



# Englobe

Soils Materials Environment

**Submitted To AECOM Canada Ltd.  
189 Wyld Street Suite 103, North Bay, Ontario P1B 1Z2  
On Behalf of the Ontario Ministry of Transportation**

**Bridge Rehabilitation – Hawk Creek Bridge  
Highway 547  
Stations 13+727 to 13+758 – Township of Esquega  
Site No. 38C-016  
GWP 5377-11-00**

## **FINAL FOUNDATION INVESTIGATION AND DESIGN REPORT**

Date: August 12, 2015  
Ref. N°: 15/04/15026-F1

**Geocres No. 42C-35**



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## Final Foundation Investigation and Design Report

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Test results mentioned herein are only valid for the sample(s) stated in this report.

Englobe's subcontractors who may have accomplished work either on site or in laboratory are duly qualified as stated in our Quality Manual's procurement procedure. Should you require any further information, please contact your Project Manager.

### Client:

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Attention: Mr. Al Rose

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## 1 INTRODUCTION

LVM-Merlex, a Division of Englobe Corp. (now known as Englobe Corp.), has been retained by AECOM Canada Ltd., on behalf of the Ministry of Transportation of Ontario (MTO), to carry out a foundation investigation to supply subsurface data for the design of a protection system to be implemented at the existing Hawk Creek Bridge during the proposed rehabilitation and conversion to semi integral abutments. The bridge is located on Highway 547, some 3.7 km north of the intersection between Highway 101 and Highway 547 in the Township of Esquega (see Drawing No. 1 in Appendix 1).

The foundation investigation location was specified by the MTO in the Terms of Reference for work under Agreement No. PO 5013-E-0051: GWP 5377-11-00 for delivering a submission of Design-Build Contract package with an option of completing detailed design for a conventional Detailed Design Contract package. The terms of reference for the original scope of work are outlined in LVM-Merlex's Proposal P-14-175 dated November 7, 2014. After the kick-off meeting held on April 27, 2015, MTO decided that the scope of work be revised for a Detailed Design Contract package. Accordingly the revised terms of reference for the foundation investigation are outlined on LVM-Merlex's letter dated June 22, 2015 (LVM-Merlex Reference No.:15/04/15026-R2).

The purpose of this investigation was to determine the subsurface conditions in the area of the bridge approaches in order to provide factual subsurface information and design recommendations for a protection system to be implemented during rehabilitation activities. Englobe investigated the foundation area by the drilling of boreholes, carrying out in-situ tests, and performing laboratory testing on select samples.

## 2 SITE DESCRIPTION

The Hawk Creek Bridge is located on Highway 547, between approximately Stations 13+727 to 13+758, Township of Esquega (Site No. 38C-016). The bridge is a two-span structure, 31 m in length along the centerline and was constructed in 1964. The topography at this site is located in an area with generally level terrain. The existing approach embankments for the bridge currently support two undivided lanes of highway, running in a south-north direction. The Hawk Creek flows from the east to the west at the bridge location (right to left). A visual review of the highway pavement surface, to the north and south of the bridge, indicates that, in general, the approaches are in fair to good condition (see Photo Essay in Appendix 4).

At the bridge location, the existing highway centerline is at elevation 316.5 m at both ends of the bridge. The highway pavement structure is constructed on the sand fill of the approach embankments, which overlie the natural earth deposits. The existing approach embankments

extending out from the existing concrete wing walls, in the area of the bridge, have been built on slope angles of approximately 2H:1V to 2.5H:1V.

## 2.1 SITE PHYSIOGRAPHY AND SURFICAL GEOLOGY

This project is located in the Geomorphic Sub-province known as the Eastern Sandy Uplands. The topography on this section of Highway 547 is generally flat. Significant layers of earth overlay the bedrock. Within the project area native overburden primarily consists of sandy silt to silty sand.

Bedrock in the area consists of the granitic rocks of early Precambrian Age.

## 3 INVESTIGATION PROCEDURES

The fieldwork for this investigation was carried out between June 11<sup>th</sup> and June 12<sup>th</sup>, 2015 during which time two (2) sampled boreholes were advanced through the existing approach slab and the approach embankment at each end of the bridge.

The field investigation was carried out using a truck and/or bombardier mounted CME drilling rig equipped with hollow stem augers, standard augers, casing equipment, and routine geotechnical sampling equipment. Soil samples were obtained at the borehole locations at regular intervals of depth using the standard 50 mm O.D. split spoon sampler advanced in accordance with the Standard Penetration Test (SPT) procedures (ASTM D-1586). The SPT method involves advancing a 50 mm O.D. split spoon sampler with the force of a 63.5 kg hammer freely dropping 760 mm. The number of blows per 300 mm penetration was recorded as the “N” value. When cohesive deposits were encountered, the in-situ strength was measured using an “N” size field vane, vane collar, and calibrated torque meter. All samples taken during this investigation were stored in labeled airtight containers for transport to our North Bay laboratory for visual examination and select laboratory testing.

Groundwater conditions in the open boreholes were observed during the advancement of and immediately following, completion of the individual boreholes. A single 19 mm diameter standpipe was installed in one open borehole prior to backfilling to allow for further monitoring of the local groundwater level. All open boreholes were backfilled upon completion with compacted auger cuttings in the general order they were removed, and where necessary, bentonite pellet backfill was added to the boreholes to bring them up to grade in accordance with requirements of Ontario Regulation 903. At the borehole through the embankment, the upper portion of the hole, where necessary, was backfilled with an asphalt cold patch to seal the existing asphalt surface.

The fieldwork for this investigation was under the full time direction of a senior member of the Englobe engineering staff, who was responsible for locating the boreholes, clearing the borehole locations of underground services, in-situ sampling and testing operations, logging of the boreholes, labeling and preparation of samples for transport to our North Bay laboratory,

plus overall drill supervision. All samples received a visual confirmatory inspection in our laboratory. Laboratory testing of select samples included routine testing for natural moisture content determination and particle size analysis. The results of the laboratory testing are presented on the individual Record of Borehole Sheets (Appendix 2), with a summary of results presented on the laboratory sheets in Appendix 3 (Figures Nos. L-1 to L-2 and Table No. L-3).

The location of the individual boreholes was determined in the field using highway chainage (established by others) and offset relative to highway centerline. The MTO co-ordinates, northing and easting, were then established for the boring locations. Elevations contained in this report are referenced to a geodetic datum. The borehole elevations are based on a survey carried out by others.

## **4 SUBSURFACE CONDITIONS**

Details of the subsurface conditions revealed by the investigation program are presented on the enclosed Records of Borehole Logs (Appendix 2) and on Drawing No. 2 (Appendix 3). Please note that stratigraphic delineations presented on the borehole logs and soil strata plot are the results of non-continuous sampling, response to drilling progress, the results of SPT, plus field observations. Typically such boundaries represent transitions from one zone to another and are not an exact demarcation of specific geological unit. Additional consideration should be given to the fact that subsurface conditions may vary markedly between adjacent boreholes and beyond any specific boring location, and are shown on the drawings for illustration purposes only.

### **4.1 HAWK CREEK BRIDGE, TOWNSHIP OF ESQUEGA**

A plan and profile illustrating the borehole locations and stratigraphic sequences is shown on Drawing No. 2, Appendix 3. During the course of the exploration program, two (2) sampled boreholes were put down at this site, with Borehole No. 1 advanced behind the south abutment to the right of centerline (right side), and Borehole No. 2 advanced behind the north abutment to the right of centerline (right side).

At the time of the subsurface investigation, the ground surface elevations at Borehole Nos. 1 and 2 were recorded at elevations 316.5 m and 316.4 m, respectively.

#### **4.1.1 Pavement Structure**

At surface at Borehole No. 1, a pavement structure consisting of asphalt some 75 mm thick overlying a concrete slab 305 mm thick was penetrated. At Borehole No. 2, a pavement structure consisting of some 125 mm of asphalt overlying a concrete slab 305 mm thick was penetrated.

#### 4.1.2 Sand Fill

Underlying the concrete approach slab at Borehole Nos. 1 and 2, a layer of brown sand and gravel, mixed with rock fill, trace to some silt to sand, trace gravel trace silt was penetrated. The natural moisture contents measured on samples of this deposit recovered from Borehole Nos. 1 and 2 were in the order of 4% to 27%, with the exception of a natural moisture content measured at 34% on a sample recovered at a depth of some 5.5 m below ground surface (i.e. about 1.5 m above the bottom of the fill deposit) from Borehole No. 1. Gradation analyses were carried out on six (6) samples of this deposit, recovered from Borehole Nos. 1 and 2, the results of which indicated 2% to 45% gravel size particles, 47% to 87% sand size particles, and 6% to 19 % silt and clay size particles (Figure No. L-1, Appendix 3). Results of grain size distribution testing carried out on six samples recovered from Borehole Nos. 1 and 2 indicate that half of fill samples meets requirements of Granular "B" Type I stated in OPS.PROV 1010 and the fine material contents of three (3) samples exceed requirements. Rock pieces of the cobble and boulder sizes were encountered at depths ranging from 0.8 m to 5.3 m below ground surface, at locations of Borehole Nos. 1 and 2, and resulted in sampler refusals during the 3<sup>rd</sup> and the 7<sup>th</sup> SPT tests at the location of Borehole No.2. Based on SPT 'N' values of 6 to 78 blows per 300 mm to 65 blows per 23 mm penetration, the compactness of this deposit was described as loose to very dense. This deposit was encountered to depths of 7.1 m and 8.6 m below grade at Borehole Nos. 1 and 2, respectively (elevations 309.4 m and 307.8 m, respectively).

