



MEMORANDUM

FOUNDATION RECOMMENDATIONS

Rosseau River Bridge East Foundation at the Rosseau River on Hwy 141

Agreement No. 5015-E-0007

Assignment No. 3

GWP No. 5394-15-00

Geocres No. 31E-361

Prepared for:

Ontario Ministry of Transportation

Regional Director's Office -NE Region

447 McKeown Avenue, Suite 301

North Bay, ON P1B 9S9

Attn: J. P. Perron

Ontario Ministry of Transportation

Pavements and Foundations Section

Materials Engineering and Research Office

Room 223, 2/F

145 Sir Williams Hearst Avenue

Toronto, ON M3M 0B6

Attn: K. Ahmad

exp Services Inc.

October 14, 2016



MEMORANDUM

Date: October 14, 2016

To: Michael Choy, P.Eng.
Foundation Engineer
Ministry of Transportation
Pavements and Foundations Section
145 Sir William Hearst Ave,
Downsview, Ontario M3M 0B6

Cc: Jean-Pierre Paron, P.Eng.
Project Manger
Ministry of Transportation
and
Ken Ahmed, P.Eng.
Senior Foundation Engineer
Ministry of Transportation

From: Nimesh Tamrakar, M.Eng.
Technical Specialist
exp Services Inc.

Cc: Stan Gonsalves, M.Eng., P.Eng
Designated MTO Contact
exp Services Inc.

Subject: Foundation Recommendations
Rosseau River Bridge East Foundation at the Rosseau River on Hwy 141
Rosseau, Ontario
Agreement No. 5015-E-0007, Assignment No. 3

1.0 Introduction

This memorandum provides geotechnical recommendations on east footing for Rosseau River Bridge replacement at the Rosseau River on Hwy 141, Rosseau, Ontario. The recommendations are based on interpretation of the factual data obtained from the additional boreholes advanced during the current investigation at the site and previous data presented in Foundation Investigation and Design report for Rosseau River Bridge Replacement (Geocres No. 31E-361). Initially spread footing option was selected for east foundation by the designer and during construction it was revealed that bedrock is sloping with



a steep slopes toward river. After additional soil and bedrock investigation and further analysis of possible spread footing options and their consequences, the designer opted to select the deep foundation option with steel H-Pile socketed into the bedrock (see new GA drawing attached in Appendix A). This memorandum generally discussed about the initially selected option of spread footing for east foundation. Since, deep foundation option with steel H-Pile socketed into the bedrock was selected and the recommendation for deep foundation options were already discussed in Foundation Investigation and Design report for Rosseau River Bridge Replacement (Geocres No. 31E-361), for report completeness recommendations for deep foundation option with steel H-Pile socketed into the bedrock also included in Section 4.1.2 below.

Boreholes for east abutment of the new permanent bridge were drilled as close as practical, based on construction activities on site. Borehole 6 was done as designated and BH 5 set back to the east about 3 m to 4 m to accommodate existing construction material stockpile. This information is presented in Foundation Investigation and Design report for Rosseau River Bridge Replacement (Geocres No. 31E-361).

Design of the footings to the east abutment was set at Elevation 253.7 m and did not accommodate the potential for sloping rock. The rock in this location slopes towards the river and to a lesser extent to the south; probes by the contractor during construction identified sloping rock with lower level than at BH5 and 6.

The additional field investigation for proposed Rosseau River Bridge East Foundation was necessary to confirm condition at the west edge of the proposed footing. The work was performed on July 15, 2016 and July 18, 2016 after the platform was built. The layout for the front of the footing and elevation of platform (Elev. 256.0 m, see attached sketch) were provided by the by the Contract Administrator (CA), refer to the attached site plan. The field program consisted of drilling of three (3) sampled boreholes (E1-16, E2-16 and E3-16) and two (2) probeholes (PB1-16 and PB2-16). These boreholes (E1-16, E2-16 and E3-16) were strategically located at the north corner, center line and south corner of the front or west edge of the east footing, respectively to verify subsurface information and depth to the bedrock at the east footing. A dynamic cone penetration test (DCPT) in probeholes (PB1-16 and PB2-16) were performed adjacent to boreholes E3-16 and E1-16, respectively to verify the blow count and depth to the bedrock. PB1-16 was advanced about 1.83 m north from BH E3-16 and PB2-16 was advanced about 1.2 m south from BH E1-16. The borehole and probehole locations are shown on Drawing 1 in Appendix B.

Boreholes (E1-16, E2-16 and E3-16) and probeholes (PB1-16 and PB2-16) were advanced using a track mounted D-56 drill rig, equipped with a hollow stem auger/diamond drilling and standard soil sampling equipment operated by a specialist drilling contractor, Walker Drilling Ltd.

The drilled boreholes (E1-16, E2-16 and E3-16) were advanced to depths of 6.5 m ,9.7 m and 7.7 m , respectively, until auger/ split spoon refusal (BH E3-16), or cored 0.8 m (BH E1-16) to 2.5 m (BH E2-16) into the bedrock. The dynamic cone penetration test in probehole (PB1-16) was advanced up to 6.1 m until cone refusal or rod bend (1.83 m) in probehole (PB2-16).



2.0 Site Description

The Rosseau River Bridge is located on Hwy 141, approximately 4.9 km east of the junction of Hwy 632. The Rosseau River Bridge is currently under rehabilitation and as part of the rehabilitation scope of the work; both east and west abutments were to be rehabilitated. The existing super structure were removed and the platform at east abutment location was build approximately at the existing footing level.

During the fieldwork for the east abutment foundation on July 15 and 18, 2016, the general site conditions were assessed. Hwy 141 runs in a generally east to west direction, and Rosseau River flows from north to south at the site, towards the Rosseau Lake. The Hwy 141 was temporarily diverted to temporary bridge with signalized one way traffic at Rossea river Bridge location. The banks of the river in the vicinity of the Bridge contained gravel, cobbles and boulders, and vegetation including grass, trees and shrubs were noted at the banks further away from the bridge.

3.0 Subsurface Conditions

As encountered in Boreholes E1-16 to E3-16, in general , the subsurface soils at the east abutment footing consist of a layer of sand and gravel fill underlain by native deposit of sand followed by sand and gravel layer and bedrock. The sand and gravel fill was encountered at the surface of the boreholes in all boreholes. The thickness of this layer ranged from 1.5 m to 2.3 m extended to Elev. 253.7 m to 254. In BH E2-16 about 0.92 m thick boulder layer was encountered in between sand and gravel fill layer. The fill was generally described as loose, moist to wet , grey to brown, and containing some cobbles and boulders. The fill layer was underlain by native sand deposit extended to depth between about 3.0 m and 3.8 m below the ground surface. The thickness of this sand deposit ranged from 1.5 m to 3.8 m extended to Elev. 252.2 m to 253 m. This native soil layer was generally described as loose to compact, brown to brownish grey, wet, containing trace to some silt, trace to some gravel. The native sand deposit was followed by sand and gravel layer extended to depth between about 5.7 m and 7.2 m below the ground surface. The thickness of this sand and gravel deposit ranged from 2.3 m to 3.4 m extended to Elev. 248.9 m to 250.4 m. The sand and gravel layer was generally described as loose to compact, brown to greyish brown, wet, containing trace to some silt, occasional cobbles and boulders. The native sand and gravel layer was followed by bedrock in boreholes E1-16 and E2-16; However in BH E3-16 it was followed by lower sand layer and bedrock. The lower sand layer encountered in borehole E3-16 was extended to depth of 7.7 m below ground surface extending to Elev. 248.3 m. The explore thickness of this layer was 1.6 m. It was generally describe as loose, greyish brown, wet, containing some silt, some gravel and some cobbles. The borehole E3-16 was terminated within this layer. The bedrock in boreholes were encountered in various depth, generally sloping from north to south. Bedrock was encountered at a depth between 5.7 m (BH E1-16) and 7.7 m (BH E3-16). The bedrock was proved either by auger/split spoon refusal (BH E3-16) or cored 0.8 m (BH E1-16) to 2.5 m (BH E2-16) into the bedrock. The boreholes E1-16 and E2-16 were terminated within bedrock.

Since, wash boring technique or water was used to advanced boreholes, the accurate groundwater level in boreholes could not be measured during short duration. However, groundwater level should be



expected slightly higher than water level in river (255.1, Dec 2015). Seasonal variations in the water table should be expected, with higher levels occurring during wetter periods of the year and lower levels during drier periods.

The detailed subsurface conditions encountered in the boreholes advanced during this investigation are presented on the borehole log sheets in Appendix C. A borehole location plan and stratigraphic section are provided in Appendix B.

4.0 Comments and Recommendations

It was initially proposed to remove material within the confines of a sheet pile cofferdam and remove materials above bedrock in the wet, placing tremie concrete on the bedrock to the level of the proposed footing (see the contractors proposed method statement/approach attached). Given the deeper depth to rock to the west and south at the proposed footing location, some alternative approaches to constructing the footing can be considered. The options include the cofferdam and excavation or partial excavation of the over burden material. The amount of groundwater depression is restricted by the condition at the site including construction, integrity and bracing requirements of any proposed sheet pile cofferdam. Current thinking is that the sheeting will not be a closed cell, but a 'C' section enveloping the west , north and south sides of the foundation. The erratic and sloping nature of the hard bedrock would likely result in the sheet piles 'hung-up' on rock with 'window' of overburden resulting.

