of peat was also observed beneath this layer (approximate elevation 216.6 m) in BH13-1; its moisture content was 17%.

4.2.4 FILL: Sandy Silt (Subgrade Material)

A sandy silt layer was encountered in BH13-1 and BH13-3 beneath the silty sand layer. This layer was 0.7 to 1.3 m thick and extended to base elevation of 215.3 to 216.3 m.

The SPT N-values for the sandy silt layer ranged from 0 (weight of hammer) to 8 blows per 0.3 m suggesting a very loose to loose state.

Moisture content and grain size distribution tests carried out on representative samples of the sandy silt layer yielded the following results:

| | |
|-------------------|-----------|
| Gravel: | 6% |
| Sand: | 43% |
| Silt Size: | 44% |
| Clay Size: | 7% |
| Moisture Content: | 10 to 11% |

The Unified Soil Classification System (USCS) group symbol for this layer is ML (sandy silt). The grain size distribution curve for the sandy silt material is provided in Figure No. 3 of Appendix C.

4.2.5 FILL: Sandy Silty Clay (Subgrade Material)

A sandy silty clay layer was encountered in BH13-2 beneath the silty sand layer. This layer was approximately 1.3 m thick and extended to base elevation of 212.9 m.

The SPT N-values for the sandy silty clay was 0 (weight of hammer) blows per 0.3 m suggesting a very soft consistency.

Moisture content and grain size distribution tests carried out on representative samples of the sandy silt layer yielded the following results:

| | |
|-------------------|----------|
| Gravel: | 1% |
| Sand: | 38% |
| Silt Size: | 47% |
| Clay Size: | 14% |
| Moisture Content: | 14 & 15% |

Atterberg limits test carried out on a representative sample from this layer indicated plasticity index of 6. The Unified Soil Classification System (USCS) group symbol for this layer is CL-ML (sandy silty clay). The grain size distribution curve for the sandy silty clay material is provided in Figure No. 3 of Appendix C; the plasticity chart is provided in Figure 4.

4.2.6 Peat

A dark brown peat layer was encountered in all boreholes immediately beneath the fill layers. Drilling was terminated within the peat layer in all boreholes upon reaching or exceeding the target investigation depth in each borehole. The boreholes extended 2.0 m, 4.4 m, and 3.2 m into the peat deposit at the borehole locations in BH13-1, BH13-2 and BH13-3 respectively.

The peat consisted predominantly of fibrous, partially decomposed vegetable matter saturated with water; it also consisted of wood inclusions and is occasionally mottled with grey sandy silt throughout.

The SPT N-values for the peat layer ranged from 3 to 24 blows per 0.3 m, indicating a soft to very stiff consistency.

Tests carried out on representative samples of the peat material yielded the following results:

| | |
|---------------------|------------|
| Gravel: | 0% |
| Sand: | 3% |
| Fines (silt & clay) | 89% |
| Moisture Content: | 79 to 236% |
| Organic Matter: | 48 to 79% |

The tested sample of the peat was found to be non-plastic. The Unified Soil Classification System for the peat is PT or organic silt (OL). The grain size distribution curve for a sample of the peat is shown in Figure No. 5 of Appendix C.

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Notes:

1. The relatively thick layer of peat observed in the boreholes is believed to indicate the presence of a localized deep pocket when compared to the peat thickness generally observed in the area. Page 5 of the Terraprobe report (Geocres 31D-373) states that there is a deep pocket of peat immediately to the east of their project limit, which was just west of the settlement area, suggesting that a borehole may have been drilled in the area for an adjacent project; Stantec did not find a report for the roadway widening in this area.
2. The relatively high blow counts noted within the peat layer suggests that a significant strength gain has taken place.
3. Geocres Report No. 31D-373 (1999) reported that the in-situ undrained shear strength using field vane was in the order of 10 to 30 kPa, based on the review of the investigation completed in 1960s in association with the construction of the original highway. The 1999 report noted undrained shear strength measurements of up to 96 kPa, with most values in the order of 40 kPa. This suggests that the peat has undergone some strength gain since the original construction of the highway, most likely as a result of the surcharge effect.
4. It is anticipated that the peat has undergone additional strength gain since the widening of Highway 9 (likely in early 2000s).
5. Based on the two consolidation tests carried out on the peat reported in the above Geocres, the estimated compression indexes (C_c) were 6.5 and 8 (average of 7.3). The corresponding coefficient of secondary compression (C_{α}) was calculated to be between 0.32 and 0.56 (average of 0.44). The estimated recompression indexes (c_r) were 1.0 and 1.3 (average 1.15). The initial void ratios (e_o) were 13.5 and 15.5 (average of 14.5).

4.3 BEDROCK

Bedrock was not encountered within the depth of exploration of this investigation.

4.4 GROUNDWATER

The depth to groundwater was estimated in all boreholes at the time of drilling on September 11 and 12, 2013. These groundwater levels are not stabilized measurements, and hence will be referred to as "inferred". The inferred groundwater levels are summarized in Table 4.2.

Table 4.2: Inferred Groundwater Levels (time of drilling)

| Borehole | Ground Surface Elevation (m) | Groundwater | |
|----------|------------------------------|-------------|---------------|
| | | Depth (m) | Elevation (m) |
| BH13-1 | 220.0 | 2.3 | 217.7 |
| BH13-2 | 219.9 | 1.5 | 218.4 |
| BH13-3 | 220.0 | 1.6 | 218.4 |

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

The water level in the South Canal (Holland Marsh Drainage Canal) was measured to be at approximate elevation of 218.2 m.

4.5 SETTLEMENT PROFILE

Figure 6 in Appendix D shows the settlement profile. The following is observed.

- The roadway surface is at el. 219.96 m and 219.97 m, west and east of the affected area.
- The lowest point of the settled area is at el. 219.77, suggesting a total settlement of 200 mm.
- The area affected by the settlement extends to about 30 m on either side of the point with the deepest settlement.

5.0 Miscellaneous

The field work was carried out under the supervision of Dan Stunden, Geotechnical Engineering Technologist, under the direction of Simon Gudina, P.Eng., Geotechnical Engineer.

USL-1 Underground Service Locators Inc. of Ottawa, Ontario, carried out the private and public utility locates for the boreholes.

The D90 drilling equipment was supplied and operated by Walker Drilling of Utopia, Ontario.

Elevation and location survey of the borehole locations was carried out by Stantec personnel.

Geotechnical laboratory testing was carried out at Stantec's Ottawa laboratory.

This report was prepared by Simon Gudina and reviewed by Raymond Haché, MTO Designated Principal Contact for Foundation Engineering.

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6.0 Closure

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

Respectfully Submitted;

STANTEC CONSULTING LTD.



Simon Gudina, Ph.D., P.Eng.
Geotechnical Engineer



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Designated Principal MTO Foundation Contact



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Highway 9 Settlement 1.4 km west of Highway 400
Township of King, Central Region

7.0 General Background

Project Purpose/Justification

Approximately 50 m long section of Highway 9 within the eastbound lanes has experienced settlement. Based on the Foundations Engineering Terms of Reference for this assignment, a visual evidence of slight differential settlement within the passing lane and relatively uniform settlement was noted within the right lane of the eastbound Highway 9.

Construction/Maintenance History

The construction history at the site indicates that, in 2009, a sinkhole developed approximately 500 west of Weston Road (approximately 240 m west of Holancin Road). The Foundations Engineering Terms of Reference (TOR) for this assignment indicates the sinkhole was subsequently repaired under contract 2009-2297. The repair included filling of the sinkhole (with soil?), and removal and replacement of a collapsed non-structural culvert. The TOR also indicates that a reinforced soil system was installed for a 100 m section of the roadway. The reinforced soil system is understood to consist of (from bottom to top) geotextile, 400 mm thick 50 mm Crusher Run Limestone, a single layer of Tensar Geogrid BX 1200 (SS2) and a designated thickness of Granular 'B'.

It has also been observed that the Holland Drainage Canal immediately south of Highway 9 (approximately 25 m from the edge of pavement of eastbound Highway 9) was dredged in the summer of 2013. Review of Geocres Report No. 31D-373 (proposed widening of Highway 9) which was completed in July 1999 suggests that Highway 9 has been widened (approximately) in the last 10 to 13 years (exact date not available at the time of writing).

Proposed Work

Localized repair of the settled Highway 9 section will be carried out. At the project site, Highway 9 has two lanes in each direction with additional passing lane in the middle, with a total width of approximately 19.0 m. The settled area is in eastbound passing lane and extends over a length of about 60 m.

Construction Staging & Detours

It is understood that a short term local road detour is not anticipated during the proposed localized repair of the settled portion of Highway 9.

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The proposed localized repair of Highway 9 will likely require roadway protection depending on the preferred rehabilitation procedure. Since only the two eastbound lanes are affected, the remaining three lanes can be used during the proposed repair of the settled roadway section.

8.0 Engineering Recommendations

8.1 GEOTECHNICAL DESIGN PARAMETERS

The soil conditions at this site generally consist of asphalt pavement over granular road base over silty gravel with sand roadway fill material, followed by a silty sand / sandy silt layer underlain by peat.

For design purposes, the following soils profile will be used:

Table 8.1: Geotechnical Model for the Settled Highway 9 Section

| Approximate Elevation | | Soil Type | Design Properties |
|--------------------------|--------------------------|--|--|
| From | To | | |
| 220.0 | 219.6 | Asphalt | - |
| 219.6 | 218 (varies slightly) | Fill: silty sand with gravel to silty gravel with sand (compact to very dense) | Total Unit Weight = 21.5 kN/m ³ (moist) Friction Angle, $\phi = 33^\circ$ $E' = 50$ MPa |
| 218 (varies slightly) | 214.4 | Silty sand (loose to compact) | Total Unit Weight = 23 kN/m ³ (saturated) Friction Angle, $\phi = 32^\circ$ $E' = 35$ MPa |
| 214.4 | 212.9 | Sandy silty clay (very soft) | Total Unit Weight = 20 kN/m ³ Undrained Shear Strength = 50 kPa |
| 212.9 | 207.9 | Peat or muck (organic silt) (stiff to very stiff) | Total Unit Weight = 12.5 kN/m ³ |

A design water level elevation of 218 m will be assumed for the site.