#### 4.1.3 Sand with Silt to Sand

Underlying the sand fill deposit at the locations of Borehole Nos.1 and 2, a layer of dark brownish grey to grey sand with silt trace clay to grey sand trace silt trace clay was penetrated. The natural moisture content measured on two samples of this deposit was in the order of 20% to 26% except the natural moisture content of one (1) sample was measured at 57 % due to trace organics and grass rootlets recovered from Borehole No. 1. Hydrometer analyses were carried out on two (2) samples of this deposit, the results of which indicated 0% to 9% gravel size particles, 64% to 93% sand size particles, 6% to 24% silt size particles, and 1% to 3% clay size particles (Figure No. L-2 in Appendix 3). Based on SPT 'N' values of 2 to 4 blows per 300 mm penetration, the compactness of this deposit was described as very loose. This deposit was encountered to depths of 9.8 m and 9.6 m below grade at locations of Boreholes Nos. 1 and 2, respectively (elevations 306.7 m and 306.8 m, respectively), where the boreholes were terminated.

#### 4.1.4 Previous Investigations

A previous foundation investigation, W.P. 144-61, was carried out at this location in 1961 by the Ontario Ministry of Transportation. Results of the previous investigation shown on a Drawing No. 61-F-120A indicated the subsurface soils on the river banks consisted of very loose silty sand overlying the loose to very dense silty sand to sandy silt. Refusal of Dynamic Cone Penetration Testing (DCPT) was countered at approximate elevation 290 m (see Enclosure No.



5, Appendix 5). Based on a Drawing No. TWP 682-16-2-A of Contract No. 63-39, the existing bridge was founded on deep foundations (324 mm diameter steel pipe piles driven to refusal at approximate elevation 288 m) at both of the north and the south abutments and the central pier (Enclosure No. 6, Appendix 5).

## **4.2 GROUNDWATER DATA**

The river water level was measured at elevation 312.0 m during the period of site investigation. Measurements of the groundwater table and cave-in levels were undertaken, where possible, in the open boreholes during the advance of the individual borings and upon completion. A standpipe was installed in Borehole No. 1 to obtain post borehole completion water level. These levels are recorded on the individual Record of Borehole Log Sheets (Appendix 2) and shown on the Borehole Locations and Soil Strata Drawing No. 2 in Appendix 3.

At the time of this investigation, the water levels were measured at elevations 311.7 m (June 12<sup>th</sup>, 2015) and 313.1 m (June 12<sup>th</sup>, 2015) at Borehole Nos. 1 and 2, respectively. It's noted that the groundwater level encountered at the location of Borehole No.2 was measured immediately after completion of drilling and the water level probably had not yet become stable.

The groundwater and creek water levels will fluctuate seasonally/yearly.

## 5 DISCUSSION AND RECOMMENDATIONS

### 5.1 GENERAL

A foundation investigation was carried out at the Hawk Creek Bridge for the design of a protection system for the proposed bridge rehabilitation and conversion to semi integral abutments. The bridge is located between approximately Highway 547 Stations 13+727 to 13+758, in the Township of Esquega, and is identified as Site No. 38C-16. The existing bridge is a two-span, concrete deck and concrete girder structure some 31 m in length along centerline.

The existing highway, at the bridge location, supports two undivided lanes of traffic, running in a south-north direction. Based on data from this foundation investigation, the embankment supporting the existing pavement structure at this site has been constructed with the fill (sand and gravel with cobble /boulder size rock pieces) overlying native sands.

Based on the Contract No. 63-39, the existing bridge is founded on deep foundations (324 mm diameter steel pipe piles driven to refusal at approximate elevation 288 m) at both of the north and the south abutments and the central pier (Enclosure No. 6, Appendix 5). The conversion to semi integral abutments will require the removal of the ballast walls. It is anticipated that, to carry out the bridge rehabilitation and convert the Hawk Creek Bridge to a semi integral abutment, an excavation some 1.2 m deep of will be required behind the existing abutments. As such, a protection system will be required at the north and south abutments of the bridge to support an excavation some 1.2 m deep behind the abutments and maintain an active lane of traffic. Based on data from this foundation investigation, the embankment fill behind the abutments supporting the approach slabs and pavement structure generally consists of sand and gravel fill to sand fill, mixed with rock pieces of the cobble and boulder sizes encountered at depths ranging from 0.8 m to 5.3 m below ground surface, at locations of Borehole Nos. 1 and 2.

### 5.2 EXCAVATION AND DEWATERING

The fill below the pavement structure and approach slabs is considered as a Type 3 soil in accordance with the Occupational Health and Safety Act and Regulations for Construction Projects. As such, to remain stable, above the water table, side walls of temporary open excavations would have to be cut back to an angle of 1H:1V. Some 1.2 m deep excavation (i.e. to elevation 315.3 m) will be required to the rear of the abutments to allow the rehabilitation work to be carried out on the ballast walls. The existing width of the approach is insufficient to allow the construction of a 1H:1V slope parallel to the active traffic lane. As such, a vertical excavation adjacent to the active traffic lane will be required and a protection system, installed perpendicular to the abutments, will be needed to support the active traffic lane. Conceptual shoring locations are illustrated on Figure No. SK-4 in Appendix 6.

Excavations must be maintained in a dewatered condition during excavation and foundation construction. The water level in the boreholes was recorded at elevations 311.7 m to 313.1 m, respectively. This level is well below the anticipated depth of excavation (elevation 315.3 m at both of abutments); therefore it is not anticipated that the groundwater table will be encountered during the shallow excavations at the abutments. If a deeper excavation is required to be advanced below the prevailing groundwater table (estimated at elevation 313 m), then groundwater control in accordance with OPSS 517 will have to be carried out.

### 5.3 PROTECTION SYSTEM

The results of this investigation indicated that, underlying the pavement structure (asphalt and concrete approach slabs), a sand fill, sand and gravel to sand trace gravel, some to trace silt, mixed with rock pieces of the cobble and boulder sizes, was encountered, in a loose to very dense state of compactness. The fill was underlain by native sands at elevations of 309.4 m and 307.8 m at the north and the south approaches, respectively.

The required depth of anticipated excavation, directly behind the abutments, will be relatively shallow, in the order of 1.2 m (elevation 315.3 m at both of abutments). In consideration of the anticipated soil conditions, the use of sheet piles of sufficiently robust cross section could be considered for a protection system due to their lower construction cost and limited depth of excavation. If a cobble/boulder size rock is encountered during driving of a sheet section, the individual sheet section could be left high and the cobble/ boulder can be removed as the excavation is advanced, followed by continued driving of the sheet. The soldier piles and lagging wall can be considered as an alternative method with the higher cost. Pre-drilling the borehole may likely be required to advance the H piles through the rock obstacles, if encountered. In order to fix the sheet toe, the sheeting should be driven to a depth of a minimum of 1.0 m below the required depth of excavation. At this depth, the toe of sheeting would be established in the sand fill deposit, mixed with rock pieces of the cobble and boulder sizes. Considering the limited depth of excavation and provided a sheet pile of sufficiently robust section is used, a whaler and raker may not be required if the top of the sheet pile wall is fixed to the existing approach slab. If fixing the sheet pile walls to the approach slab is not feasible, a whaler with raker or a tieback system would have to be installed. If tiebacks are required, the resistance (R) for grouted anchors, located outside the active failure wedge, in cohesionless soils can be estimated from the following equation as supplied in the Canadian Foundation Manual (4<sup>th</sup> Edition):

$$R = \sigma_z' A_s L_s \alpha_g$$

Where:  $\sigma_z'$  = effective vertical stress at the midpoint of the load carrying bond length

$A_s$  = effective unit surface area of the anchor

$L_s$  = effective embedment length of the anchor

$\alpha_g$  = anchorage coefficient use 1.0 for granular backfill

For tieback design, a triangular earth pressure distribution over the height of the cut would be appropriate for design.

Unless the pull-out resistance (capacity) of the anchor is proven with a load test program, the allowable anchor load (as suggested by the Canadian Foundation Engineering Manual, 4th Edition), is commonly obtained by dividing the computed capacity of the anchor by a factor of safety of 3. Alternatively, proprietary anchor systems can be used.

If excavation to a greater depth than the anticipated 1.2 m is required, then the shoring system would have to be advanced to a greater depth, dependant on the final depth of excavation below the top of pavement. Based on the results of this investigation, the embankment fill mixed with rock pieces of the cobble and boulder sizes appears to contain obstructions. Although obstructions were encountered at the approach boreholes at various depths, sheet piles are considered acceptable for use in shallow to deeper excavations in the embankment fills, provided that sheet piles have sufficiently robust cross section.

Considering the cohesionless nature of the embankment fills (pavement structure over sand fills) a rectangular apparent pressure distribution over the height of the excavation would be appropriate for design of the temporary shoring. The width of the apparent rectangular pressure distribution, over the height of excavation, can be considered equal to  $0.65 \cdot K_a \cdot \gamma \cdot H$ , where:

$K_a$  = active earth pressure,

$\gamma$  = unit weight, and

$H$  = height of the wall above the base of excavation.

Surcharge loads from the active lane of traffic must also be considered for designing the temporary shoring system.

A table outlining the possible temporary excavation protection/flexible retaining systems and their relative advantages, disadvantages, and costs, as well as comments on the viability of the methods is provided on Table A in Appendix 6.