Based on sub surface information obtained from the boreholes recommendations pertaining to the east abutment foundations of the Rosseau River Bridge are summarized in the following sections. The geotechnical parameters provided in the memorandum are recommended in accordance with the *Canadian Highway Bridge Design Code (CHBDC) (CAN/CSA-S6-14)*.

4.1 Structure Foundations

4.1.1 Shallow Foundation

Initially spread footing option was selected by the designer. The following 3 alternative approaches are considered feasible given the condition established by the investigation. In looking at these options, the bracing requirements for any sheeting proposed for cofferdam must be examined.



Table 1. Footing Options

Option	Material at Founding Level	Foundation Elevation (m)	Foundation Location	Approx. Excavation Elevation (m)	Rank
1	Tremie concrete to underside of footing over native loose to compact sand and gravel, limited pumping of foundation, to the extent practical. Regular concrete placement if dewatering and safe excavation done	253.7	Three quadrant of east footing (ie. North halve and south east quadrant of east footing)	252.7	1
			South quadrant of east footing (i.e south west corner of east footing)	251.8	
2	Tremie concrete over bedrock, limited pumping of foundation, to the extent practical.	253.7	Three quadrant of east footing (ie. North halve and south east quadrant of east footing)	250.4	2
			South quadrant of east footing (i.e south west corner of east footing)	248.3	
3	1.7 m thick Granular A Pad over Tremie concrete over bedrock, limited pumping of foundation, to the extent practical.	253.7	Three quadrant of east footing (ie. North halve and south east quadrant of east footing)	250.4	3
			South quadrant of east footing (i.e south west corner of east footing)	248.3	

Option 1 if executed properly, is considered the best approach given the circumstances. It is understood that Option 1 is also preferred by the Contractor and MTO. In order to execute Option 1, two possible approaches are available and their advantages and disadvantages are described below. However, method development including dewatering/piping control is the Contractor's responsibility; the comments on approaches given below are for general guidance. In executing this option, the sheet piles should be extended to the bedrock surface to the extent practical, given the conditions and to permit flexibility in carry out the work. Given the history of this site, consideration should be given to incorporating this sheeting into a suitable scour protection scheme.

Suggested approaches to execute Option 1

- a) Excavation in the dry
 - Excavation by backhoe with smooth bucket.



- Surface compaction
- Piping or “boiling” must be controlled. When piping takes place the upward seepage reduces the effective weight of the soil thereby reducing the ability to offer lateral support for the sheeting.
- Piping is normally controlled by dewatering outside the coffer dam and/or by driving the sheeting deeper

b) Excavation in the wet and placing concrete by tremie methods

- Dewatering to lower head as possible
- Excavation by backhoe with smooth bucket and/or air lift starting from the South-West side (deeper side).
- Examination of excavated surface by probing
- Place tremie concrete
- Extend excavation approximately 1.0 m to the East to account for the material removal
- Place tremie concrete as desire
- Leave sheeting in place

Table 2. Advantages and Disadvantages of approaches to execute Option1

Option	Advantage	Disadvantage
Excavation in the dry	<ul style="list-style-type: none"> • More confidence with excavated surface • Less potential for disturbance of subsoil • Can perform proofrolling before placing concrete • Mass concrete can be placed in dry condition • Sheeting can be sacrificed to enhance scour protection 	<ul style="list-style-type: none"> • Need extensive dewatering • Risk of piping if not properly executed • High cost for installation of dewatering wells or well points
Excavation in the wet and placing concrete by tremie methods	<ul style="list-style-type: none"> • More cost effective • Easier to execute • Less dewatering scheme is required • Low risk of piping or “boiling” • Sheeting can be sacrificed to enhance scour protection 	<ul style="list-style-type: none"> • Difficult to clean and verify surface to receive tremie • No proofrolling can be done • Difficult to measure quality of tremie concrete

4.1.1.1 Geotechnical Resistances

In the context of the CHBDC, a satisfactory foundation design would require, in terms of Limit States Design, the factored geotechnical resistance of its foundation to withstand and not exceed the imposed Ultimate Limit State loads - (ULS) Design Approach, and its ability to deform acceptably under the Service Limit State loads - (SLS) Design Approach. These associated loads are typically known as unfactored and factored loads, respectively.



Therefore, spread footings placed on the properly prepared subgrade at the design levels given in Table 3, should be designed based on the factored resistances at ULS and geotechnical reactions at SLS for 25 mm of settlement given in Table 2 below. The footing width of 3 m is assumed.

As loading information was not available at the time of the authoring of the report and as such, the below values assume a vertical concentric loading condition only.

Table 3. Geotechnical resistance at ULS and geotechnical reaction at SLS for a 3 m wide footing

Option Number	Soil at Founding level	Foundation Elevation ¹ (m)	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa) ^{2,3}
1	Tremie concrete over native loose to compact sand and gravel ⁴	253.7	450	300
2	Tremie concrete over bedrock		750	500 ⁶
3	1.7 m thick Granular A Pad ⁵ over Tremie concrete over bedrock		600	400

Note: ¹ Founding elevation based on information provided on email dated July 22, 2016.

² for concentric vertical loading condition only. Load eccentricity and load inclination effects need to be considered in accordance with CHBDC (2014).

³ for maximum settlement of 25 mm

⁴ Careful construction techniques are critical in excavations for the footings so as not to disturb the subgrades and should be subject to strict QA procedures

⁵ the granular material used for the granular pad shall be granular 'A' conforming to OPSS 1010 and compacted to 100 % SPMDD. Please refer to MTO standard drawing in Appendix for pad dimensions and side slopes.

⁶ Higher bearing pressures for option 3 are possible depending on the efficiently removal process for loose material over bedrock.

Since the ULS resistance and the settlement depend on the footing size and depth of embedment, the geotechnical resistances given in Table 4 should be reviewed if the selected footing width or founding elevations differ from those given in the table.

Tremie procedure must conform to MTO and good practice.



4.1.1.2 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance between the subgrade and concrete should be calculated in accordance with Section 6.7.5 of the CHBDC. The unfactored values of the coefficient of friction, $\tan \delta$, between the base of cast-in-place concrete footing and the granular subgrade soils below the frost level are presented in Table 4. A factor of 0.8 should be applied in calculation of the horizontal resistance in accordance with CHBDC.

Table 4. Recommendations for coefficient of friction

Interface	Coefficient of Friction, $\tan \delta^*$
tremie Concrete and sand and gravel subgrade	0.55
Granular A Pad over tremie concrete	0.65
Concrete and tremie concrete	0.65

*- based on NAVFAC 1986, Table 1, pg. 7.2-63

4.1.2 Deep Foundation

Considering the site specific conditions, steel H-piles (HP 310 x 79 or HP 310 x 110) can be used to support a bridge designed with integral abutments or semi-integral abutments. The piles will be installed through the upper loose to compact sandy deposits, and socketed into bedrock. The socketing of minimum 1.5 m into the unyielding bedrock is recommended. However the socketing length of piles at back row can be shortened to 1.0 m. Based on the depth to bedrock encountered in the deep boreholes drilled at the locations during current and previous investigation (Geocres No. 31E-361) of the proposed structure it appears that the termination depths for the piles could be variable. However, for design purpose, the tip elevations for the piles discussed in this report are estimated and given in Table 5. It should be noted that the minimum length of the pile above the bedrock surface should be 5 m. It is anticipated that pile cap elevations would be below a frost depth of 1.8 m.

4.1.2.1 Geotechnical Axial Resistances of Piles

The factored geotechnical axial resistances at ULS and geotechnical axial reactions at SLS for 25 mm of displacement for the recommended driven piles are presented in Table 5. These values represent the structural capacity of the steel member having a steel yield strength of 300 MPa, rather than a geotechnical limitation. It is anticipated that for H-piles driven and seated on the underlying unyielding bedrock, the geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored axial resistance at ULS; as such, ULS conditions will govern for this foundation type.



Table 5. Factored geotechnical resistances for considered piles

Abutment	Pile Founding Stratum	Estimated Tip Elevation (m)	Approx. Design Pile Length (m)	Factored Geotechnical Axial Resistance at ULS (kN/pile)		Geotechnical Axial Resistance at SLS (kN/pile)	
				HP 310 x 79	HP 310 x 110	HP 310 x 79	HP 310 x 110
East	~ 1.5 m Socketed into Bedrock ¹	Approx. 249.3' to 247.5 (minimum 1.5 m deep socket)	5.0 ²	1,450	2,000	NA ³	NA ³

Notes:

1. Socketing length of minimum 1.5 m for front row piles and minimum 1.0 m for back row piles
2. Based on cut- off Elevation 254.3 m provided on GA drawing and minimum 5 m length requirement for integral abutment
3. NA-not applicable since for H-piles driven and seated on the underlying unyielding bedrock, the geotechnical resistance at SLS for 25 mm of settlement will be greater than the factored axial resistance at ULS and ULS conditions will govern

4.1.2.2 Resistance of Piles to Lateral Loads

For vertical piles, the resistance to lateral loading has to be derived from the soil in front of the piles. That resistance may be estimated using Subgrade Reaction Theory (with deformations less than 5% of pile diameter) in which the coefficient of horizontal subgrade reactions k_s is based on the following equations:

For cohesionless soils:

$$k_s = n_h(z/d)$$

where,

k_s =coefficient of horizontal subgrade reactions (MPa/m)

d =pile diameter (m)

n_h =constant of horizontal subgrade reaction (MPa/m)

z =depth below ground surface (m)

The recommended value of n_h is 5 MPa/m for loose to compact silty sand encountered at this site.