8.2 TIME DEPENDENT SETTLEMENT OF PEAT

8.2.1 Vertical Stress Above the Peat

The effective vertical stress at the base of the embankment can be estimated as:

$$\sigma'_v = \gamma_f H - \gamma_w H_w$$

where γ_f is the unit weight of the fill material.

Assuming the $H = 7$ m based on the deepest fill height observed in Boreholes BH13-2.

$$\sigma'_v = \pm 105 \text{ kPa}$$

8.2.2 Construction Settlement (Primary Consolidation)

The maximum settlement S_c due to the embankment construction (primary consolidation) can be approximated by assuming that the peat was normally consolidated prior to embankment construction. Thus the primary consolidation settlement S_c can be estimated as:

$$S_c = C_c * H_o / (1 + e_o) * \log[(\sigma_{vi} + \Delta\sigma_v) / \sigma_{vi}]$$

Where H_o is the peat thickness, e_o is the initial void ratio of the peat, σ_{vi} is the initial effective overburden stress and $\Delta\sigma_v$ is the vertical stress increase.

Assuming that the initial peat removal was to a depth of about 5 m and that it was replaced with the subgrade fills, the additional load on the peat at that time would have been close to the existing effective stresses (since peat γ_{sat} is only slightly more than the γ_{water}).

To assess the initial compression which would have occurred in 1999, the following pre-construction parameters reported by Terraprobe were used.

$$H_o = 5.0 \text{ m}, e_o = 14.5, C_c = 7.3$$

Using the above calculation method and parameters, the estimated primary consolidation settlement would have been in the order of 1.6 to 2.0 m. Most of the primary settlement within the peat would have occurred during fill placement, and certainly within two to three weeks after fill placement. Therefore at the time of paving it is assumed that all primary settlement was completed.

It is noted that above primary settlement estimate was calculated based on test results from an adjacent contract and that those conditions may not have been the same as those at the current site. In general the parameters appear to be quite high relative to the moisture content of the organic materials. Regardless, given that primary settlement is anticipated to have been completed at the time of paving, the quantity of primary settlements which would have occurred does not impact the selection of the proposed localized repair method.

8.2.3 Long-Term Settlement (Secondary Compression)

Peat deposits typically experience significant settlement by the end of construction mostly due to primary consolidation settlement (as estimated above) followed by additional settlements due to secondary compression. The secondary compression (creep) settlement can be estimated as

$$S_s = H_o * C_\alpha / (1 + e_o) * \log(t/t_o)$$

where S_s is the creep settlement within the time interval t_o to t and all other parameters are as defined previously. The parameter t_o is generally taken to be the time at the end of primary consolidation and t is the time of interest.

The observed settlement of 200 mm (Figure 6) is assumed to have occurred between the time of repair (anticipated to be early 2010) and late 2013. This represents a time period of

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approximately 3 to 4 years. For the purpose of analysis, it has been assumed that the 200 mm of settlement has occurred over 4 years.

Estimate of settlement for the remainder of the design life

In order to assess the amount of secondary consolidation anticipated should no modification be carried out, we have assumed the following design parameters for the organic peat:

- Moisture content 120%
- Compression index, C_c 1.25
- Secondary compression index, $C_{\alpha\epsilon}$ 0.08
- Void ratio, e_0 4.92

The above parameters reflect the current conditions, rather than those at initial construction. The reasoning to use current condition parameters are as follows:

- Initial construction was around 1999 at which time it is estimated that primary consolidation would have occurred during the construction phase, followed by long term secondary compression over the following 10 years.
- In late 2009 or 2010, the culvert reconstruction was carried out and the profile grade would have been reestablished. This construction would have increased the loading on the peat, triggered a new round of primary consolidation, and reinitiated secondary compression.

Figure 7 in Appendix D describes the stress history model assumed in interpreting future anticipated settlements and in developing localized repair options.

The slope of the secondary compression line is estimated to be:

$$200 \text{ mm} / \log (1460 \text{ days} / 20 \text{ days})$$

This slope would suggest the following amounts of future settlement if the existing profile was maintained (ie. If no repair work is carried out).

| | |
|----------|-------|
| 5 years | 40 mm |
| 10 years | 60 mm |
| 15 years | 75 mm |

These estimates are extrapolated from the amount of secondary compression that has occurred since the profile grade was reestablished. The actual peat thickness is not known, however, the slope of the secondary compression line incorporates the effects of the deposit thickness.

Since the roadway profile is to be reestablished, significant additional loading would occur if the design profile grade was established by placing new materials. This additional loading would trigger new primary consolidation, and reinitiate secondary consolidation. Anticipated settlements should the grades be reestablished by adding additional fill would be greater than that estimated above.

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Estimate of Settlement Under Short-Term Regrading

For short term regarding the following has been considered:

- Temporary grade correction of up to 200 mm using hot mix asphalt and pre-applied tack-coat
- Short term periods of 1 year and 2 year
- Load application of 3 kPa on the peat after considering stress redistribution within overlying overburden
- Primary consolidation completed at the end of 20 days.

Using the parameters listed above, the following amounts of settlements would be anticipated after repaving for a short term repair option.

| | |
|----------|-----------------|
| 20 days | 12 mm (primary) |
| 365 days | 95 mm |
| 730 days | 115 mm |

This amount of surface deformation is less than currently present, suggesting that regarding is a suitable short term repair option.

Benefits of Unloading Using Light Weight Fill

Using the parameters listed above, the benefits of unloading were evaluated by the following process.

1. Estimate the primary consolidation associated with the unloading pressure (for calculation purposes only).
2. Vertical upward shift of the secondary compression line equal to the estimate of theoretical consolidation associated with the unloading pressure.
3. Estimating the time delay before secondary compression reinitiates by using the slope of the current secondary compression line.

Using this approach the following summarizes the benefits associated with pressure unloading at the site.

Table 8.2: Pressure Unloading Benefits

| Pressure Unloading | Time Delay to Reinitiate Secondary Compression | Future Settlement | |
|--------------------|--|-------------------|----------|
| | | 10 years | 15 years |
| 10 kPa | 5 years | 30 mm | 50 mm |
| 15 kPa | 9 years | 5 mm | 25 mm |
| 20 kPa | 16 years | 0 | 0 |

8.3 LONG TERM REPAIR OPTIONS FOR THE SETTLED ROADWAY

Possible long term repair options for the settled Highway 9 segment are compared in the following table.

Table 8.3: Comparison of Roadway Repair/Renovation

| Option | Advantages | Disadvantages | Relative Cost | Risk & Consequences |
|--|---|---|---------------|---|
| Regrade (Raise profile of settled roadway) | <ul style="list-style-type: none"> • Simple construction • No excavation involved • Short construction time | <ul style="list-style-type: none"> • Additional weight • May need more frequent maintenance | Low | <ul style="list-style-type: none"> • Additional creep settlement • Entire road section would require temporary regarding • Not considered feasible |
| Excavate and replace with lightweight fill | <ul style="list-style-type: none"> • Weight is removed • Can mitigate long-term settlement • More durable solution | <ul style="list-style-type: none"> • May involve significant excavation • Requires roadway protection and groundwater control | Medium | <ul style="list-style-type: none"> • Groundwater control issues • Future settlements still anticipated, but significantly smaller than previous |
| Soil improvement | <ul style="list-style-type: none"> • Minimum excavation • More durable solution | <ul style="list-style-type: none"> • Mobilization cost can be high for a small site • Likely long construction time | High | <ul style="list-style-type: none"> • Impact on other lanes • Significant construction effort • Full depth of peat not known • Not considered feasible |

The expanded polystyrene (EPS) approach to achieve a 15 kPa stress reduction on the peat would need to consider the following.

- Final roadway surface 219.96 m
- Minimum pavement structure over EPS 0.61 m
- EPS thickness 0.90 m
- Maximum bottom el. of EPS 218.45 m
- Groundwater level 218.20 m
- Cost (material) ±\$175/m³

The cellular concrete approach to achieve a 15 kPa stress reduction on the peat would involve:

- Final roadway surface 219.56 m
- Minimum pavement structure on cellular concrete 0.30 to 0.35 m
- Cellular concrete thickness 1.10 m
- Maximum bottom el. of cellular concrete 218.51 m
- Groundwater level 218.20 m
- Cost (material) \$200/m³

The key advantages of the cellular concrete when compared to EPS blocks are as follows:

- The EPS blocks require a flat prepared surface to sit upon, whereas the cellular concrete will configure to the prepared subgrade regardless of shape (it is self levelling).
- The cellular concrete eliminates the thickness of subbase gravel required for the pavement.
- The EPS blocks require a protection cover, typically concrete.
- Once the excavation has been prepared, cellular concrete placement can be accomplished within a day. Based on a case study published by Cematrix, a production rate of 120 m³ can be achieved and a similar project on Dixie Road in the region of Peel was completed within one week.

Given the construction schedule advantages, it is recommended that a cellular concrete approach be carried forward.

8.4 LONG TERM REPAIR RECOMMENDATION

It is recommended that the local repair be carried out using cellular concrete. The planning and design of the repair should consider the following:

- Repair the two eastbound lanes and the shoulder up to the shoulder rounding (approximate width of 10 m).
- Repair for the full length of the settled area (approximately 60 m).
- MTO pavement designers to confirm the asphalt layers required based on traffic volumes.
- The base gravel placed over the cellular concrete should consist of 200 mm OPSS Granular "O".
- Cellular concrete with a 28 day compressive strength of 1.0 MPa is recommended at this time. A lower strength cellular concrete may be considered based on pavement design requirements, but should not be less than 0.71 MPa.
- A cellular concrete thickness of 1.1 m is recommended to provide an approximate stress reduction of 15 kPa.
- After excavation to the target depth, the exposed subgrade must be compacted to 95% of standard Proctor maximum dry density.
- The depth of excavation is anticipated to be about 1.5 m below the roadway profile, therefore traffic should be maintained at 2.5 m away from the top of excavation.
- The existing pavement materials within the frost zone were non-frost susceptible, and therefore frost transitions are not anticipated to be required. Nevertheless, it is recommended that end treatments be developed to provide a stiffness transition within the adjacent pavement support.
- End treatments should include
 - Granular B Type II extending from the base of the cellular concrete 2 m in length, then followed by a 10H:1V transition up to the existing subbase level.
 - Provision of a biaxial geogrid placed at 0.3 m below the top of the cellular concrete, extending from 1.5 m into the cellular concrete and 8 m into the Granular B Type II transition.
 - The biaxial geogrid should have a minimum tensile strength of 548 lbs/ft (8.0 kN/m) at 2% strain in the machine direction.