The protection system can be designed using the lateral earth pressure parameters provided in section 5.4 Lateral Earth Pressures below.

The temporary protection system should be designed and constructed to comply with OPSS 539. In consideration of the location of the protection system and traffic volume, a Performance Level 2 is considered appropriate.

## 5.4 LATERAL EARTH PRESSURES

Lateral earth pressures should be computed in accordance with the Canadian Highway Bridge Design Code (CHBDC). The design parameters for the sand fill and native materials are as follows:

PARAMETER	SAND FILL	SAND
Unit Weight (kN/m <sup>3</sup> )	21.0	19.0
Angle of Internal Friction	32°	29°
Coefficient of Active Earth Pressure ( $K_a$ )	0.31	0.35
Coefficient of Passive Earth Pressure ( $K_p$ )	3.22	2.86
Coefficient of Earth Pressure at Rest ( $K_o$ )	0.47	0.52

For rigid structures, the “at-rest” condition ( $K_o$ ) applies. For flexible structures the “active” condition ( $K_a$ ) applies.

## 5.5 BACKFILL AND COMPACTION

The existing backfill at the abutments was generally in a loose to very dense condition. Following completion of the rehabilitation and prior to backfilling the excavation, the existing subgrade should be proof rolled with a minimum of five overlapping passes of a hand operated vibratory compactor with a minimum weight of 400 kg (or a centrifugal force of 50 kN). Backfilling should be carried out in accordance with OPSS 902 and compaction should be carried out in accordance with OPSS 501.

## 5.6 CONSTRUCTION CONCERNS

Considering the relatively shallow depth of expected excavations and nature of the approach fill and native materials, no major construction concerns are anticipated if the works are carried out in general conformance to that discussed herein; however the rock pieces of cobble and boulder sizes encountered at varying depths in the embankment fills must be recognized by the contractor. The contractor should be prepared to deal with these materials for the temporary protection system, dewatering and other construction activities.

## 6 STATEMENT OF LIMITATIONS

The design recommendations given in this geotechnical report are applicable only to the project described in the text and only if constructed substantially in accordance with details of alignment and elevations stated in the report. Since all details of the design may not be known, in our analysis certain assumptions had to be made. The actual conditions may however, vary from those assumed, in which case changes and modifications may be required to our geotechnical recommendations. We recommend, therefore, that we be retained and provided the opportunity during the design stage to review the design drawings, site survey information, proposed elevations, etc. to verify that they are consistent with our recommendations or the assumptions made in our analysis. It is further recommended that we be retained to review the final design drawings and specifications relative to the geotechnical recommendations.

If, during construction, conditions in the field vary from those assumed at the design stage, an engineer from this office must be notified immediately.

Proper subgrade preparation, groundwater control, compaction, etc. are all critical aspects of the bearing capacity of native soils. It must be noted that different aspects of the geotechnical design are based on the assumption that Englobe will be retained during site preparation and construction of the proposed works to ensure that both the geotechnical site characteristics and the construction operations/techniques are consistent with our recommendations. Should Englobe not be involved during the full construction phase, our liability is strictly limited to the factual information contained herein only.

The comments in this report are intended solely for the guidance of the design engineer and address the geotechnical conditions only, at this preliminary design stage. The number of boreholes required to determine the localized conditions between boreholes directly affecting construction costs, equipment, scheduling, etc. would in fact be greater than what has been carried out for design purposes. Therefore, contractors bidding on this project or undertaking this work should make their own interpretations of the factual borehole results and carry out further work as they deem necessary to assess the scope of the project.

Section 5 of this reported is intended for the use of the client and the design team only and is not intended to be included in the tender documents. Inclusion of the factual information (Sections 1 to 4 inclusive) in the tender documents is furnished merely for the general information of bidders and is not in any way warranted or guaranteed by or on behalf of the owner or the owner's consultants and its subconsultants or the consultants' or subconsultants' employees, and neither the owner nor its consultants or its employees shall be liable for any representations negligent or otherwise contained in the documents.

## Appendix 1 Drawings

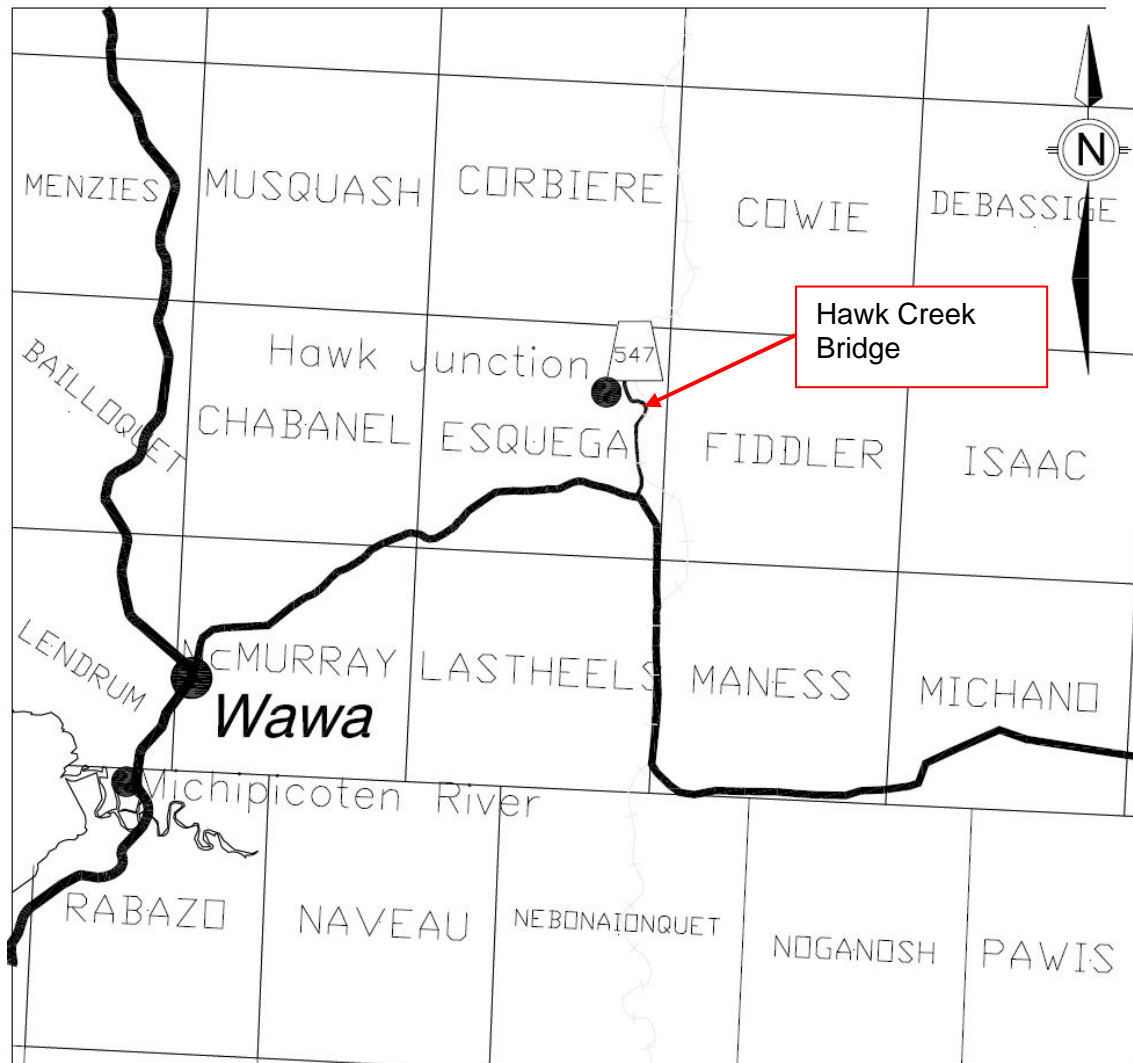
Drawing No. 1

Key Plan

# MACRO KEY PLAN

Drawing No.1

NOT TO SCALE



## FINAL FOUNDATION INVESTIGATION AND DESIGN REPORT

**GWP 5377-11-00**

Highway 547

Hawk Creek Bridge

Site No. 38C-016

Township of Esquega



Reference No: 15/04/15026-F1

August 2015



## Appendix 2   Subsurface Data

Enclosure No. 1	List of Abbreviations and Symbols
Enclosure Nos. 2 and 3	Record of Borehole Sheets

## LIST OF ABBREVIATIONS & DESCRIPTION OF TERMS

The abbreviations and terms, used to describe retrieved samples and commonly employed on the borehole logs, on the figures and in the report are as follows:

### 1. ABBREVIATIONS

AS	Auger Sample
CS	Chunk Sample
DS	Denison type sample
FS	Foil Sample
NFP	No Further Progress
PH	Sampler advanced by hydraulic pressure
PM	Sampler advanced by manual pressure
RC	Rock core with size & percentage of recovery
SS	Split Spoon
ST	Slotted Tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash Sample
WH	Sampler advanced by static weight of hammer and/or rods
Rec	% recovery from individual run of rock core
RQD	Rock quality designation (%)

### 2. PENETRATION RESISTANCE/"N"

*Dynamic Cone Penetration Test (DCPT):*

A continuous profile showing the number of blows for each 300 mm of penetration of a 50 mm diameter 60° cone attached to AW rod driven by a 63 kg hammer falling 760 mm.