Lateral loading could be resisted fully or partially by use of battered piles. The front piles could be installed at a batter of up to 4 vertical to 1 horizontal while the back row piles can be battered up to 10 vertical to 1 horizontal by simply tilting the pile-driver leads.

Group action for lateral loading should be considered where the pile spacing in the direction of the loading is less than eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor R, as indicated in Table 6. Subgrade reaction reduction factors for other pile spacing values may be interpolated for pile spacing in between those listed in this table.

Table 6. Lateral load capacity reduction factor for pile group

Pile Spacing in Direction of Loading D=Pile Diameter/Width	Subgrade Reaction Reduction Factor R
8d	1
6d	0.7
4d	0.4
3d	0.25

4.1.2.3 Negative Skin Friction (Downdrag Loads) on Piles

Since there is no significant raise of the approach embankment and the foundation soil is cohesionless, the negative skin friction (or downdrag load) will not need to be taken into consideration during design of the piles supporting the integral abutment.

4.1.3 Frost Protection

The frost depth in the area of the bridge is estimated to be approximately 1.8 m in accordance with OPSD 3090.100. During construction of any temporary and permanent support system using shallow foundations should be provided a minimum 1.8 m of soil cover or equivalent frost protection should be provided using thermal insulation.

4.1.4 Lateral Earth Pressures

The abutment stems, temporary shoring and cofferdam that may be required for excavation should be designed to resist lateral earth pressure. Where the abutment stems can be drained effectively to eliminate hydrostatic pressure on the walls, earth pressures equation can be simplified in accordance with the the Canadian Highway Bridge Design Code (CHBDC).

The expression for calculating lateral earth pressure is given by:

$P = K(\gamma h + q)$ for non-braced cut, or $K (0.65\gamma H + q)$ for braced support



where

P = earth pressure intensity at depth h , kPa

K = earth pressure coefficient

γ = unit weight of retained soil, kN/m³

q = surcharge near wall, kPa

h = depth to point of interest, m

H = depth of excavation, m

The mobilization of full active or passive resistance requires a measurable and perhaps significant wall movement or rotation. Therefore, unless the structural element can tolerate these deflections, the at-rest earth pressure should be used in design.

The effect of compaction surcharge should be taken into account in the calculations of active and at-rest earth pressures. The lateral pressure due to compaction should be taken as at least 12 kPa at the surface, and its magnitude should be assumed to diminish linearly with depth to zero at the depth where the active (or at rest) pressure is equal to 12 kPa. This pressure distribution should be added to the calculated active (or at rest) pressure. Notwithstanding, lighter compaction equipment and smaller lifts should be used adjacent to walls to prevent overstressing.

For design purposes, the unfactored static earth pressure parameters given in Table 7 can be used (assuming wall friction is neglected, the back wall is vertical and the ground surface is horizontal both on the retained side as well as in front of the toe):

Table 7 Material types and unfactored earth pressure properties under static conditions

Material	Unfactored Friction Angle ϕ' (°)	Coefficient of Active Earth Pressure (K_a)	Coefficient of Passive Earth Pressure (K_p)	Coefficient of Earth Pressure at Rest (K_o)	Unit Weight γ kN/m ³
Compacted Granular A	35	0.27	3.69	0.43	22
Compacted Granular B, Type II	32	0.31	3.25	0.47	21
Native Loose to Compact Sand	30	0.33	3.0	0.5	20
Native Compact Sand and Gravel	32	0.31	3.25	0.47	21



5.0 Construction Considerations

5.1 Excavations

All excavations must be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety (OHSA) and good construction practice. The soils which should be excavated for construction of the abutments (i.e. sand and gravel fill to loose to compact sand) are considered as Type 3 soils above the groundwater table and Type 4 soils below the groundwater table. Temporary excavations (i.e. those that are open only for a short period) above the groundwater table may be made with side slopes not steeper than about 1H:1V, while the temporary slopes below the groundwater table have to be formed at 3H:1V unless a suitable dewatering system is installed to lower the water level below the base of the excavation or the excavation is braced within the confines of a properly designed cofferdam.

5.2 Temporary Shoring/Scour Protection

Temporary excavation support systems, if any, should be designed and constructed in accordance with OPSS.PROV 539. The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS.PROV 539. Requirement to retaining sheeting for scour protection is proposed.

5.3 Construction Sequence for East abutment pile installation

- Drill the holes into the soil and rock as required
- Clean the holes. The drilled holes for sockets should be made clean from mud and loose soil particles before pouring grouting (tremie concrete). The grouting material should be non-shrinkable material with smaller size of material not larger than sand particle.
- Fill the lower part of the holes inside the rock with tremie concrete (1.5 m front row/ 1.0 m back row).
- Install the piles into the tremie filled holes making sure they sit on bedrock and are centered in the holes.
- Wait 24 hrs and fill the remaining portion of the holes above tremie with clear stone.

5.4 Cofferdam

The design of these cofferdams, which are temporary retaining structures, is the responsibility of the Contractor. The cofferdam must be designed to withstand the anticipated design loads and to be watertight as practically possible. The Contractor is also responsible for cofferdam's materials, construction, monitoring and removal. Cofferdams should be designed in accordance with OPSS 539 by a licensed Professional Engineer experienced in shoring design.



5.5 Dewatering


Dewatering shall be carried out in accordance with OPSS 517 and OPSS 518. It is responsibility of the Contractor to propose a suitable dewatering system based on the time of construction, water levels and river flow conditions for prior approval of the MTO. The method used should not undermine the existing road embankment or adjacent side slopes.

Dewatering may require water taking permits (i.e. Permit To Take Water -PTTW). A PTTW is required for any water taking if the volume exceeds 50,000 L/day. The rate and volume required for dewatering will be dependent on construction methods and staging chosen by the Contractor.

We trust the above meets with your present requirements. If you have any question, please contact us.

Sincerely,


for Nimesh Tamrakar, M.Eng.
Technical Specialist


Stan E. Gonsalves, M.Eng., P.Eng.
Executive Vice- President
Designated MTO Contact