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- The pavement designers will need to consider the lateral drainage impacts. It is recommended that subdrains be incorporated on both sides of the cellular concrete with an invert of at least 1.0 m below top of pavement.
- The top of the cellular concrete should be provided with a 2% cross-fall.

8.5 CONSTRUCTION STAGING (LONG TERM REPAIR)

The proposed roadway repair requires closure of the two eastbound lanes of Highway 9 plus a 2.5 m offset for traffic flow. No construction staging is anticipated.

8.6 SHORT TERM REPAIR RECOMMENDATION

It is recommended that the short term repair consist of raising the grade using hot mix asphalt. As discussed in Section 8.2.3 it should be anticipated that up to 115 mm of settlement would occur in a two year period. The long term repair strategy should be applied within two years or alternatively, the short term repair should be reapplied at the end of two years.

Surface preparation, tack-coat, and pavement type details should be obtained from MTO's Geotechnical Section.

9.0 References

ASTM. 1999. Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils (ASTM D1586). ASTM International, West Conshohocken, PA.

ASTM. 2000. Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System) (ASTM D2487). ASTM International, West Conshohocken, PA.

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

Terraprobe. 1999. Foundation Investigation Report Highway 9 from West Canal Bank Road to 0.5 km West of Weston Road. Technical Report, July 1999.

Thurber Engineering Ltd. 1997. Foundation Investigation Report for Highway 9 – Holland Drainage Canal East Bridge Widening. Technical Report, May 1997.

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10.0 Closure

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

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

This report was prepared by Simon Gudina and reviewed by Raymond Haché, MTO Designated Principal Contact for Foundation Engineering.

Respectfully submitted,

STANTEC CONSULTING LTD.



Simon Gudina, Ph.D., P.Eng.
Geotechnical Engineer



Raymond Haché, M.Sc., P.Eng.
Designated Principal MTO Foundation Contact



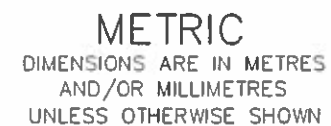
v:\01224\active\1224109xx\122410967\report\final\rpt_hwy9_settlement_fnd.docx

APPENDIX A

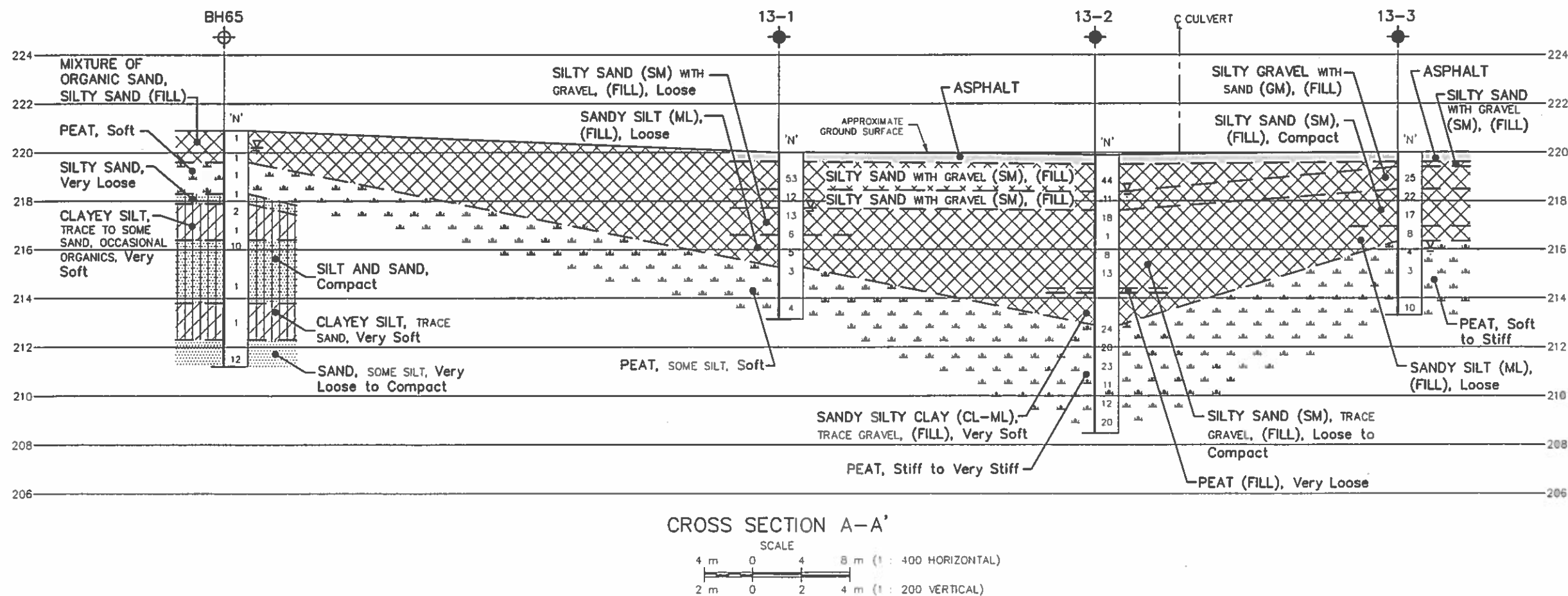
Drawing No. 1 – Borehole Location Plan and Soil Strata Plot


Site Photos

Historic Soil Data from Nearby Sites



HIGHWAY 9
1.4 km WEST OF HWY 400
BOREHOLE LOCATIONS & SOIL STRATA






| | | | |
|---|--|---------------|--------------------------|
|  | Project No.: 122410967 | WO 2013-11019 | Site Photographs |
| | Project : Highway 9 Settlement west of Highway 400, Township of King, ON | | Date: September 12, 2013 |

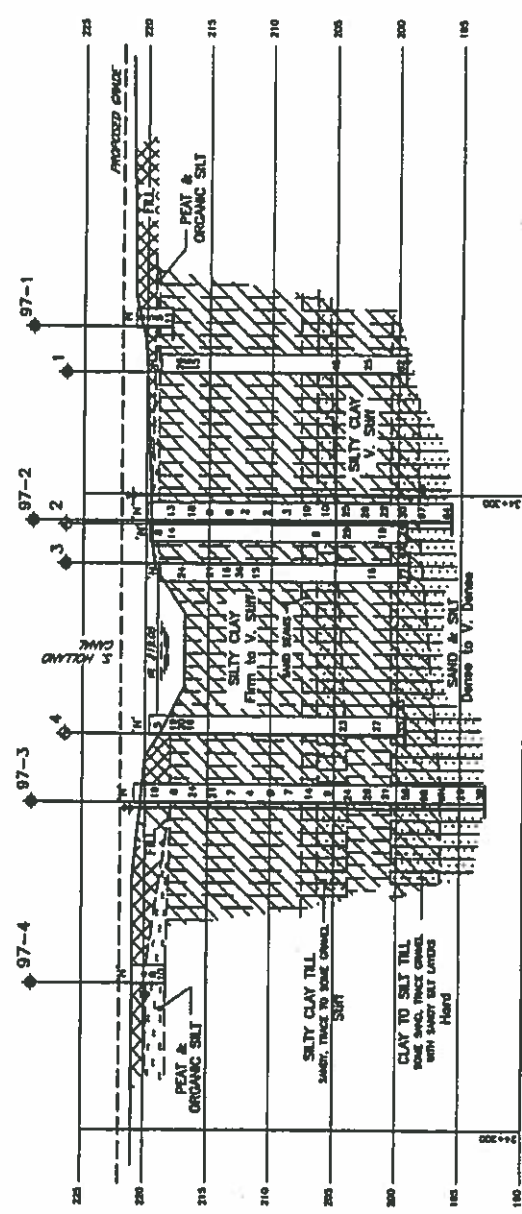
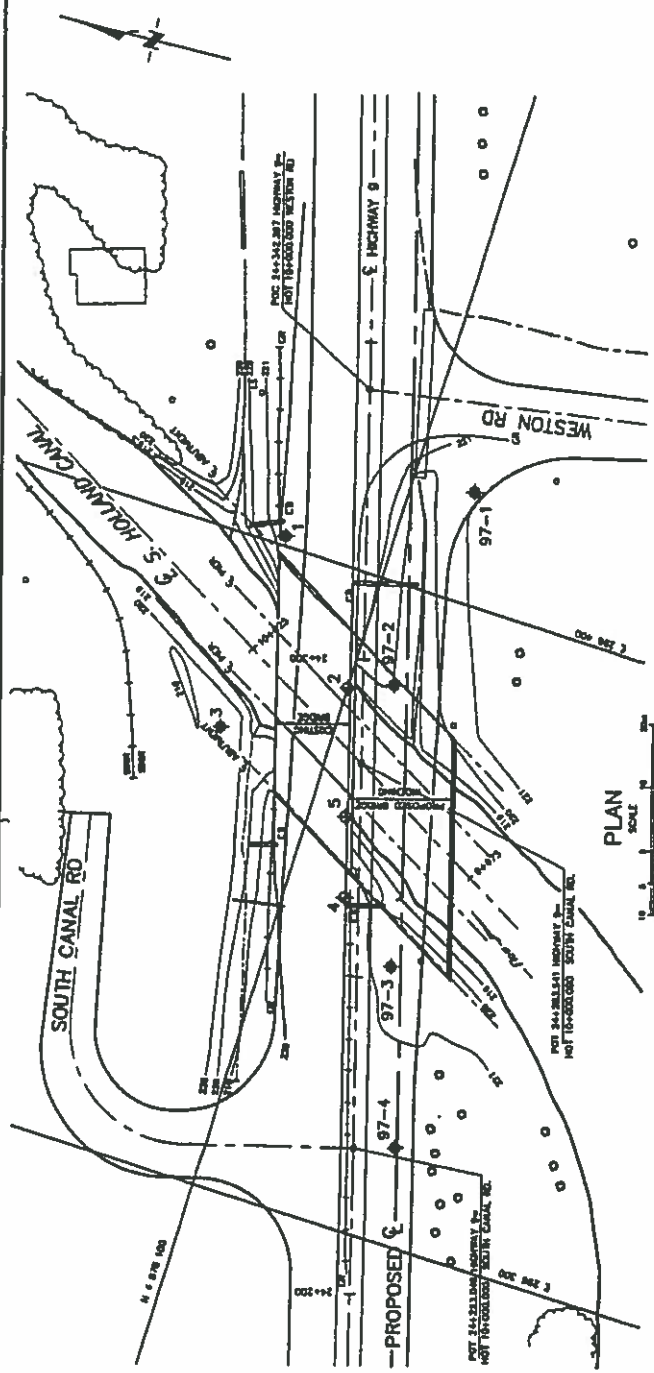