Plotted as —●—●—●—●—

*Standard Penetration Test (SPT) or "N" Values*

The number of blows of a 63 kg hammer falling 760 mm required to advance a 50 mm O.D. drive open sampler 300 mm.

### 3. SOIL DESCRIPTION

a) *Cohesionless Soils:*

"N" (blows/0.3 m)	Relative Density
0 to 4	very loose
4 to 10	loose
10 to 30	compact
30 to 50	dense
over 50	very dense

b) *Cohesive Soils:*

Undrained Shear Strength (kPa)	Consistency
Less than 12	very soft
12 to 25	soft
25 to 50	firm
50 to 100	stiff
100 to 200	very stiff
over 200	hard

### 3. SOIL DESCRIPTION (Cont'd)

c) *Cohesive Soils:*

RQD (%)	Classification
Less than 25	Very poor quality
25 to 50	Poor quality
50 to 75	Fair quality
75 to 90	Good quality
90 to 100	Excellent quality

d) *Method of Determination of Undrained Shear Strength of Cohesive Soils:*

- + 3.2 - Field Vane test in borehole.  
The number denotes the sensitivity to remoulding.
- D - Laboratory Vane Test
- " - Compression test in laboratory

For a saturated cohesive soil the undrained shear strength is taken as one-half of the undrained compressive strength.

e) *Soil Moisture:*

Moisture	Described as
Dry	Below optimum moisture content
Moist	Near optimum moisture content
Wet	Above optimum moisture content

### 4. TERMINOLOGY

Terminology used for describing soil strata is based on the proportion of individual particle sizes present in the samples (please note that, with the exception of those samples subject to a grain-size analysis, all samples were classified visually and the accuracy of visual examination is not sufficient to determine exact grain sizing):

Trace, or occasional	Less than 10%
Some	10 to 20%
With	20 to 30%
Adjective (i.e. silty or sandy)	30 to 40%
And (i.e. sand and gravel)	40 to 60%

Terminology for cobbles and boulders is based on auger response and field observations:

Occasional	Obstructions encountered in borehole, however advance is not impeded
Numerous	Obstructions are essentially continuous over drilled length

**SAMPLE DESCRIPTION NOTES:**

1. **FILL:** The term fill is used to designate all man-made deposits of natural soil and/or waste materials. The reader is cautioned that fill materials can be very heterogeneous in nature and variable in depth, density and degree of compaction. Fill materials can be expected to contain organics, waste materials, construction materials, shot rock, rip-rap, and/or larger obstructions such as boulders, concrete foundations, slabs, abandoned tanks, etc.; none of which may have been encountered in the borehole. The description of the material penetrated in the borehole therefore may not be applicable as a general description of the fill material on the site as boreholes cannot accurately define the nature of fill material. During the boring and sampling process, retrieved samples may have certain characteristics that identify them as 'fill'. Fill materials (or possible fill materials) will be designated on the Borehole Logs. If fill material is identified on the site, it is highly recommended that testpits be put down to delineate the nature of the fill material. However, even through the use of testpits defining the true nature and composition of the fill material cannot be guaranteed. Fill deposits often contain pockets or seams of organics, organically contaminated soils or other deleterious material that can cause settlement or result in the production of methane gas. It should be noted that the origins and history of fill material is frequently very vague or non-existent. Often fill material may be contaminated beyond environmental guidelines and the material will have to be disposed of at a designated site (i.e. registered landfill). Unless requested or stated otherwise in this report, fill material on this site has not been tested for contaminants however, environmental testing of the fill material can be carried out at your request. Detection of underground storage tanks cannot be determined with conventional geotechnical procedures.
2. **TILL:** The term till indicates a material that is an unstratified, glacial deposit, heterogeneous in nature and, as such, may consist of mixtures and pockets of clay, silt, sand, gravel, cobbles and/or boulders. These heterogeneous deposits originate from a geological process associated with glaciation. It must be noted that due to the highly heterogeneous nature of till deposits, the description of the deposit on the borehole log may only be applicable to a very limited area and therefore, caution must be exercised when dealing with a till deposit. When excavating in till, contractors may encounter cobbles/boulders or possibly bedrock even if they are not indicated on the borehole logs. It must be appreciated that conventional geotechnical sampling equipment does not identify the nature or size of any obstruction.
3. **BEDROCK:** Auger refusal may be due to the presence of bedrock, but possibly could also be due to the presence of very dense underlying deposits, boulders or other large obstructions. Auger refusal is defined as the point at which an auger can no longer be practically advanced. It must be appreciated that conventional geotechnical sampling equipment does not differentiate between nature and size of obstructions that prevent further penetration of the boring below grade. Bedrock indicated on the borehole logs will be labeled 'possibly' or 'probable' etc. based on the response of the boring and sampling equipment, surrounding topography, etc. Bedrock can be proven at individual borehole locations, at your request, by diamond core drilling operations or, possibly, by testpits. It must also be appreciated that bedrock surfaces can be, and most times are, very erratic in nature (i.e. sheer drops, isolated rock knobs, etc.) and caution must be used when interpreting subsurface conditions between boreholes. A bedrock profile can be more accurately estimated, at the clients' request, through a series of closely positioned unsampled auger probes combined with core drilling.
4. **GROUNDWATER:** Although the groundwater table may have been encountered during this investigation and the elevation noted in the report and/or on the record of boreholes, it must be appreciated that the elevation of the groundwater table will fluctuate based upon seasonal conditions, localized changes, erratic changes in the underlying soil profile between boreholes, underlying soil layers with highly variable permeabilities, etc. These conditions may affect the design and type and nature of dewatering procedures. Cave-in levels recorded in borings give a general indication of the groundwater level in cohesionless soils however, it must be noted that cave-in levels may also be due to the relative density of the deposit, drilling operations etc.

REFERENCE	15/04/15026	DATUM	Geodetic	LOCATION	N 5326653.4 E 264111.1 - Esquega Twp., Station 13+725.5	ORIGINATED BY	JL
PROJECT	GWP 5377-11-00, Highway 547 - F1	BOREHOLE TYPE	Truck Mounted CME 45 - Hollow Stem Augers		COMPILED BY	SH	
CLIENT	AECOM	DATE (Started)	11 June 2015	TIME (Completed)	1:10:00 PM	CHECKED BY	MAM
		DATE (Completed)	11 June 2015				

MEL-GEO 15029 - BOREHOL LOGS - F1.GPJ MEL-GEO.GDT 10/8/15

**METRIC****RECORD OF BOREHOLE NO. 2**

REFERENCE 15/04/15026 DATUM Geodetic LOCATION N 5326685.7 E 264098.5 - Esquega Twp., Station 13+760 ORIGINATED BY JL

PROJECT GWP 5377-11-00, Highway 547 - F1 BOREHOLE TYPE Truck Mounted CME 45 - Hollow Stem Augers COMPILED BY SH

CLIENT AECOM DATE (Started) 12 June 2015 TIME (Completed) 12:30:00 PM CHECKED BY MAM

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	STRATA PLOT	NUMBER	TYPE	"N" VALUES			20	40					
316.4	Ground Surface													
0.0	127 mm Asphalt													
316.0	305 mm Concrete													
0.4	FILL- sand with gravel trace silt, to sand trace gravel trace silt, rock pieces of the sizes of cobble and boulder encountered at depths from 1.7 m to 5.3 m below ground surface		1	SS	24									22 72 (6)
	brown, moist		2	SS	21									
	(compact/very dense)		3	SS	52/0.25 m									
			4	SS	32									31 50 (19)
	loose		5	SS	7									
			6	SS	53									
			7	SS	65/0.23 m									
			8	SS	16									27 64 (9)
	sand, trace gravel, trace silt greyish brown to grey		9	SS	5									
	loose		10	SS	6									
307.8	SAND - trace silt, trace clay trace grass rootlets													
8.6	grey, wet		11	SS	2									0 93 6 1
306.8	(very loose)													
9.6	End of Sampling End of Borehole													
COMMENTS							+ 3, × 3 : Numbers on right refer to Sensitivity Numbers on left refer to values greater than 120 kPa ○ 3% STRAIN AT FAILURE		WATER LEVEL RECORDS Date (dd/mm/yy)/Time    Water Depth (m)    Cave In (m) 1) 12/6/15 12:30:00 PM    3.3    3.4 2)    -    - 3)    -    -					

The stratification lines represent approximate boundaries. The transition may be gradual.