TaeChul Kim, M.E.Sc., P.Eng.
Senior Foundation/Geotechnical Specialist



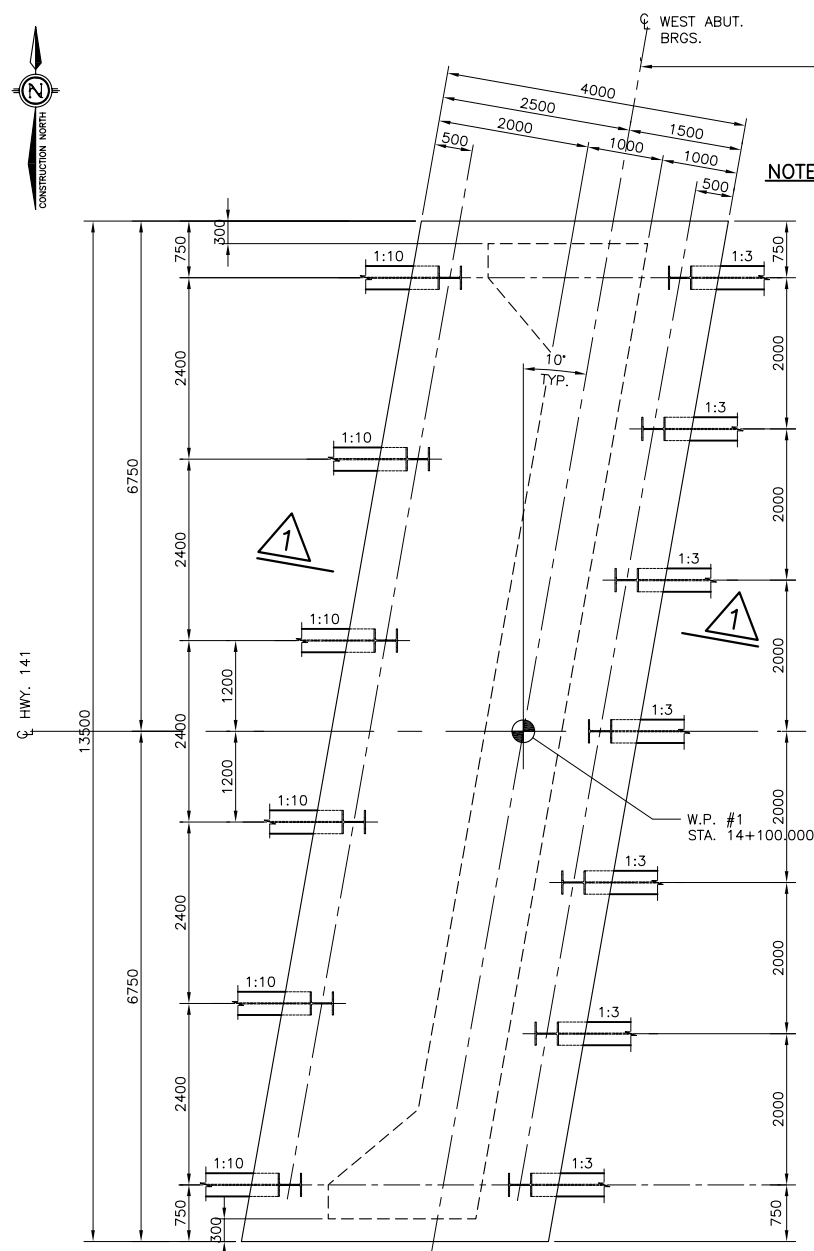
Appendices

- Appendix A: General Arrangement Drawing
- Appendix B: Borehole location Plan and Stratigraphic sections
- Appendix C: Borehole log
- Appendix D: Granular A Core Drawing
- Appendix E: Contractors method statement/approach

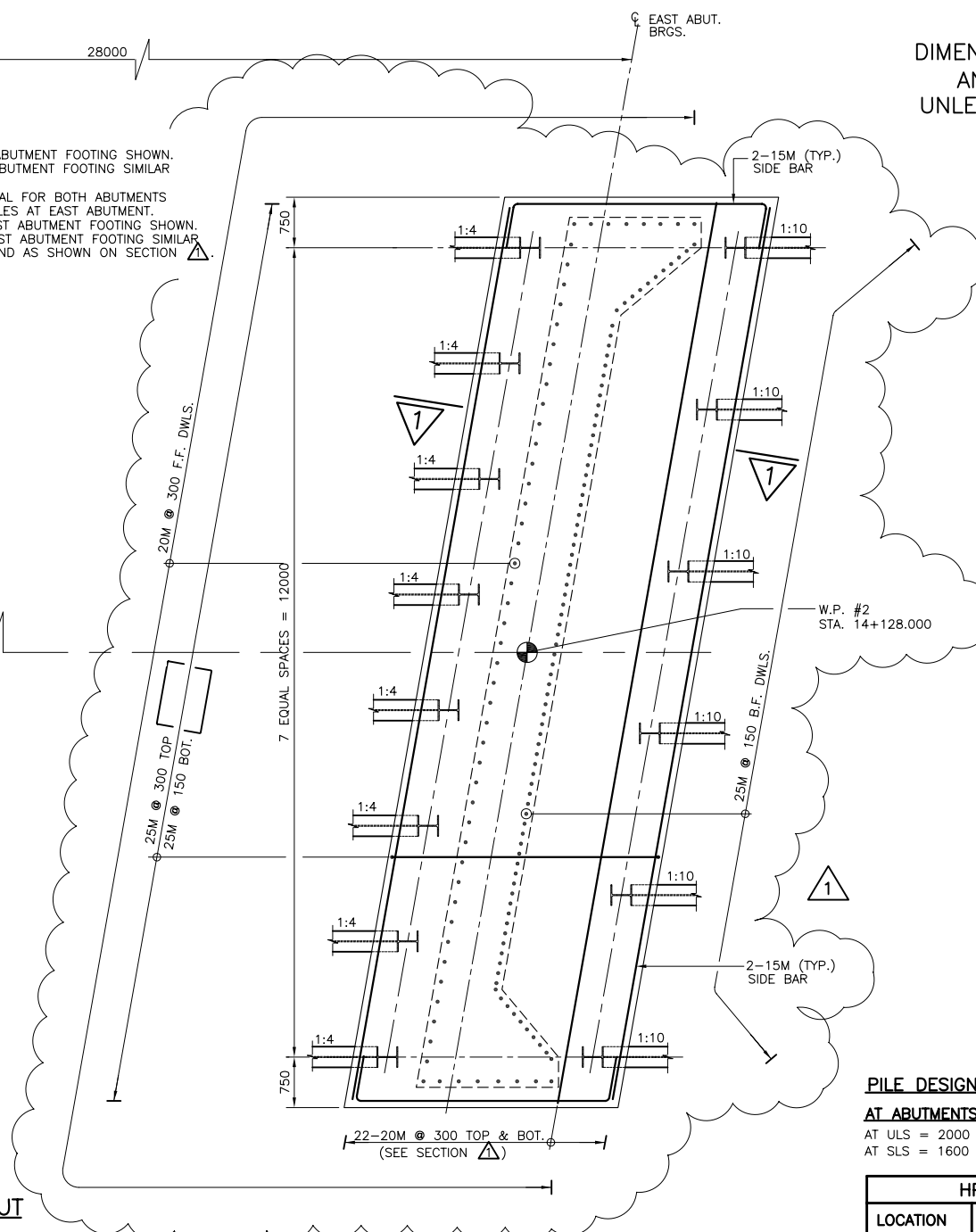


Appendix A

General Arrangement Drawing

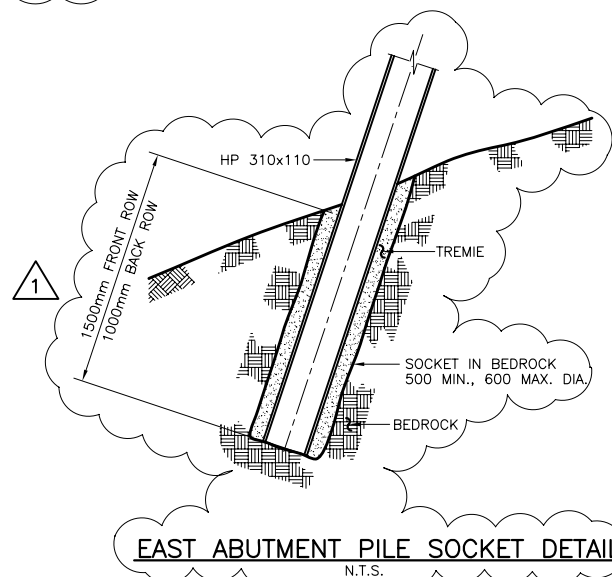
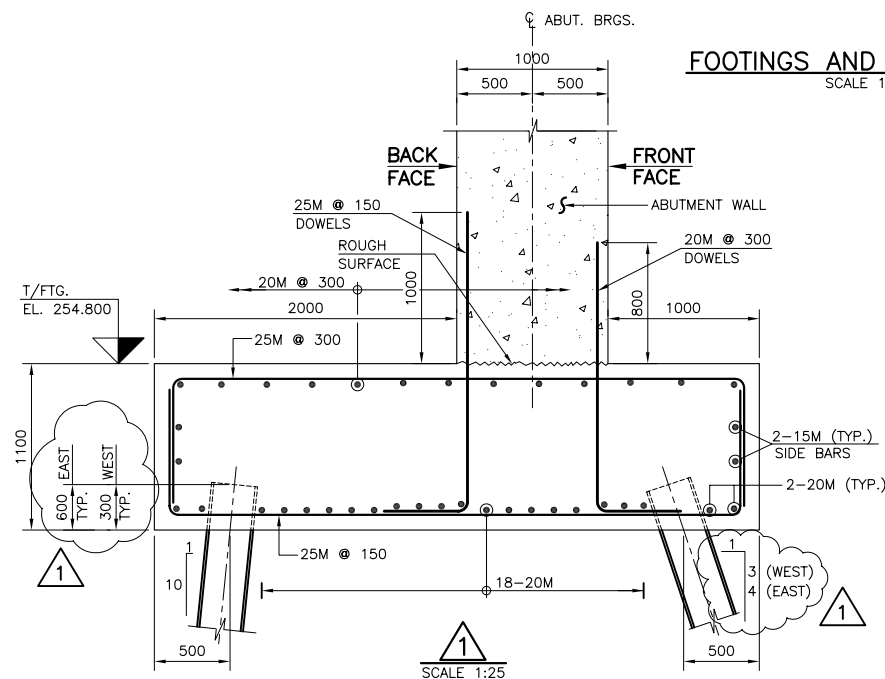


NOTE: DIMENSIONS AT WEST ABUTMENT FOOTING SHOWN.
DIMENSIONS AT EAST ABUTMENT FOOTING SIMILAR
BUT OPPOSITE HAND.
PILE PATTERNS IDENTICAL FOR BOTH ABUTMENTS
EXCEPT FOR FRONT PILES AT EAST ABUTMENT.
REINFORCEMENT AT EAST ABUTMENT FOOTING SHOWN.
REINFORCEMENT AT WEST ABUTMENT FOOTING SIMILAR
BUT OPPOSITE HAND AND AS SHOWN ON SECTION A