Site Photo No.: 1 Looking southeast along Highway 9 at the site



Site Photo No.: 2 Looking southwest along Highway 9 at the site

| | | | |
|---|--|---------------|--------------------------|
|  | Project No.: 122410967 | WO 2013-11019 | Site Photographs |
| | Project : Highway 9 Settlement west of Highway 400, Township of King, ON | | Date: September 12, 2013 |
| |  | | |
| Site Photo No.: 3 | Looking south towards the settled Hwy 9 Lane (near BH13-2) | | |
| |  | | |
| Site Photo No.: 4 | Looking northeast at the site | | |



| PROFILE | PROPOSED | £ |
|---------|----------|---|
|---------|----------|---|



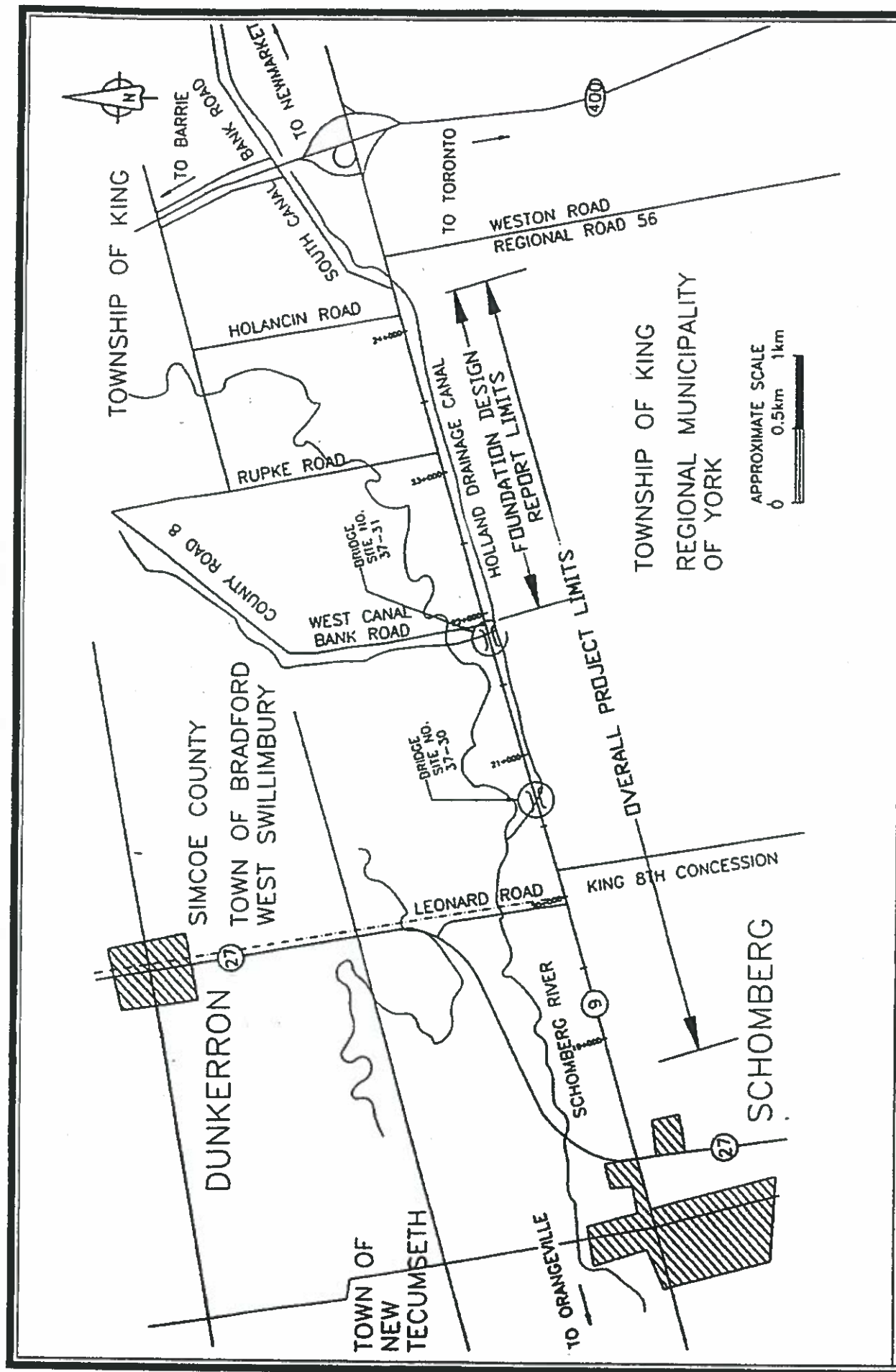
| LEGEND | | CO-CORRELATION | | DATE |
|--------|-------------|----------------|-------------|------------|
| No | DESCRIPTION | NO | DESCRIPTION | DATE |
| 97-1 | 220.0 | 4870061.120 | 4870061.120 | 290417.330 |
| 97-2 | 210.0 | 4870061.127 | 4870061.127 | 290404.550 |
| 97-3 | 220.0 | 4870061.136 | 4870061.136 | 290402.310 |
| 97-4 | 221.1 | 4870071.030 | 4870071.030 | 290413.254 |
| 1 | 210.0 | 4870110.0 | 4870110.0 | 290401.5 |
| 2 | 210.7 | 4870100.0 | 4870100.0 | 290302.0 |
| 3 | 210.0 | 4870117.0 | 4870117.0 | 290307.0 |
| 4 | 210.2 | 4870080.0 | 4870080.0 | 290308.0 |
| 5 | 210.0 | | | 290402.0 |

—NOTE—

The boundaries between and within have been established only at 5000 ft. altitudes. Deeper there the boundaries are obscured from geological evidence.

NOTE:The complete business investigation and design report for this project and other related documents may be obtained at the Engineering Technical office, Department, information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102-2 of Form 101.

[illegible]



SITE KEY PLAN

Geocres 31D-373

APPENDIX B

Symbols and Terms Used on Borehole Records

Borehole Records

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

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

Terminology describing soil structure:

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

Terminology describing soil types:

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

Terminology describing cobbles, boulders, and non-matrix materials (organic matter or debris):

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

| | |
|-----------------------------|---------------|
| <i>Trace, or occasional</i> | Less than 10% |
| <i>Some</i> | 10-20% |
| <i>Frequent</i> | > 20% |

Terminology describing compactness of cohesionless soils:

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

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

Terminology describing consistency of cohesive soils:

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

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

ROCK DESCRIPTION

Terminology describing rock quality:

| RQD | Rock Mass Quality |
|--------|-------------------|
| 0-25 | <i>Very Poor</i> |
| 25-50 | <i>Poor</i> |
| 50-75 | <i>Fair</i> |
| 75-90 | <i>Good</i> |
| 90-100 | <i>Excellent</i> |

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

Terminology describing rock mass:

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

Terminology describing rock strength:

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

Terminology describing rock weathering:

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

STRATA PLOT

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



Boulders
Cobbles
Gravel



Sand



Silt



Clay



Organics



Asphalt



Concrete



Fill



Bedrock

SAMPLE TYPE

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

WATER LEVEL MEASUREMENT



measured in standpipe,
piezometer, or well



inferred

RECOVERY

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

N-VALUE





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

DYNAMIC CONE PENETRATION TEST (DCPT)

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

OTHER TESTS

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

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



RECORD OF BOREHOLE No BH13-1

1 OF 1

METRIC

W.P. _____ LOCATION _____ N: 4 876 174 E: 295 919 ORIGINATED BY DS
 DIST _____ HWY 9 _____ BOREHOLE TYPE Hollow stem augers, splitspoon sampler COMPILED BY KF
 DATUM Geodetic DATE 2013 09 11 - 2013 09 11 CHECKED BY _____