**EnGlobe Corp.**

120 Progress Court, North Bay, On P1A 0C2 Phone: (705)476-2550 Fax: (705)476-8882 Email: northbay@vm.ca

MEL-GEO 15029 - BOREHOL LOGS - F1.GPJ MEL-GEO.GDT 10/8/15

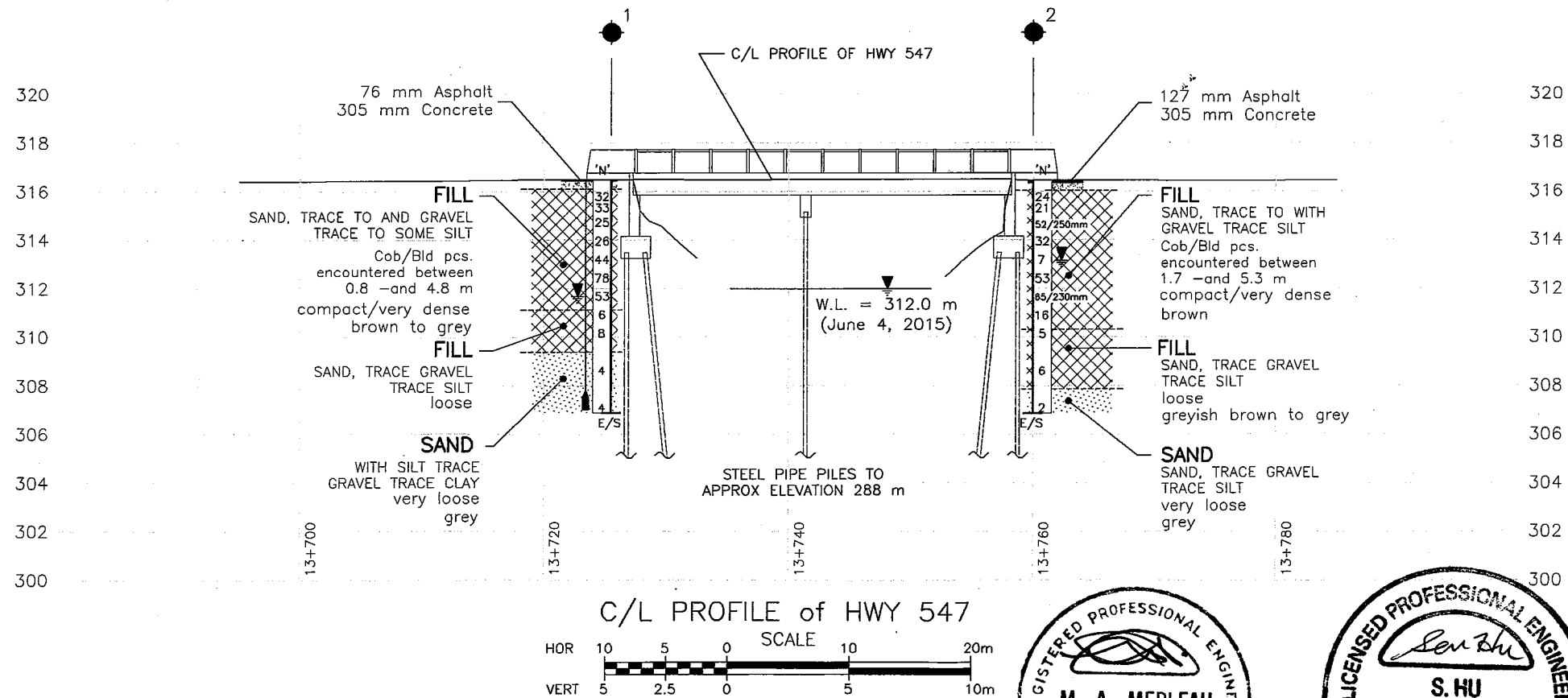
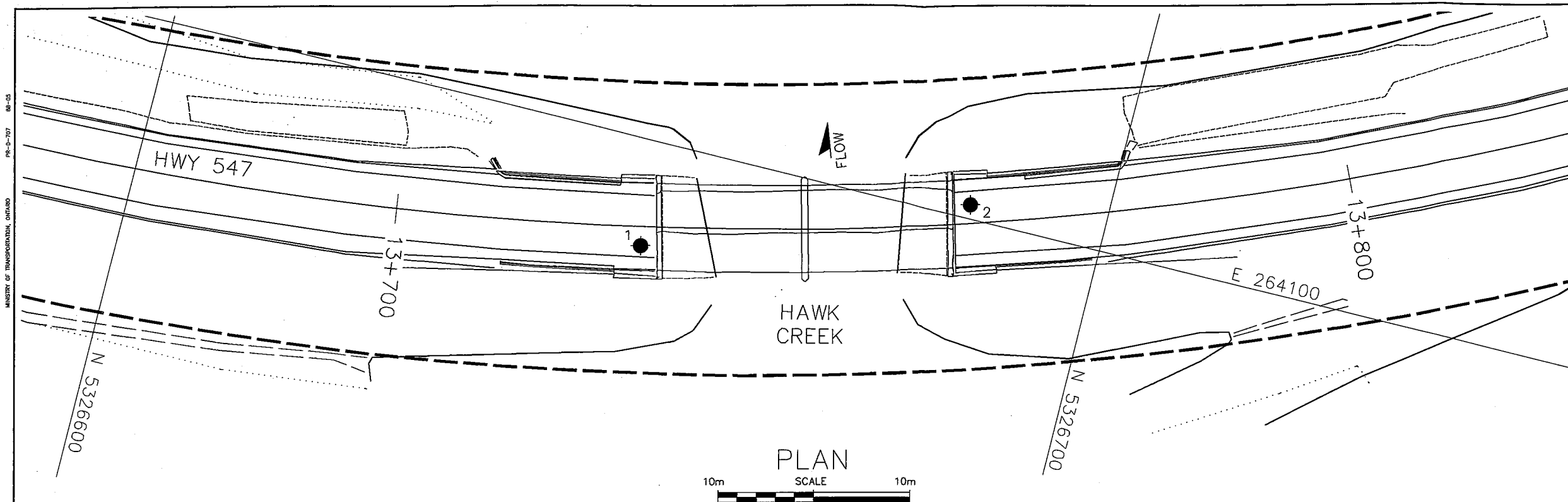
## Appendix 3 Laboratory Data

Drawing No. 2: Borehole Location and Soil Strata

Figure Nos. L-1 and L-2: Grain Size Distribution Curves

Table No. L-3: Laboratory Test Summary Sheet

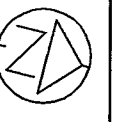
CAD FILE LOCATION AND NAME: G:\2015\15026 - EDN, Highway 547 GWP 5377-11-00 (AECOM)\Drawings\Hawk Creek Bridge\Working - Do Not Move or Delete Files\15026-F1 - Borehole Location Plan, Hawk Creek Bridge.dwg  
MODIFIED: 8/10/2015 8:01:30 AM BY: GRASRY  
DATE PLOTTED: 8/12/2015 11:08:24 AM BY: RYAN GRASSER



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

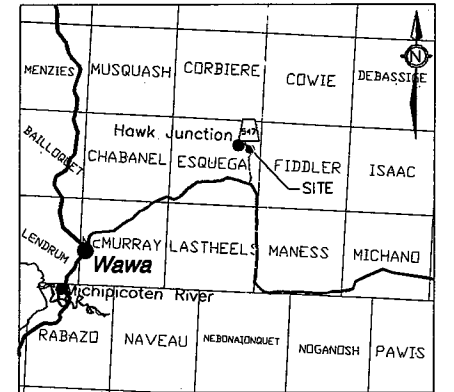


DISTRICT  
CONT. No.  
GWP No. 5377-11-00



HWY 547  
HAWK CREEK BRIDGE  
SITE NO. 38C-016  
BOREHOLE LOCATIONS  
AND SOIL STRATIGRAPHY

SHEET  
2



KEY PLAN  
N.T.S.

LEGEND

- Borehole w/ DCPT
- Borehole
- N
- Blows/0.3 m (Std Pen Test, 475 J/blow)
- DCPT
- Blows/0.3 m (60° Cone, 475 J/blow)
- Water Level at Time of Investigation
- A/R
- Auger Refusal at Elevation
- E/S
- End of Sampling
- Piezometer

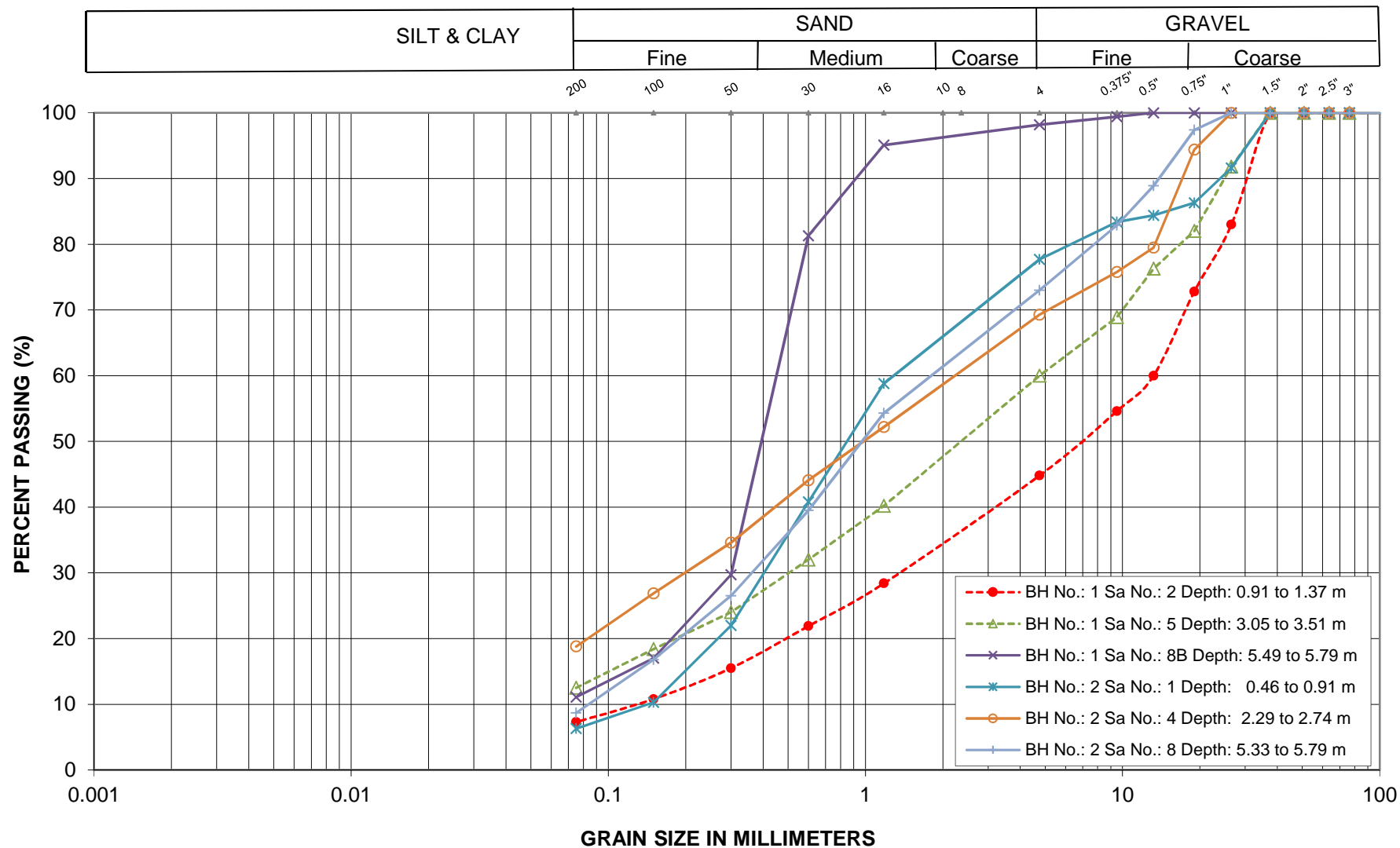
BOREHOLE No.	ELEVATION	O/S	NORTHING	EASTING
1	316.5	2.0m Rt	5326653.4	264111.1
2	316.4	2.0m Lt	5326685.7	264098.5