FOOTINGS AND PILE LAYOUT

SCALE 1:50



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

CONT No 2016-5008
WP No 5394-15-01

ROSSEAU RIVER
BRIDGE REPLACEMENT
FOUNDATION LAYOUT

SHEET
28C



Ministry of Transportation
Highway Standards Branch
Bridge Office

NOTES:

1. THIS DRAWING TO BE READ IN CONJUNCTION WITH DWGS. 6 AND 7.
2. WEST ABUTMENT PILES TO BE FITTED WITH DRIVING SHOES AS PER OPSPD-3000.1000 .
3. WEST ABUTMENT PILES TO BE FITTED WITH ROCK DRIVING SHOES AND DRIVEN INTO BEDROCK IN ACCORDANCE WITH OPSS 903.
4. LAYOUT DIMENSIONS FOR PILES ARE GIVEN AT THE UNDERSIDE OF FOOTING.
5. PILE LENGTHS SHOWN ARE THE THEORETICAL LENGTHS FROM THE CUT-OFF TO THE ESTIMATED PILE TIP ELEVATION. ACTUAL LENGTHS WILL BE DETERMINED IN THE FIELD.
6. ALL PILES ARE HP 310x110 STEEL H PILES.
7. THE CONTRACTOR IS REMINDED THAT EXISTING BEDROCK SURFACE ELEVATIONS MAY BE WIDELY VARIABLE.
8. DRILLED HOLES FOR SOCKETS SHALL BE MADE CLEAN FROM MUD AND LOOSE SOIL PARTICLES BEFORE POURING TREMIE.

CONSTRUCTION SEQUENCE FOR EAST ABUTMENT PILE INSTALLATION

1. DRILL THE 500 TO 600mm DIA. HOLES INTO THE SOIL AND ROCK AS SHOWN ON THE DRAWING.
2. CLEAN THE HOLES.
3. FILL THE LOWER PART OF THE HOLES INSIDE THE ROCK WITH TREMIE (1500mm FRONT/1000mm BACK ROW).
4. INSTALL THE PILES INTO THE TREMIE FILLED HOLES MAKING SURE THEY SIT ON BEDROCK AND ARE CENTERED IN THE HOLES.
5. WAIT 24 HOURS AND FILL THE REMAINING PORTION OF THE HOLES ABOVE TREMIE WITH CLEAR STONE.

LEGEND

W.P. DENOTES WORKING POINT
T/FTG. DENOTES TOP OF FOOTING

PILE DESIGN DATA


AT ABUTMENTS:

AT ULS = 2000 kN
AT SLS = 1600 kN

HP 310x110 PILE DATA				
LOCATION	BATTER	No. REQ'D	CUT-OFF ELEV.	LENGTH (m)
WEST ABUT.	1:3	7	254.000	11.25
FOOTING	1:10	6		10.75
EAST ABUT.	1:4	8	254.300	VARIABLE
FOOTING	1:10	6		VARIABLE



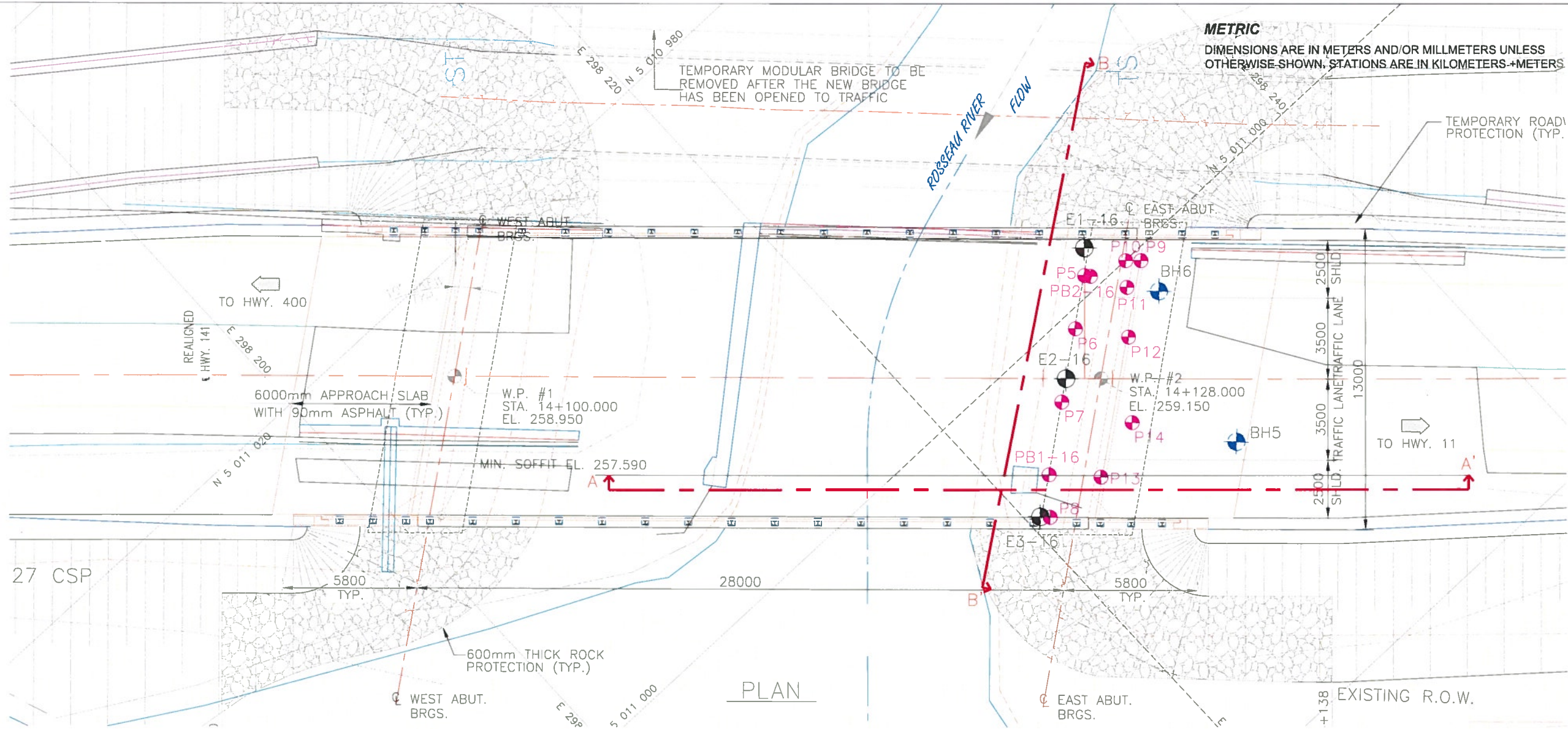
DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

REVISIONS							
		08/08/16	EAST ABUTMENT CHANGED TO PILED				
			DESCRIPTION				
DESIGN	S.U.	CHK	W.K.	CODE	S6-14	CL 625-ONT	DATE JAN. 2016
DRAWN	A.P.	CHK	S.U.	SITE	42-013		DWG 5

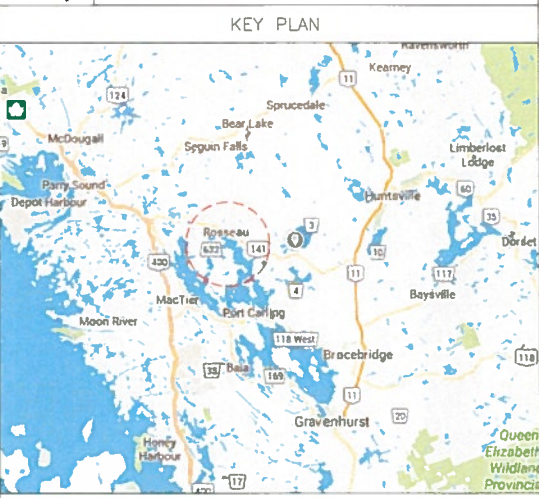


Appendix B

Borehole location Plan and Stratigraphic sections



METRIC
DIMENSIONS ARE IN METERS AND/OR MILLIMETERS UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETERS + METERS



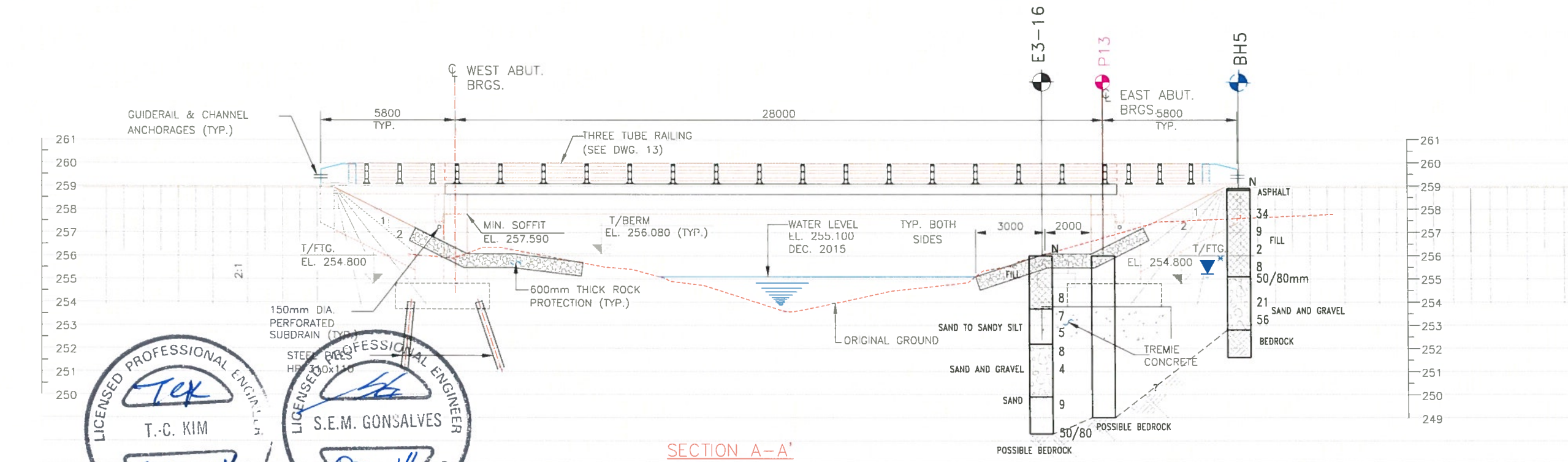
- LEGEND**
- Location of Current Investigation Boreholes
 - Location of Previous Investigation Boreholes
 - Probe Hole (DCPT)