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | PLASTIC LIMIT W _P | NATURAL MOISTURE CONTENT W | LIQUID LIMIT W _L | UNIT WEIGHT γ | REMARKS & GRAIN SIZE DISTRIBUTION (%) | | | |
|---------------|---|------------|---------|------|------------|----------------------------|-----------------|---|----|----|----|-----|------------------------------------|-------------------------------------|-----------------------------------|-------------------------|---|-------------------|--|--|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | | | | | | WATER CONTENT (%) | | |
| | | | | | | | | | | | | | | | | | | | | |
| 220.0 | Asphalt | | | | | | | 20 | 40 | 60 | 80 | 100 | | | | | | | | |
| 0.0 | 380 mm ASPHALT | | | | | | | | | | | | | | | | | | | |
| 219.6 | | | 1 | BS | | | | | | | | | | | | | | | | |
| 0.4 | FILL: Silty sand with gravel (SM) Brown | | | | | | | | | | | | | | | | | | | |
| | | | 2 | SS | 53 | | 219 | | | | | | | | | | 27 57 (16) | | | |
| 218.5 | | | | | | | | | | | | | | | | | | | | |
| 1.5 | FILL: Silty gravel with sand (GM) Brown, moist | | 3 | SS | 12 | | 218 | | | | | | | | | | | | | |
| 217.7 | | | | | | | | | | | | | | | | | | | | |
| 2.3 | FILL: Silty sand (SM) with gravel Loose Brown, moist to wet - traces of peat | | 4 | SS | 13 | | 217 | | | | | | | | | | 18 51 (31) | | | |
| 216.6 | | | | | | | | | | | | | | | | | | | | |
| 3.4 | FILL: Sandy silt (ML) Loose Brown, mosit | | 5 | SS | 6 | | 216 | | | | | | | | | | | | | |
| | | | 6 | SS | 5 | | | | | | | | | | | | | | | |
| 215.3 | | | | | | | | | | | | | | | | | | | | |
| 4.7 | PEAT Soft Dark brown, wet - some silt | | 7 | SS | 3 | | 215 | | | | | | | | | 132 | | | | |
| | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | |
| | | | | | | | | | | | | | | | | | | | | |
| | | | 8 | SS | 4 | | 214 | | | | | | | | | 79 | Org M = 48% | | | |
| 213.1 | | | | | | | | | | | | | | | | | | | | |
| 6.9 | End of Borehole | | | | | | | | | | | | | | | | | | | |

STN13-ONTARIO MTO STANTEC 122410967 - HWY 9 1.5KM WEST OF HWY 400.GPJ ONTARIO MOT.GDT 14/1/14

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



RECORD OF BOREHOLE No BH13-2

1 OF 1

METRIC

W.P. _____ LOCATION _____ N: 4 876 181 E: 295 944 ORIGINATED BY DS
DIST _____ HWY 9 _____ BOREHOLE TYPE Hollow stem augers, splitspoon sampler COMPILED BY KF
DATUM Geodetic DATE 2013 09 12 - 2013 09 12 CHECKED BY _____

| SOIL PROFILE | | | SAMPLES | | | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT | | | | | UNIT WEIGHT γ kN/m ³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) | |
|---------------|---|------------|---------|------|------------|----------------------------|-----------------|---|--|--|--|--|---|---|--------------------------|
| ELEV DEPTH | DESCRIPTION | STRAT PLOT | NUMBER | TYPE | "N" VALUES | | | SHEAR STRENGTH kPa | | | | | | | |
| | | | | | | | | 20 40 60 80 100 | | | | | | | |
| 219.9 | Asphalt | | | | | | | | | | | | | | |
| 0.0 | 390 mm ASPHALT | | 1 | BS | | | | | | | | | | | |
| 219.5 | | | | | | | | | | | | | | | |
| 0.4 | FILL: Silty sand with gravel (SM) Brown | | | | | | | | | | | | | | |
| | | | 2 | SS | 44 | | | | | | | | | | |
| 218.4 | | | | | | | | | | | | | | | |
| 1.5 | FILL: Silty gravel with sand (GM) Brown to grey, moist - polypropylene geogrid at bottom of fill | | 3 | SS | 11 | | | | | | | | | | 46 36 (18) |
| 217.6 | | | | | | | | | | | | | | | |
| 2.3 | FILL: Silty sand (SM) Loose to compact Brown, moist to wet Trace gravel | | 4 | SS | 18 | | | | | | | | | | |
| | | | 5 | SS | 1 | | | | | | | | | | |
| | | | 6 | SS | 8 | | | | | | | | | | 7 52 (41) |
| | | | 7 | SS | 13 | | | | | | | | | | |
| 214.4 | | | | | | | | | | | | | | | |
| 214.5 | FILL: Peat Very loose Brown | | 8 | SS | WH | | | | | | | | | | |
| 5.7 | FILL: Sandy silty clay (CL-ML) Very soft Brown, moist to wet - trace gravel | | 9 | SS | WH | | | | | | | | | | 1 38 47 14 |
| 212.9 | | | | | | | | | | | | | | | |
| 7.0 | PEAT Stiff to very stiff Dark brown, moist to wet - traces of sand and wood - mottled with grey fine sandy silt throughout | | 10 | SS | 24 | | | | | | | | | | |
| | | | | | | | | | | | | | | | |
| | | | 11 | SS | 20 | | | | | | | | | | Org M = 63% |
| | | | | | | | | | | | | | | | |
| | | | 12 | SS | 23 | | | | | | | | | | 0 3 89 8 Non-plastic |
| | | | | | | | | | | | | | | | |
| | | | 13 | SS | 11 | | | | | | | | | | |
| | | | | | | | | | | | | | | | |
| | | | 14 | SS | 12 | | | | | | | | | | WH = Weight of hammer |
| | | | | | | | | | | | | | | | |
| | | | 15 | SS | 20 | | | | | | | | | | Org M = 54% |
| 208.5 | | | | | | | | | | | | | | | |
| 11.4 | End of Borehole | | | | | | | | | | | | | | |

\times^3, \times^3 : Numbers refer to Sensitivity \circ 3% STRAIN AT FAILURE

STN13-ONTARIO MTO STANTEC 122410967 - HWY 9 1.5KM WEST OF HWY 400.GPJ ONTARIO MOT.GDT 14/1/14

| | | | |
|------------------|---|-------------------------|------------------|
| W.P. _____ | LOCATION _____ | N: 4 876 189 E: 295 968 | ORIGINATED BY DS |
| DIST _____ HWY 9 | BOREHOLE TYPE Solid stem augers, splitspoon sampler | | COMPILED BY KF |
| DATUM Geodetic | DATE 2013 09 11 - 2013 09 11 | | CHECKED BY _____ |

[illegible]

STN13-ONTARIO MTO STANTEC 122410967 - HWY 9 1.5KM WEST OF HWY 400.GPJ ONTARIO MOT.GDT 14/1/14

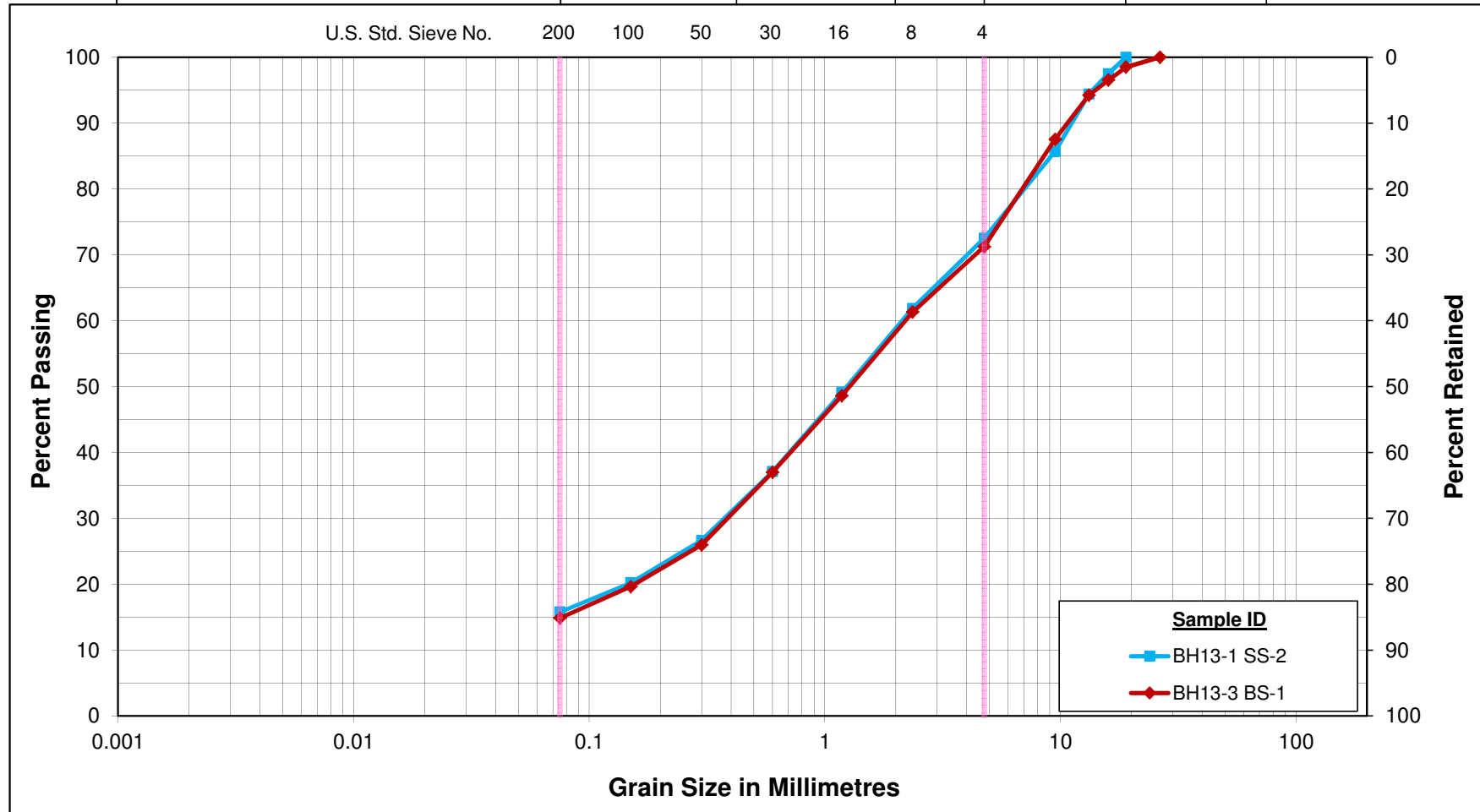
✕³, ✕³: Numbers refer to Sensitivity
 ○^{3%} STRAIN AT FAILURE

APPENDIX C

Laboratory Test Results

Unified Soil Classification System

| CLAY & SILT | SAND | | | Gravel | |
|-------------|------|--------|--------|--------|--------|
| | Fine | Medium | Coarse | Fine | Coarse |