**NOTES:**  
The boundaries between soil strata have been established at the borehole locations only. The boundaries illustrated and stratigraphy between boreholes on this drawing are assumed based on borehole data and may vary. They are intended for design only.

Base plan and alignment provided in digital format by Callon Dietz on June 16, 2015

GEOCRES No. 42C-35

REVISIONS	DESCRIPTION
DESIGN	CHK
DRAWN	RG
CODE	LOAD
DATE	AUG/15
SITE	38C-016
STRUCT	SCHEME
TDWG	2

**GRAIN SIZE ANALYSIS**

LOCATION: Hwy 547, Hawk Creek Bridge

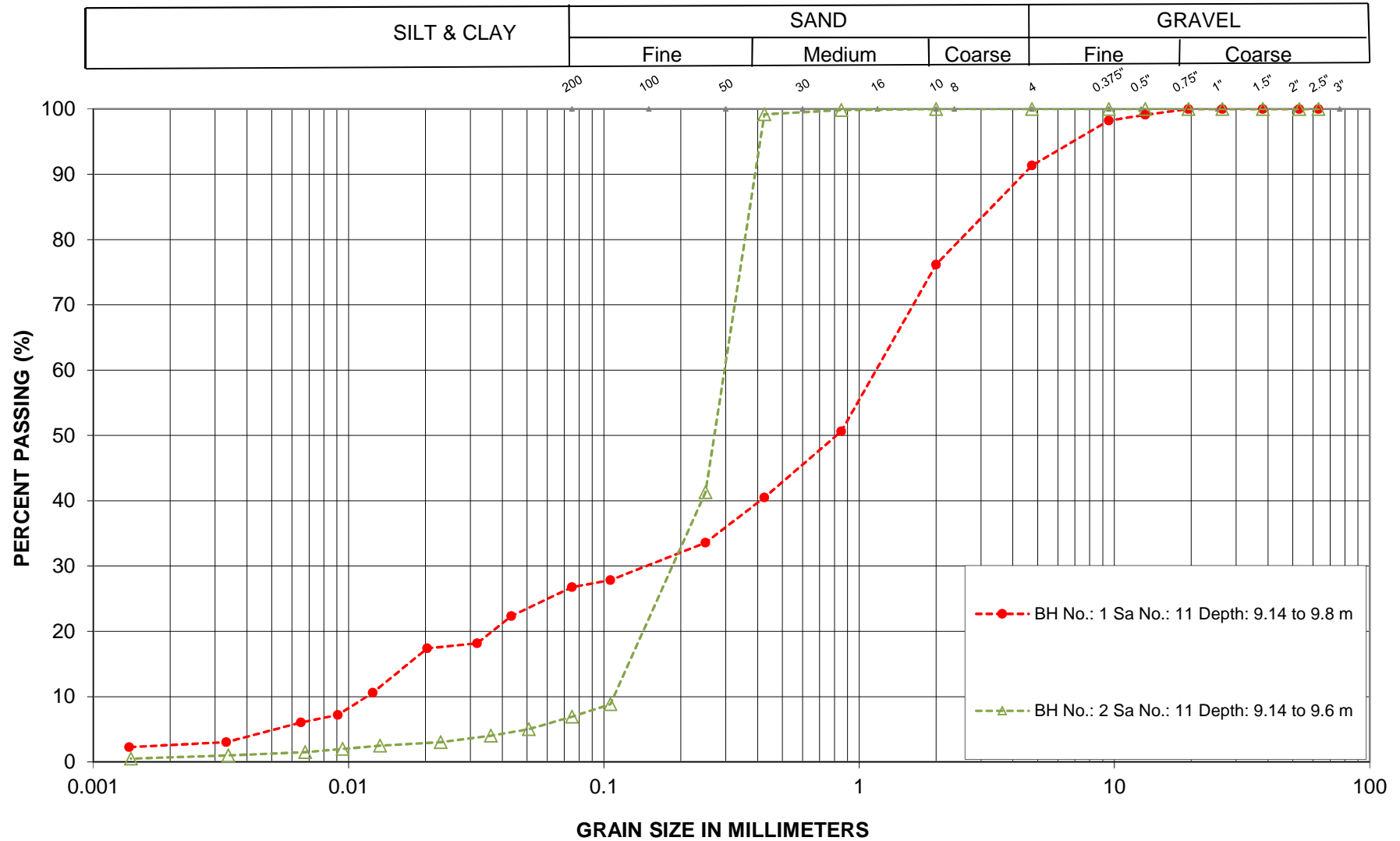
Sand Fill

Englobe Corp.

**FIGURE L-1**  
**GWP 5377-11-00**



## GRAIN SIZE ANALYSIS



LOCATION: Hwy 547, Hawk Creek Bridge

SAND to SAND with SILT

Englobe Corp.

FIGURE L-2  
GWP 5377-11-00

## Laboratory Tests - Summary Sheet



Borehole No.	Sample No.	Depth	Grain Size Analysis				NMC	Atterberg Limits			SPT 'N'	USCS	Unit Weight (kN/m3)	Remarks
			Gravel Size (%)	Sand Size (%)	Silt Size (%)	Clay Size (%)		LL (%)	PL (%)	IP (%)				
1	1	0.7					4.1				32			
	2	1.1	45	48	7		8.1				33			
	3	1.8					22.8				25			
	4	2.5					19.6				26			
	5	3.3	40	47	13		9.5				44			
	6	4.0					20.7				78			
	7	4.8					19.9				53			
	8A	5.4					26.2				6			
	8B	5.6	2	87	11		34.4							
	9	6.3					26.9				8			
	10	7.9					56.5				4			
	11	9.5	9	64	24	3	20.0				4			
2	1	0.7	22	72	6		3.9				24			
	2	1.1					4.1				21			
	3	1.7					5.3				52/0.25 m			
	4	2.5	31	50	19		6.0				32			
	5	3.3					7.9				7			
	6	4					5.8				53			
	7	4.7					10.6				65/0.23 m			
	8	5.6	27	64	9		15.5				16			
	9	6.3					19.3				5			
	10	7.85					20.2				6			
	11	9.4	0	93	6	1	26.4				2			

## **Appendix 4**

## **Photo Essay**

Enclosure No. 4:

Photo Essay

Bridge South side– Looking North

Photo: 1



Bridge North Approach Embankment – Looking Northwest

Photo: 2



Project: Hwy 547 – Hawk Creek Bridge, Township of Esquega

Photos Provided By:Englobe

Date: June 2015

Upstream of Creek– Looking Northeast

Photo: 3



Downstream of Creek to Hawk Lake – Looking Northwest

Photo: 4



Project: Hwy 547 – Hawk Creek Bridge, Township of Esquega

Photos Provided By:Englobe

Date: June 2015

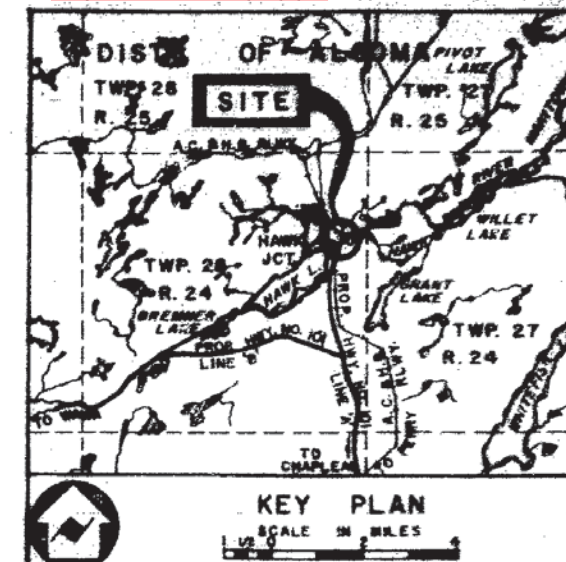
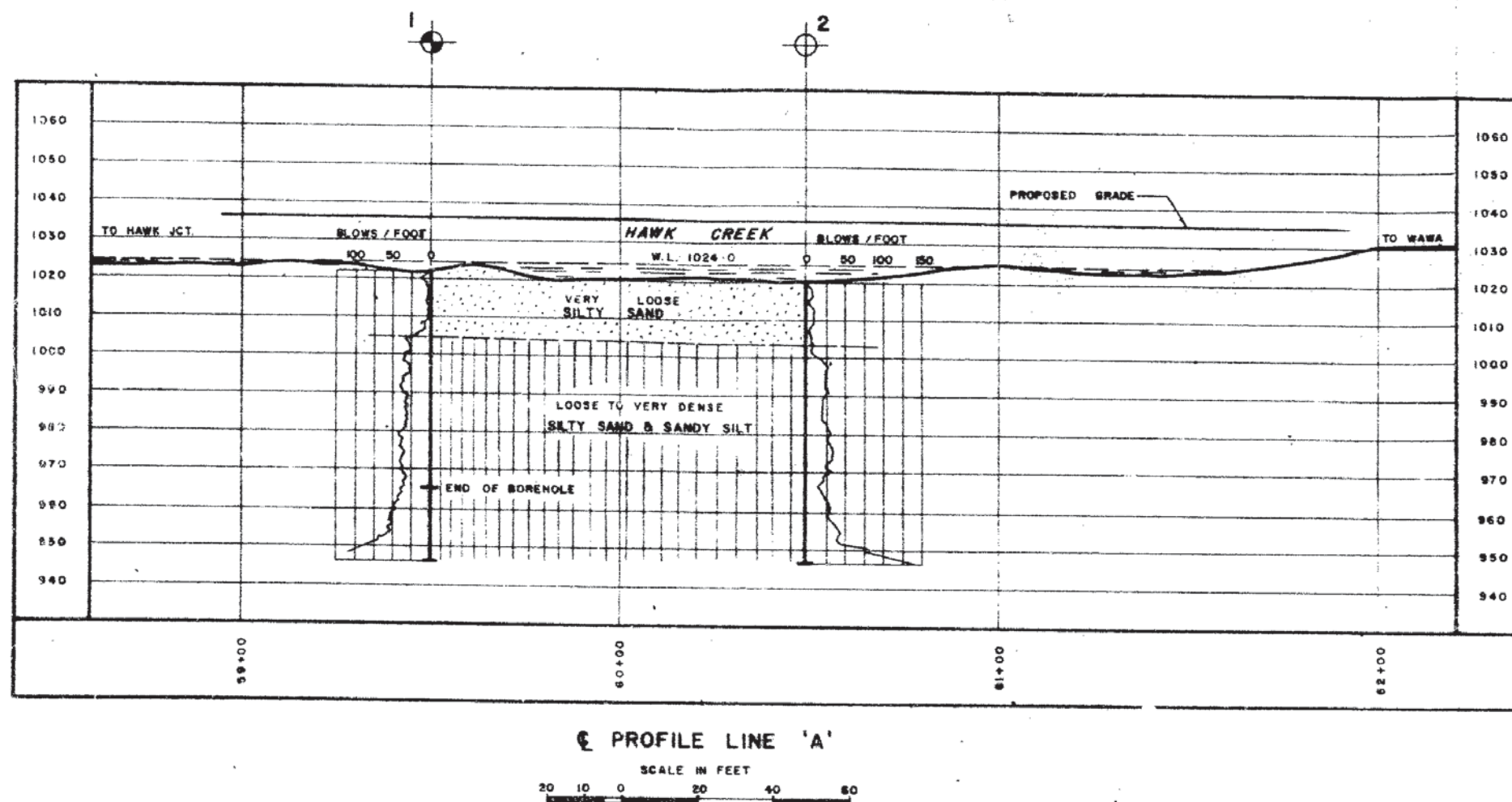
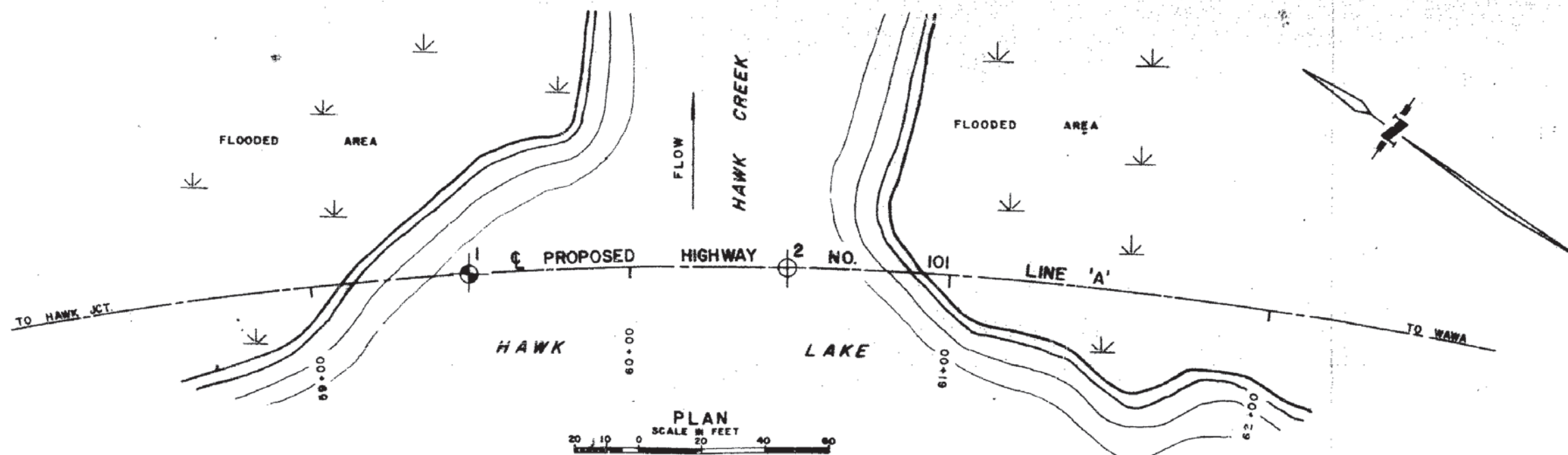
## **Appendix 5**

## **Historical Data**

Enclosure Nos. 5 and 6:

Historical Drawings



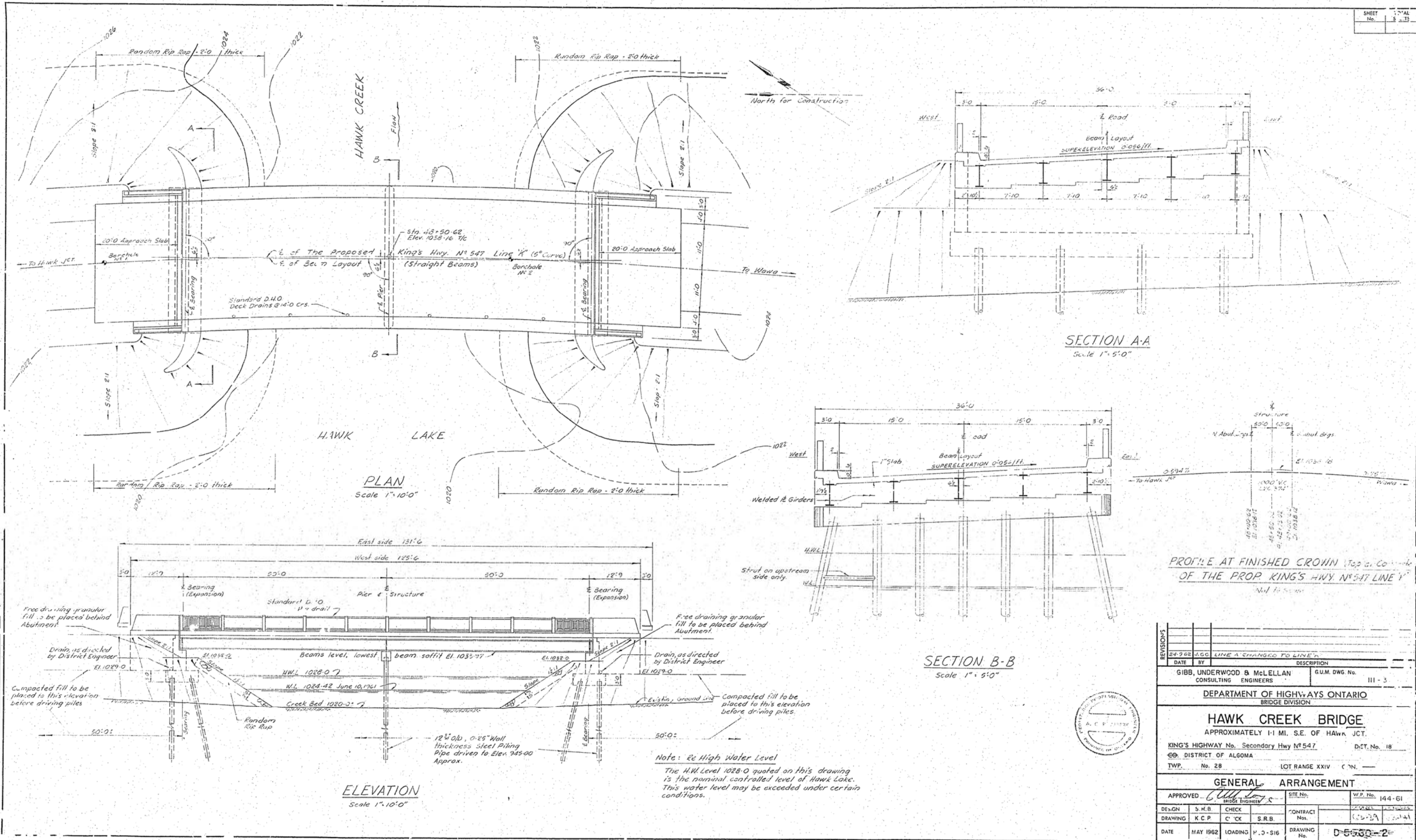


LEGEND			
	BORE & PENETRATION HOLE		
	PENETRATION HOLE (CONE)		
	BORE HOLE		
	WATER LEVELS - Established at Time of Field Investigation, DEC. 8, 1961		
NO.	ELEVATION	STATION	OFFSET
1	1022.0	59+50	E
2	1020.0	60+50	E

NOTE -  
The boundaries between soil strata have been established only at bore hole locations. Between bore holes the boundaries are assumed from geological evidence and may be subject to considerable error.