- SOIL STRATA SYMBOLS**
- ASPHALT
 - FILL
 - COBBLES AND BOULDERS
 - BEDROCK
 - PEAT/SANDY PEAT
 - SILTY SAND
 - SAND AND GRAVEL

BH No.	APPROX. ELEV. (m)	MTM ZONE 10	
		NORTH	EAST
E1-16	256.0	5011001.6	298230.0
E2-16	256.0	5010998.0	298225.4
E3-16	256.0	5010994.4	298220.5
PB1-16	256.0	5010995.5	298222.0
PB2-16	256.0	5011000.5	298229.2
BH 5	258.9	5010994.5	298229.0
BH 6	258.9	5010998.0	298231.0
P5	256.0		
P6	256.0		
P7	256.0		
P8	256.0		
P9	256.0		
P10	256.0		
P11	256.0		
P12	256.0		
P13	256.0		
P14	256.0		

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

The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office. Downview. Information contained in the report and related documents are specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen. Cond.

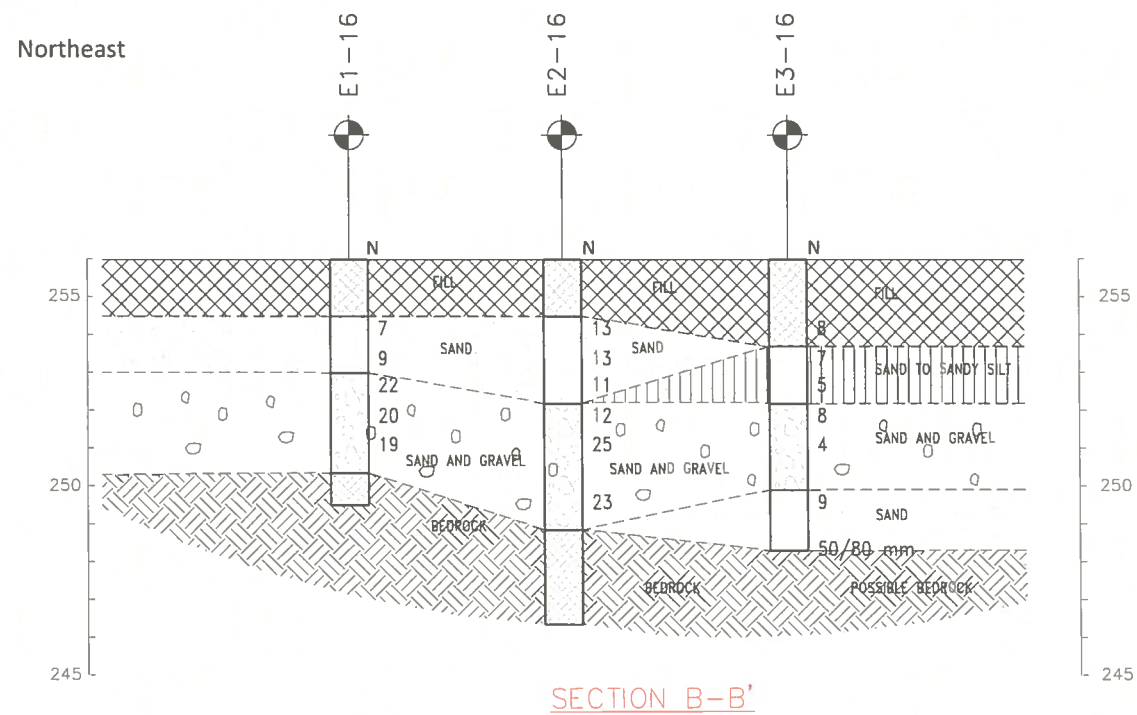
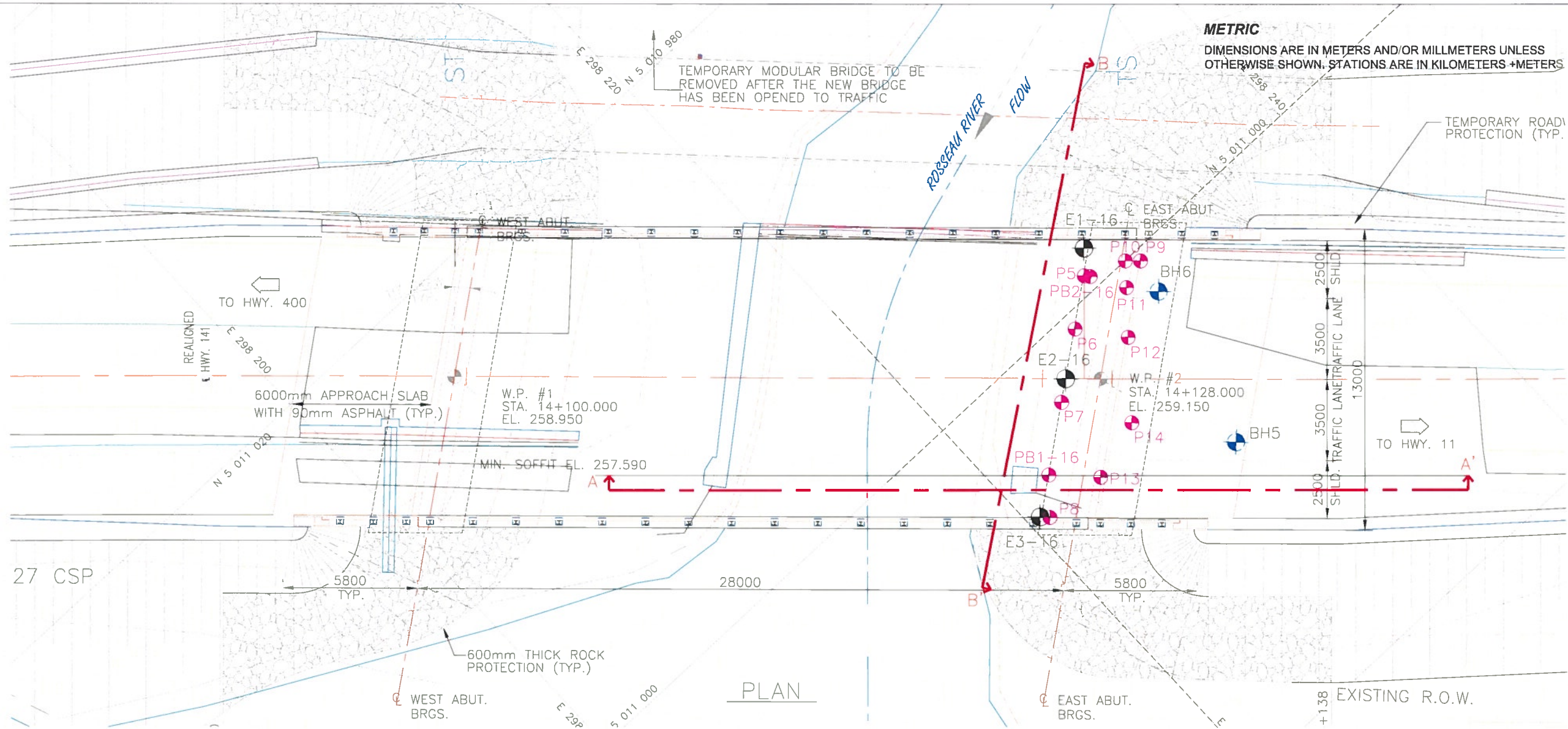


SECTION A-A'

LICENSED PROFESSIONAL ENGINEER
T.C. KIM
PROVINCE OF ONTARIO
Oct 14 '16

LICENSED PROFESSIONAL ENGINEER
S.E.M. GONSALVES
PROVINCE OF ONTARIO
Oct 14 '16

DATE	BY	DESCRIPTION
OCTOBER 2016	TK	FINAL SUBMISSION
JULY 2016	TK	SUBMISSION FOR REVIEW
SCALE		PROJECT NO. ADM 00233185-C0
SUBM'D TK	CHECKED TK	DATE OCT 2016 SITE No.
DRAWN SH	CHECKED TK	APPROVED ST DWG. 1



Southwest



AGREEMENT NO. 5015-E-0007
ASSIGNMENT No. 3
GWP No.