GRAIN SIZE DISTRIBUTION

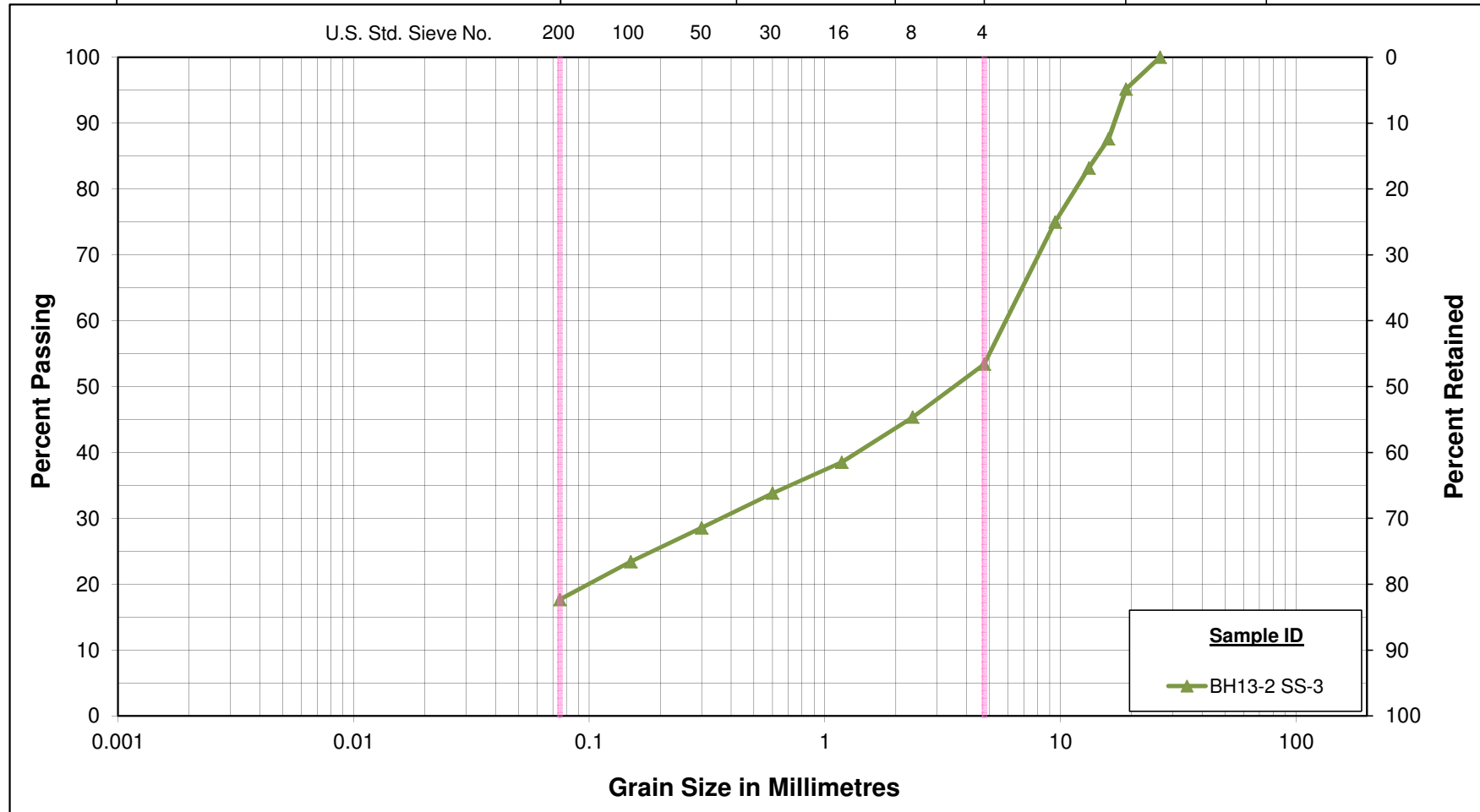
FILL: Silty sand with gravel (SM)

Figure No. 1a

Project No. 122410967

Unified Soil Classification System

| CLAY & SILT | SAND | | | Gravel | |
|-------------|------|--------|--------|--------|--------|
| | Fine | Medium | Coarse | Fine | Coarse |



GRAIN SIZE DISTRIBUTION

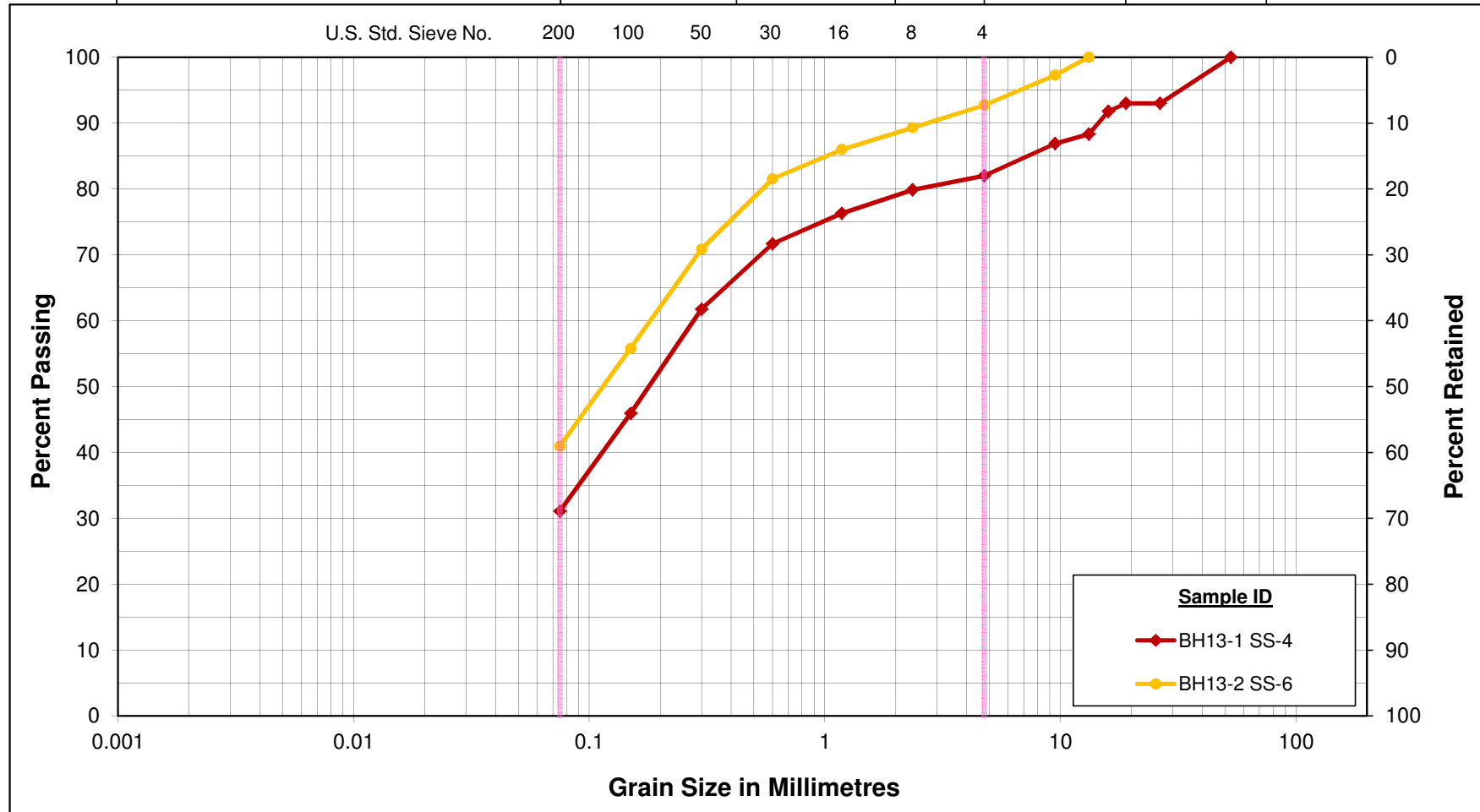
FILL: Silty gravel with sand (GM)

Figure No. 1b

Project No. 122410967

Unified Soil Classification System

| CLAY & SILT | SAND | | | Gravel | |
|-------------|------|--------|--------|--------|--------|
| | Fine | Medium | Coarse | Fine | Coarse |