DEPARTMENT OF HIGHWAYS - ONTARIO			
MATERIALS & RESEARCH DIVISION - FOUNDATION SECTION			
<b>HAWK CREEK AND PROPOSED HIGHWAY NO. 101 LINE 'A'</b>			
ORIGINATED W. KULBATICAS	DISTRICT NO. 18	DATE FEB. 8, 1962	
DRAWN F. CLARK	W.P. NO. 144-81	JOB NO. 61-F-120	
CHECKED <i>[Signature]</i>	CONTRACT NO.	REVISION: 00	
APPROVED <i>[Signature]</i>		61-F-120A	





PROFILE AT FINISHED CROWN (Top of Center) OF THE PROPOSED KING'S HWY. NO. 547 LINE 1

REVISIONS					
24.9.62	J.G.C.	LINE A CHANGED TO LINE 1			
DATE	BY	DESCRIPTION			
GIBB, UNDERWOOD & McLELLAN		G.U.M. DWG. No.		III - 3	
CONSULTING ENGINEERS					
DEPARTMENT OF HIGHWAYS ONTARIO					
BRIDGE DIVISION					
HAWK CREEK BRIDGE					
APPROXIMATELY 1.1 MI. S.E. OF HAWK JCT.					
KING'S HIGHWAY No. Secondary Hwy No 547				D.C.T. No. 18	
CO. DISTRICT OF ALGOMA					
TWP. No. 28		LOT RANGE XXIV C.N. —			
GENERAL ARRANGEMENT					
APPROVED		SITE No.		W.P. No. 144-61	
DESIGN		CHECK		CONTRACT	
DRAWING		S.R.B.		Nos.	
DATE		LOADING		DRAWING	
MAY 1962		J.C. - S16		No. D-550-2	

Twp # 682-1-2-19

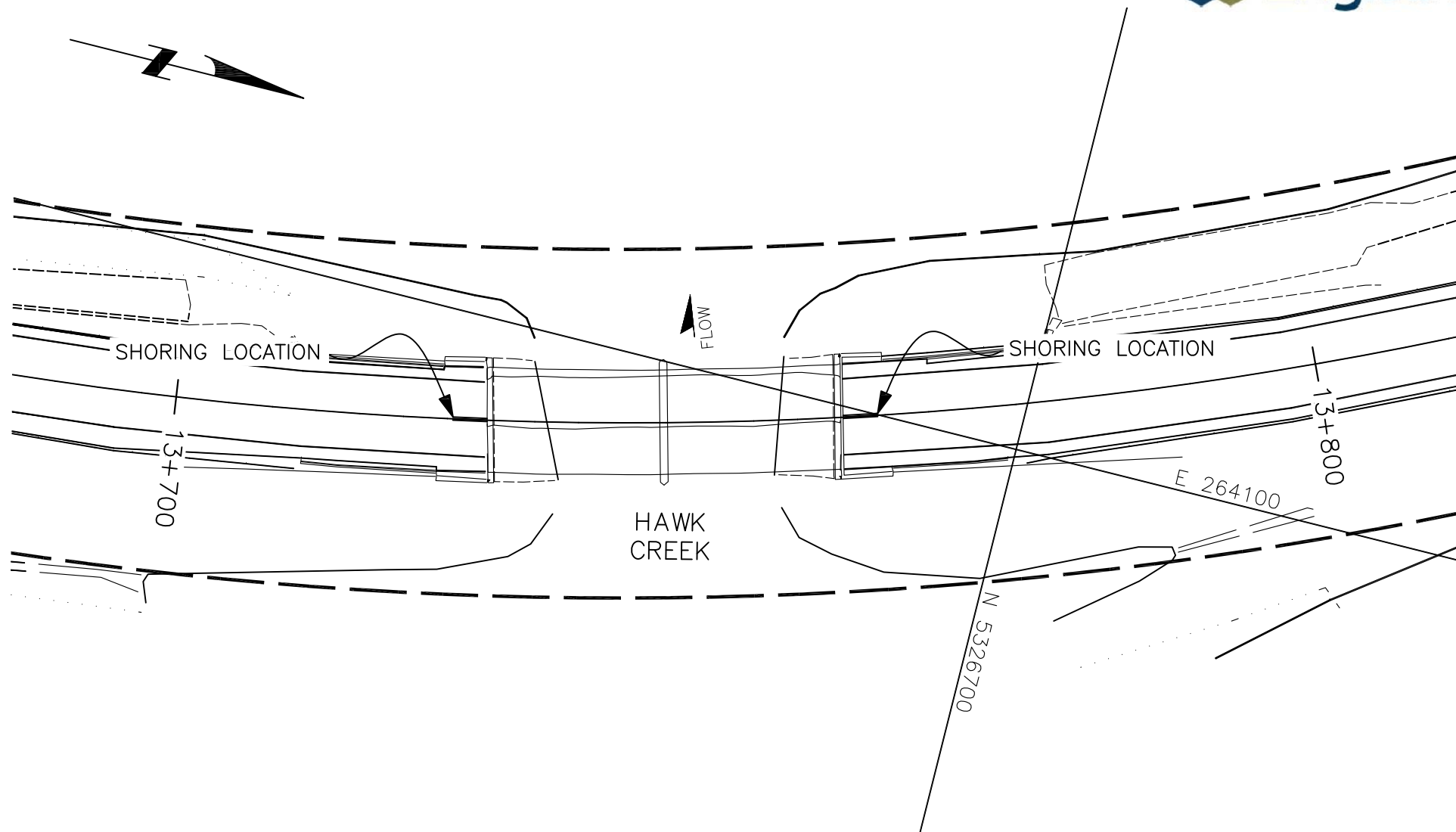


## Appendix 6      Design Data

Table A:	Comparison of Shoring Alternatives
Figure No. SK-4:	Conceptual Shoring Location Plan
	Notice to Contractor - Obstructions

**Table A – Comparison of Shoring Alternatives**

Method	Depth Range (m)	Advantages	Disadvantages	Remarks	Estimated Costs
Wood Sheeting	1.5 – 5	-Low cost, -Easily installed in good ground conditions	-Limited by soil conditions, -Limited depth of installation, -Low strength, -discontinuous	Not recommended due to existence of rockfill mixed in embankment fill	\$ 650/m <sup>2</sup>
Steel Sheet Piles	5 – 21	-High strength, continuous, -Readily available	-Limited by soil conditions (i.e. obstructions)	Recommended, provided with sufficient robust cross section	\$ 650/m <sup>2</sup>
Pre-cast concrete panels	3 – 10	-Durable -Assists in minimizing seepage	-Limited depths -Can be damaged during installation -Limited by soil conditions (i.e. obstructions)	Not considered due to higher cost	
Soldier piles and lagging	5 – 25	-Easy installation -Readily available -Adaptable to various ground conditions	-Pre-drilling may be required -Possible ground loss	Considered as an alternative due to higher costs than steel sheet piles	\$ 725/m <sup>2</sup>
Tangent/ Secant/ Staggered Drilled Piles	10 – 18	-Readily available -Adaptable to various ground conditions	-Possible ground loss and/or seepage -Poor alignment tolerance	Not Considered due to higher costs	
Concrete Diaphragm	10 – 30	-High Strength -Durable -Can be permanent	-High cost -Requires specialized equipment/control	Not Considered due to higher costs	
Micropiles with reinforced shotcrete face		-Can be installed in various ground conditions -High strength -Good tolerance	-High Cost -Requires specialized equipment	Not Considered due to higher costs	



## METRIC

Dimensions are in meters  
and/or millimeters unless  
otherwise shown. Stations are  
in kilometers + meters.



Highway 547, Township of Esquega - Hawk Creek Bridge  
Conceptual Shoring Location Plan

FIGURE SK-4

15/04/15026-F1

Hawk Creek Bridge, Highway 547, Esquega Twp.

**NOTICE TO CONTRACTOR – Obstructions in Fill and Native Soils**

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**Special Provision**

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The Contractor is advised that, at the borehole locations, embankment fills include gravels, sands and silts, and contain a mix of cobble/boulder sizes at varying depths. The contractor should be prepared to deal with these materials for temporary protection system, dewatering and other construction activities.



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