GRAIN SIZE DISTRIBUTION

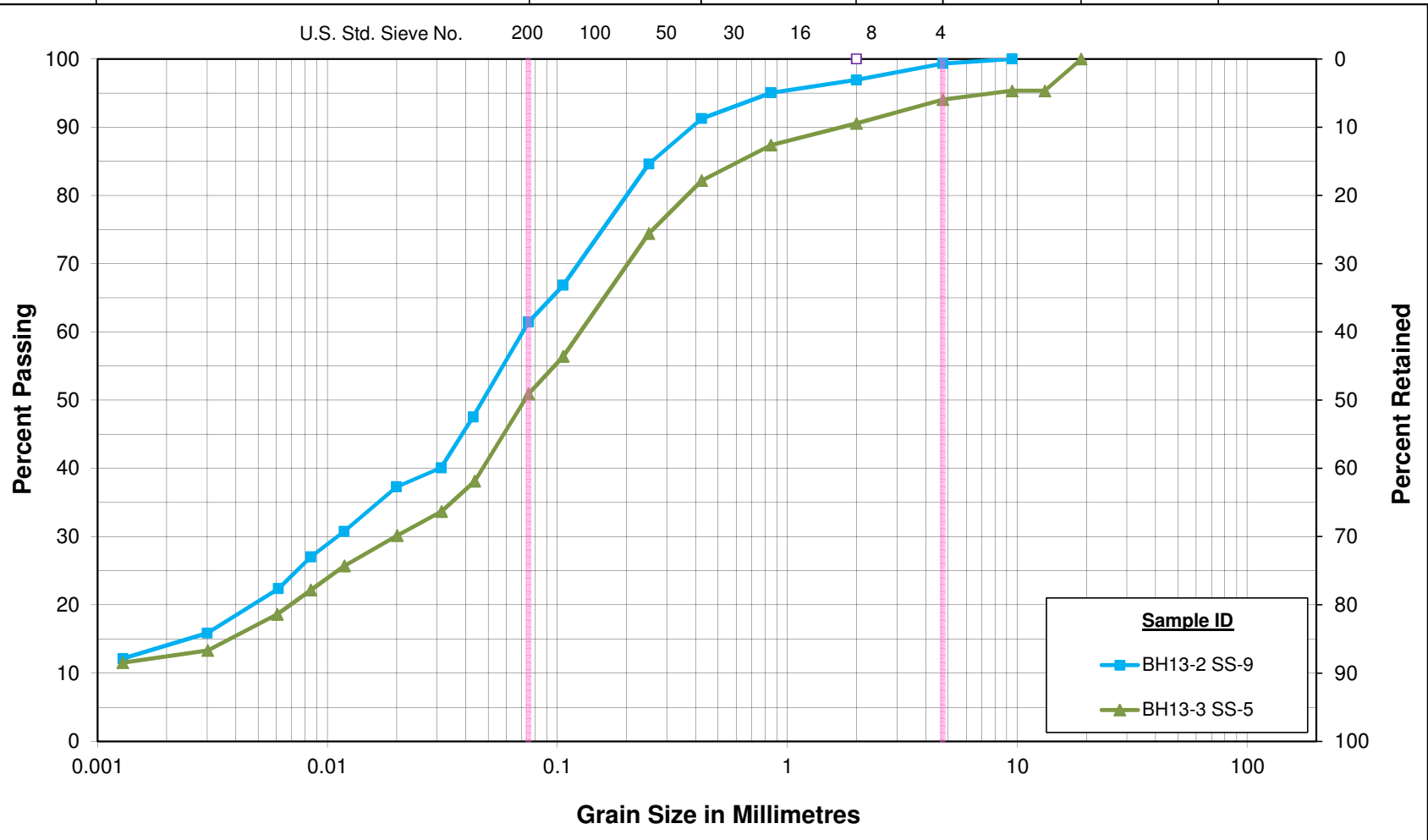
FILL: Silty sand gravel (SM)

Figure No. 2

Project No. 122410967

Unified Soil Classification System

| CLAY & SILT | SAND | | | Gravel | |
|-------------|------|--------|--------|--------|--------|
| | Fine | Medium | Coarse | Fine | Coarse |



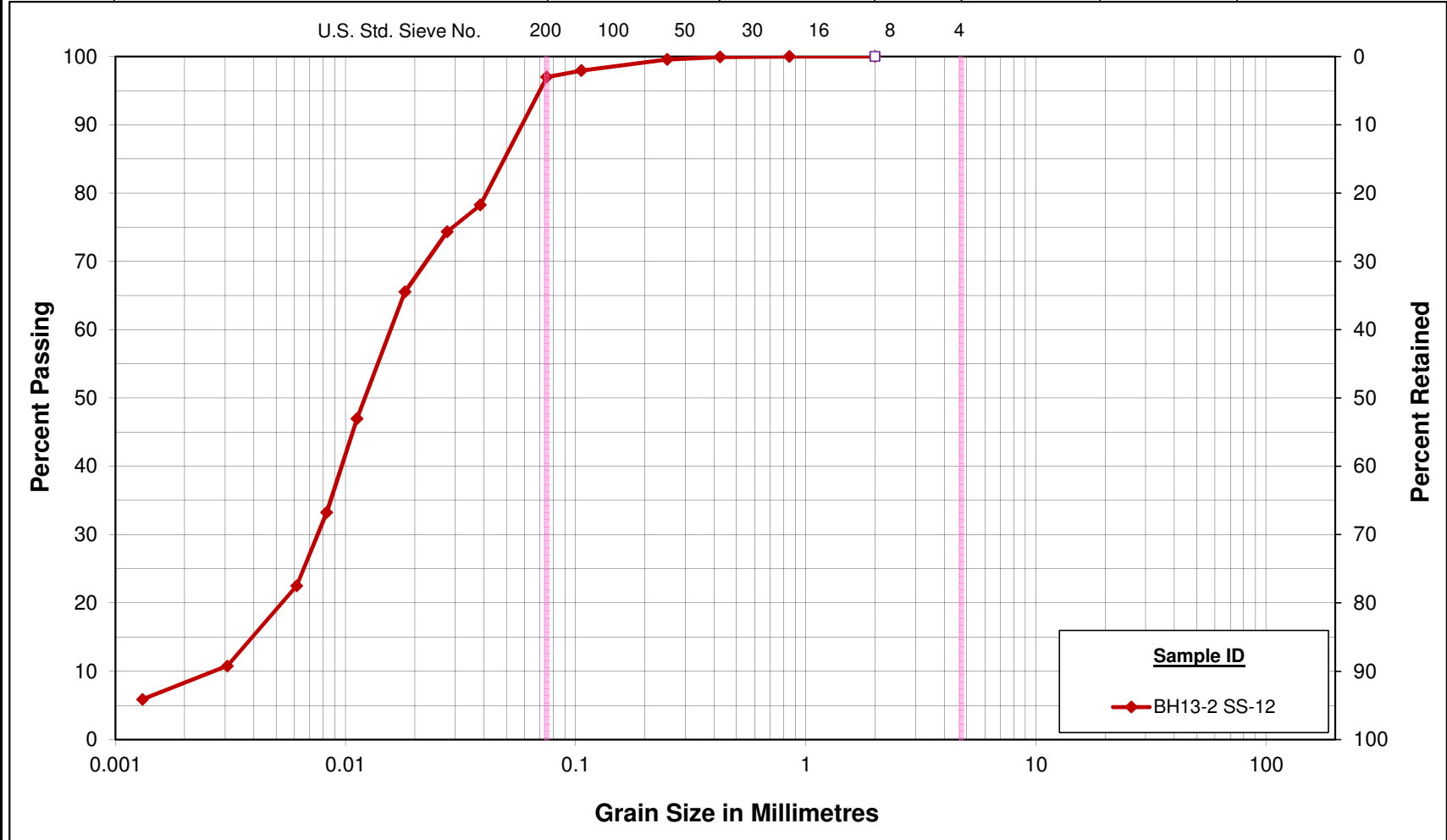
GRAIN SIZE DISTRIBUTION
 FILL: Sandy silt (ML) and Sandy silty clay (CL-ML)

Figure No. 3

Project No. 122410967

Unified Soil Classification System

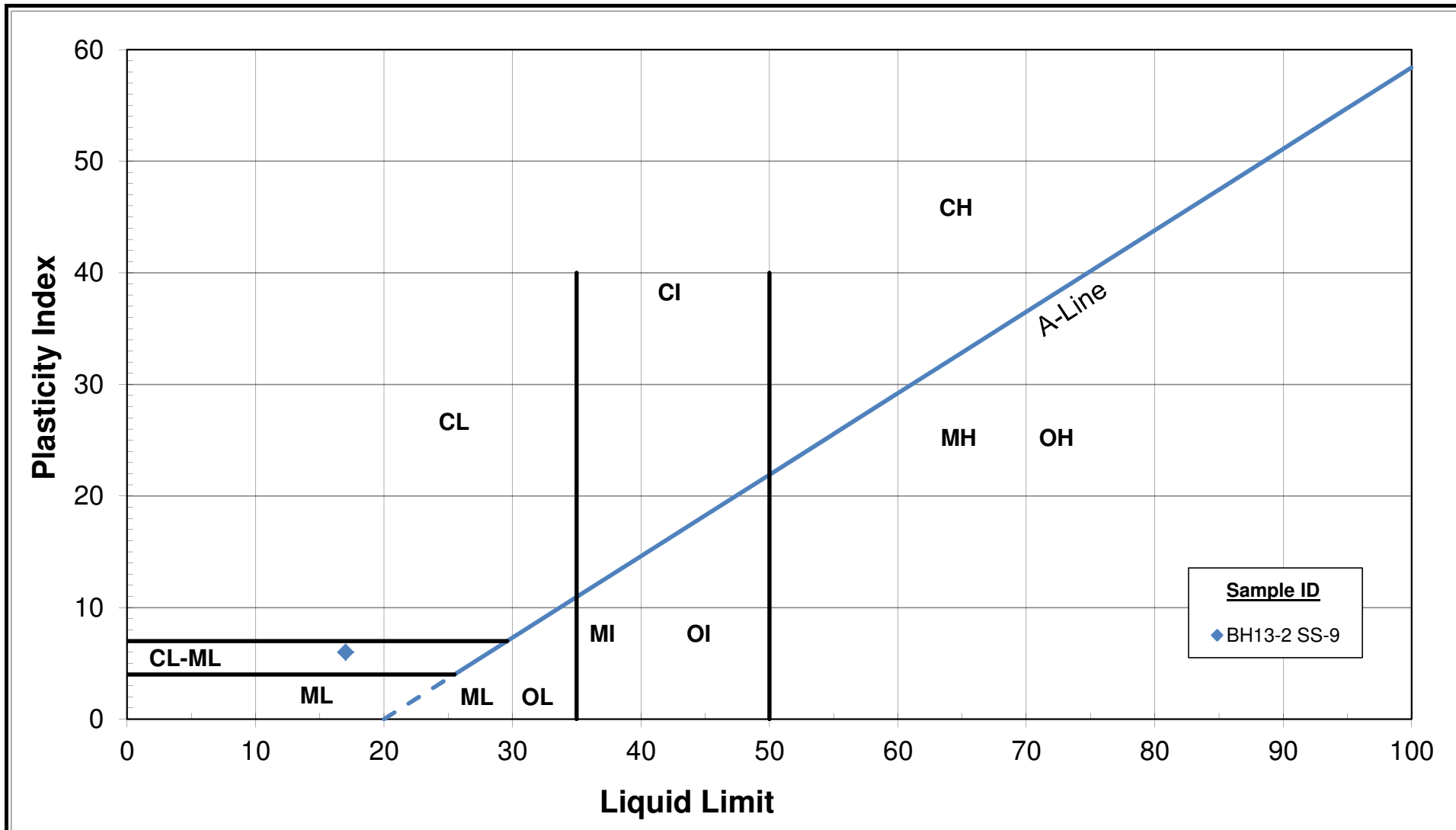
| CLAY & SILT | SAND | | | Gravel | |
|-------------|------|--------|--------|--------|--------|
| | Fine | Medium | Coarse | Fine | Coarse |



GRAIN SIZE DISTRIBUTION
Peat (PT) (Organic Silt (OL))

Figure No. 4

Project No. 122410967



APPENDIX D

Settlement Profile along the Right EBL

General Settlement Interpretation

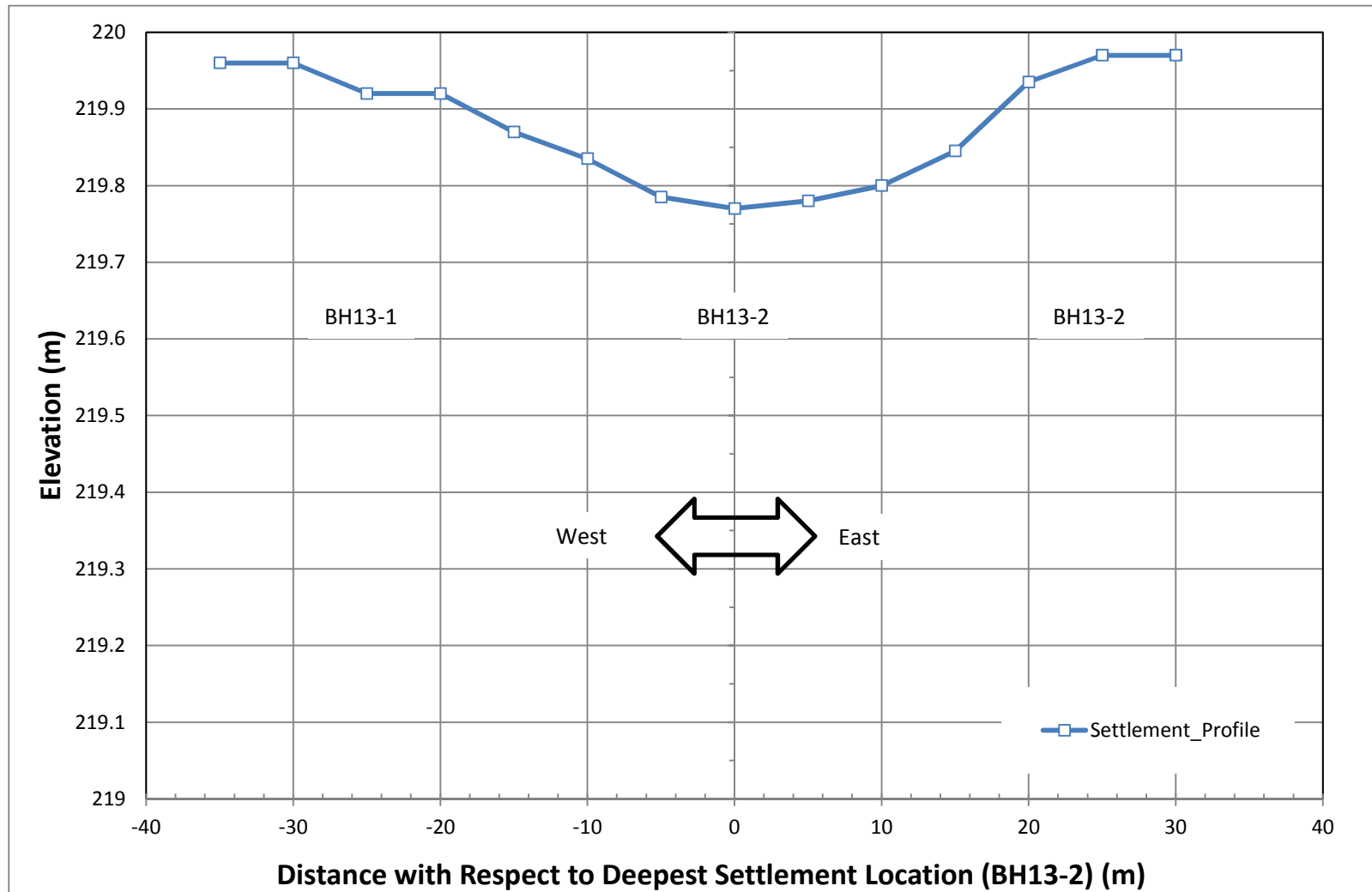
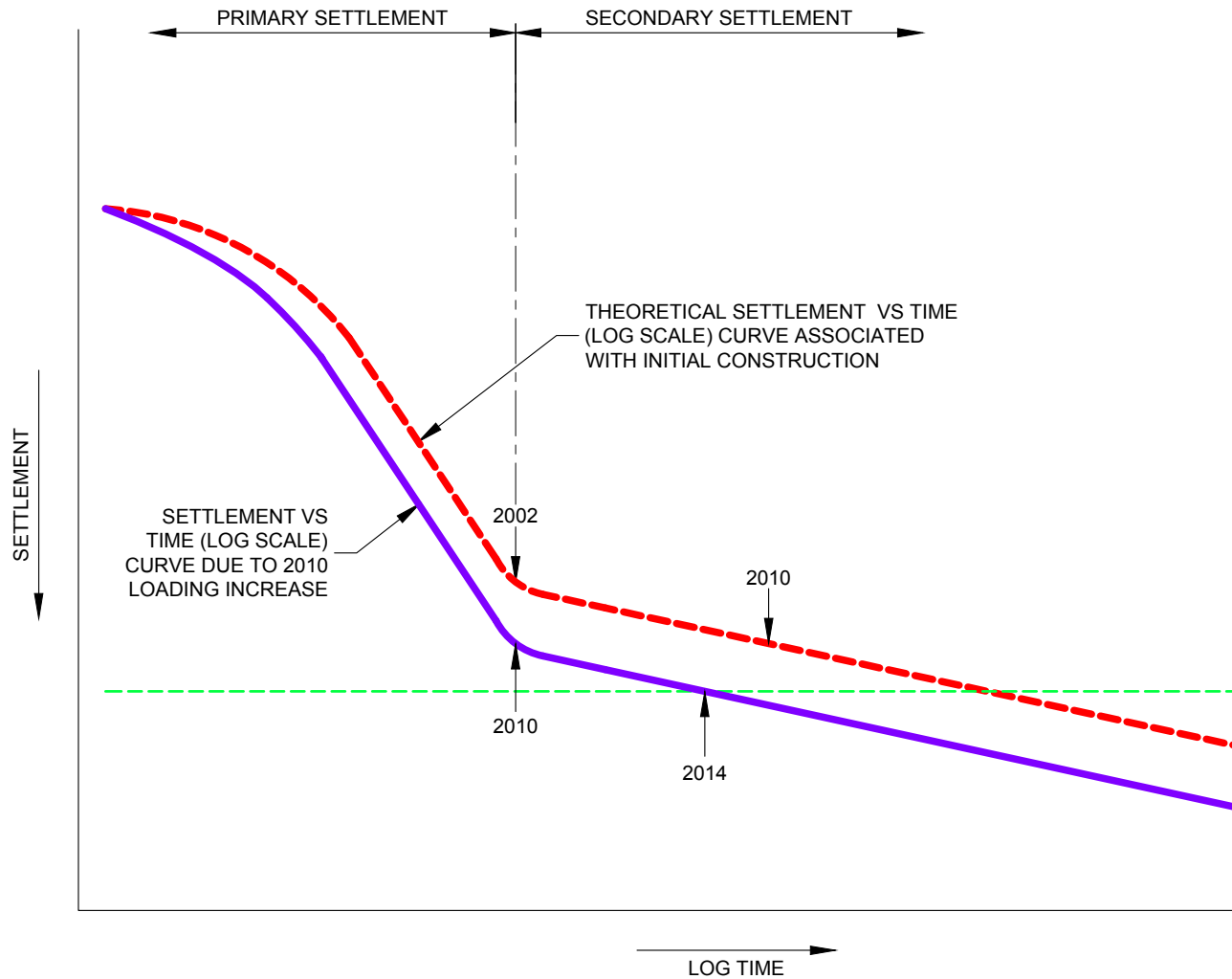


Figure 6
Settlement Profile along the Right EBL



GENERAL INTERPRETATION METHOD :

1. THE INITIAL CONSTRUCTION OF THE WIDENING WOULD HAVE FOLLOWED THE RED SETTLEMENT VERSUS LOG-TIME CURVE.
2. THE PRIMARY CONSOLIDATION OF THE PEAT WOULD HAVE BEEN COMPLETED WITHIN 2 TO 3 WEEKS OF ACHIEVING THE FINAL GRADE.
3. THE SECONDARY SETTLEMENT BETWEEN 2002 AND 2010 WOULD HAVE BEEN SOME AMOUNT GREATER THAN 200 mm. THIS IS ASSUMED SINCE THE SECONDARY SETTLEMENT BETWEEN 2010 AND 2014 IS IN THE ORDER OF 200 mm.
4. IN 2010, RECONSTRUCTION AND REPROFILING WOULD HAVE OCCURRED AS PART OF THE CULVERT REPAIR WORK.
5. REPROFILING OF THE ROADWAY (2010) WOULD HAVE INTRODUCED A NEW LOADING RESULTING IN A REINITIATION OF SECONDARY SETTLEMENT PROCESS.
6. THE CURRENT RATE OF SECONDARY CONSOLIDATION HAS BEEN EVALUATED AS 200 mm/LOG (1460 DAYS/20 DAYS) .
7. THE DESIGN APPROACH IS TO REDUCE THE LOADING ON THE PEAT WHICH WILL TEMPORARILY HALT THE SECONDARY SETTLEMENT.

January, 2014
Project No. 122410967



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Ottawa, ON, Canada K2C 3G4
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Client/Project
MTO
Highway 9
1.4 km West of Highway 400

Figure No.

7

Title

General Settlement
Interpretation