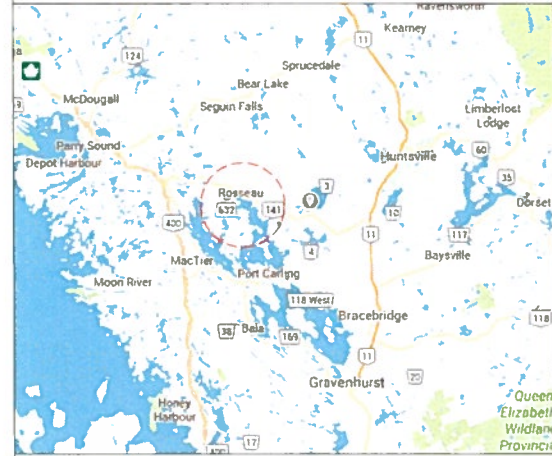
(SITE NO. 42-013, HWY 141)
ROSSEAU RIVER BRIDGE REPLACEMENT
SITE PLAN/ BOREHOLE LOCATIONS

SHEET
2



exp Services Inc.

KEY PLAN



LEGEND

- Location of Current Investigation Boreholes
- Location of Previous Investigation Boreholes
- Probe Hole (DCPT)

SOIL STRATA SYMBOLS

- ASPHALT
- FILL
- COBBLES AND BOULDERS
- BEDROCK
- PEAT/SANDY PEAT
- SILTY SAND
- SAND AND GRAVEL

BH No.	APPROX. ELEV. (m)	MTM ZONE 10	
		NORTH	EAST
E1-16	256.0	5011001.6	298230.0
E2-16	256.0	5010998.0	298225.4
E3-16	256.0	5010994.4	298220.5
PB1-16	256.0	5010995.5	298222.0
PB2-16	256.0	5011000.5	298229.2
BH 5	258.9	5010994.5	298229.0
BH 6	258.9	5010998.0	298231.0
P5	256.0		
P6	256.0		
P7	256.0		
P8	256.0		
P9	256.0		
P10	256.0		
P11	256.0		
P12	256.0		
P13	256.0		
P14	256.0		

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

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

DATE	BY	DESCRIPTION
OCTOBER 2016	TK	FINAL SUBMISSION
JULY 2016	TK	SUBMISSION FOR REVIEW
SCALE		PROJECT NO. ADM 00233185-C0
SUBM'D TK	CHECKED TK	DATE OCT 2016 SITE No.
DRAWN SH	CHECKED TK	APPROVED ST DWG. 2



Appendix C

Borehole logs

Explanation of Terms Used on Borehole Records

SOIL DESCRIPTION

Terminology describing common soil genesis:

Topsoil: mixture of soil and humus capable of supporting good vegetative growth.

Peat: fibrous fragments of visible and invisible decayed organic matter.

Fill: where fill is designated on the borehole log it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of site fill materials. All fills should be expected to contain obstruction such as wood, large concrete pieces or subsurface basements, floors, tanks, etc.; none of these may have been encountered in the boreholes. Since boreholes cannot accurately define the contents of the fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant ongoing and future settlements. Fill at this site may have been monitored for the presence of methane gas and, if so, the results are given on the borehole logs. The monitoring process does not indicate the volume of gas that can be potentially generated nor does it pinpoint the source of the gas. These readings are to advise of the presence of gas only, and a detailed study is recommended for sites where any explosive gas/methane is detected. Some fill material may be contaminated by toxic/hazardous waste that renders it unacceptable for deposition in any but designated land fill sites; unless specifically stated the fill on this site has not been tested for contaminants that may be considered toxic or hazardous. This testing and a potential hazard study can be undertaken if requested. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common and are generally not detected in a conventional geotechnical site investigation.

Till: the term till on the borehole logs indicates that the material originates from a geological process associated with glaciation. Because of this geological process the till must be considered heterogeneous in composition and as such may contain pockets and/or seams of material such as sand, gravel, silt or clay. Till often contains cobbles (60 to 200 mm) or boulders (over 200 mm). Contractors may therefore encounter cobbles and boulders during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited zone; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till materials.

Terminology describing soil structure:

Desiccated: having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.

Stratified: alternating layers of varying material or color with the layers greater than 6 mm thick.

Laminated: alternating layers of varying material or color with the layers less than 6 mm thick.

Fissured: material breaks along plane of fracture.

Varved: composed of regular alternating layers of silt and clay.

Slickensided: fracture planes appear polished or glossy, sometimes striated.

Blocky: cohesive soil that can be broken down into small angular lumps which resist further breakdown.

Lensed: inclusion of small pockets of different soil, such as small lenses of sand scattered through a mass of clay; not thickness.

Seam: a thin, confined layer of soil having different particle size, texture, or color from materials above and below.

Homogeneous: same color and appearance throughout.

Well Graded: having wide range in grain sized and substantial amounts of all predominantly on grain size.

Uniformly Graded: predominantly on grain size.

All soil sample descriptions included in this report follow generally the ASTM D2487-11 Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System) with some modification to reflect current MTO practices. The system divides soils into three major categories: (1) coarse grained, (2) fine-grained, and (3) highly organic. The soil is then subdivided based on either gradation or plasticity characteristics. The system provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification. The classification excludes particles larger than 76 mm. Please note that, with the exception of those samples where a grain size analysis has been made, all samples are classified visually in accordance with ASTM D2488-09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). Visual classification is not sufficiently accurate to provide exact grain sizing or precise differentiation between size classification systems. Others may use different classification systems; one such system is the ISSMFE Soil Classification.

ISSMFE SOIL CLASSIFICATION											
CLAY	SILT			SAND			GRAVEL			COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE	FINE	MEDIUM	COARSE		
<div><div></div><div>0.002</div><div></div><div>0.006</div><div></div><div>0.02</div><div></div><div>0.06</div><div></div><div>0.2</div><div></div><div>0.6</div><div></div><div>2.0</div><div></div><div>6.0</div><div></div><div>20</div><div></div><div>60</div><div></div><div>200</div><div></div></div>											
EQUIVALENT GRAIN DIAMETER IN MILLIMETRES											
CLAY (PLASTIC) TO				FINE		MEDIUM		CRS.		FINE COARSE	
SILT (NONPLASTIC)				SAND				GRAVEL			
UNIFIED SOIL CLASSIFICATION											

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 and as described below in accordance with Note 16 in ASTM D2488-09a:

Table a: Percent or Proportion of Soil, Pp

	Criteria
Trace	Particles are present but estimated to be less than 5%
Few	$5 \leq Pp \leq 10\%$
Little	$15 \leq Pp \leq 25\%$
Some	$30 \leq Pp \leq 45\%$
Mostly	$50 \leq Pp \leq 100\%$

The standard terminology to describe cohesionless soils includes the compactness as determined by the Standard Penetration Test 'N' value:

Table b: Apparent Density of Cohesionless Soil

	'N' Value (blows/0.3 m)
Very Loose	$N < 5$
Loose	$5 \leq N < 10$
Compact	$10 \leq N < 30$
Dense	$30 \leq N < 50$
Very Dense	$50 \leq N$

The standard terminology to describe cohesive soils includes consistency, which is based on undrained shear strength as measured by insitu vane tests, penetrometer tests, unconfined compression tests or similar field and laboratory analysis, Standard Penetration Test 'N' values can also be used to provide an approximate indication of the consistency and shear strength of fine grained, cohesive soils:

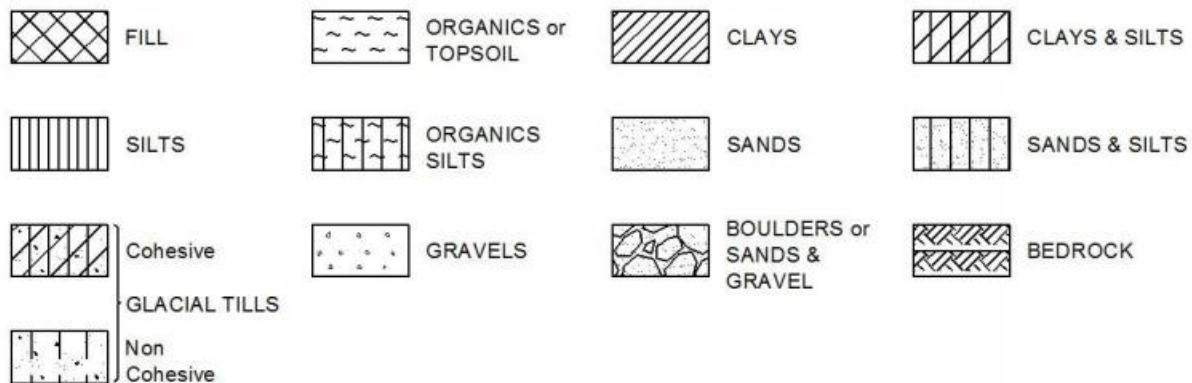
Table c: Consistency of Cohesive Soil

Consistency	Vane Shear Measurement (kPa)	'N' Value
Very Soft	<12.5	<2
Soft	12.5-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

Note: 'N' Value - The Standard Penetration Test records the number of blows of a 140 pound (64kg) hammer falling 30 inches (760mm), required to drive a 2 inch (50.8mm) O.D. split spoon sampler 1 foot (305mm). For split spoon samples where full penetration is not achieved, the number of blows is reported over the sampler penetration in meters (e.g. 50/0.15).

STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols:



WATER LEVEL MEASUREMENT



ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

SS	Split spoon sample (obtained from the Standard Penetration Test)
WS	Wash sample
BS	Bulk sample
TW	Thin wall sample or Shelby tube
PS	Piston sample
AS	Auger sample
VT	Vane test
GS	Grab sample
HQ, NQ, etc.	Rock core samples obtained with the use of standard size diamond drilling bits

STRESS AND STRAIN

u_w	kPa	Pore water pressure
r_u	1	Pore pressure ratio
σ	kPa	Total normal stress
σ'	kPa	Effective normal stress
τ	kPa	Shear stress
$\sigma_1, \sigma_2, \sigma_3$	kPa	Principal stresses
ε	%	Linear strain
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	Principal strains
E	kPa	Modulus of linear deformation
G	kPa	Modulus of shear deformation
μ	1	Coefficient of friction

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	Coefficient of volume change
c_c	1	Compression index
c_s	1	Swelling index
c_r	1	Recompression index
c_v	m ² /s	Coefficient of consolidation
H	m	Drainage path
T_v	1	Time factor
U	%	Degree of consolidation
σ'_{v0}	kPa	Effective overburden pressure
σ'_p	kPa	Preconsolidation pressure
τ_f	kPa	Shear strength
c'	kPa	Effective cohesion intercept
ϕ'	—°	Effective angle of internal friction
c_u	kPa	Apparent cohesion intercept
ϕ_u	—°	Apparent angle of internal friction
τ_R	kPa	Residual shear strength
τ_r	kPa	Remoulded shear strength
S_t	1	Sensitivity = c_u/τ_r

PHYSICAL PROPERTIES OF SOIL

P_s	kg/m ³	Density of solid particles
γ_s	kN/m ³	Unit weight of solid particles
ρ_w	kg/m ³	Density of water
γ_w	kN/m ³	Unit weight of water
ρ	kg/m ³	Density of soil
γ	kN/m ³	Unit weight of soil
ρ_d	kg/m ³	Density of dry soil
γ_d	kN/m ³	Unit weight of dry soil
ρ_{sat}	kg/m ³	Density of saturated soil
γ_{sat}	kN/m ³	Unit weight of saturated soil
ρ'	kg/m ³	Density of submerged soil
γ'	kN/m ³	Unit weight of submerged soil
e	1, %	Void ratio
n	1, %	Porosity
w	1, %	Water content
S_r	%	Degree of saturation
W_L	%	Liquid limit
W_P	%	Plastic limit
W_s	%	Shrinkage limit
I_p	%	Plasticity index = $(W_L - W_P)$
I_L	%	Liquidity index = $(W - W_P)/I_p$
I_C	%	Consistency index = $(W_L - W)/I_p$
e_{max}	1, %	Void ratio in loosest state
e_{min}	1, %	Void ratio in densest state
I_D	1	Density index = $(e_{max} - e)/(e_{max} - e_{min})$
D	mm	Grain diameter
D_n	mm	N percent - diameter
C_u	1	Uniformity coefficient
h	m	Hydraulic head or potential
q	m ³ /s	Rate of discharge
v	m/s	Discharge velocity
i	1	Hydraulic gradient
k	m/s	Hydraulic conductivity
j	kN/m ³	Seepage force

Brampton, Ontario

RECORD OF BOREHOLE No E1-16

1 OF 1

METRIC

W. P. _____ LOCATION Rosseau River Bridge East Footing, MTM Z10 (N5011001.6 E298230) ORIGINATED BY RB
 DIST HWY 141 BOREHOLE TYPE D-56, Hollow stem auger/Diamond Drill, Cased Hole COMPILED BY NT
 DATUM Geodetic DATE 2016/07/18 - 2016/07/18 CHECKED BY SG

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: Cu, KPa										WATER CONTENT (%)		
								○ UNCONFINED		+ FIELD VANE								× QUICK TRIAXIAL		
256.0	Ground Surface						20	40	60	80	100						GR SA SI CL			
	FILL: SAND AND GRAVEL some cobbles, moist, brown		1	AG																
254.5																				
1.5	SAND some gravel, trace silt, brownish grey, wet, Loose		2	SS	7															
			3	SS	9															
253.0																				
3.0	SAND AND GRAVEL grey, wet, Compact		4	SS	22															
	- Very little recovery, possibly pushing rock down		5	SS	20															
			6	SS	19															
250.4																				
5.7	BEDROCK grey/black, pink granite																			
	NQ Coring		7	NQ																
	Length (m) RQD (%)																			
	Run 1 0.85 98																			
249.5																				
6.5	END OF BOREHOLE																			
	NOTES: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. Interpretation assistance by exp is required before used by others. 3. Stabilized groundwater level couldnot be measure in short duration, since wash boring technique was used to advance boreholes.																			

EXP RECORD OF BOREHOLE 5015-E-0007 ASS(G.3-BH LOGS.GPJ) ONTARIO MOT.GDT 7/22/16

Brampton, Ontario

RECORD OF BOREHOLE No E2-16

1 OF 1

METRIC

W. P. _____

LOCATION Rosseau River Bridge East Footing, MTM Z10 (N5010998 E298225.4)

ORIGINATED BY NT

DIST HWY 141

BOREHOLE TYPE D-56, Hollow stem auger/Diamond Drill, Cased Hole

COMPILED BY NT

DATUM Geodetic

DATE 2016/07/15 - 2016/07/15

CHECKED BY SG

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			20	40	60	80	100					
256.0	Ground Surface																
	FILL: SAND AND GRAVEL WITH BOULDER some cobbles, moist, brown -Boulder @ 0.3 m upto 1.2 m		1	NQ			255										
254.5																	
1.5	SAND some gravel, trace silt, occasional cobbles, brownish grey to grey, wet, Compact		2	SS	13		254										
			3	SS	13												
			4	SS	11		253										
252.2																	
3.8	SAND AND GRAVEL greyish brown to grey, wet, Compact		5	SS	12		252										
			6	SS	25		251										
			7	SS	23		250										
248.9							249										
7.2	BEDROCK grey/black, pink granite NQ Coring		8	NQ			248										
	Length (m) RQD (%) Run 1 1.0 47 Run 2 1.5 90																
			9	NQ			247										
246.4																	
9.7	END OF BOREHOLE																
	NOTES: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. Interpretation assistance by exp is required before used by others. 3. Stabilized groundwater level couldnot be measure in short duration, since wash boring technique was used to advance boreholes.																

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

EXP RECORD OF BOREHOLE 5015-E-0007 ASSIG.3-BH LOGS.GPJ ONTARIO MOT.GDT 7/22/16

Brampton, Ontario

RECORD OF BOREHOLE No E3-16

1 OF 1

METRIC

W. P. _____ LOCATION Rosseau River Bridge East Footing, MTM Z10 (N5010994.4 E298220.5) ORIGINATED BY NT
 DIST HWY 141 BOREHOLE TYPE D-56, Hollow stem auger COMPILED BY NT
 DATUM Geodetic DATE 2016/07/15 - 2016/07/15 CHECKED BY SG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W _p	W	W _L		
256.0	Ground Surface																
	FILL: SAND AND GRAVEL some silt, some cobbles and boulder, some black organic spot, moist to wet, brown and grey		1	AG													
			2	SS	8												
253.7																	
2.3	SAND TO SANDY SILT trace gravel, some clay, brown to brownish grey, wet, Loose -150 mm clayey silt encountered @ 2.3 m		3	SS	7												
			4	SS	5												
252.2																	
3.8	-Started adding water in auger @ 3.81 SAND AND GRAVEL trace to some silt, brown to greyish brown, wet, loose to very loose		5	SS	8												
			6	SS	4												
	-Auger Grinding possible boulder @ 5.3 m																
249.9																	
6.1	SAND some silt, some gravel, some cobbles, greyish brown, wet, loose -Auger grinding @ 6.85 m up to 7.6 m, possible cobbles		7	SS	9												
248.3	-becoming sand and gravel		8	SS	50/80 mm												
7.7	END OF BOREHOLE																
	NOTES: 1. This drawing is to be read with the subject report and project numbers as presented above. 2. Interpretation assistance by exp is required before used by others. 3. Stabilized groundwater level couldnot be measure in short duration, since wash boring technique was used to advance boreholes.																

EXP RECORD OF BOREHOLE 5015-E-0007 ASS(G.3-BH LOGS.GPJ) ONTARIO MOT.GDT 7/22/16

1 OF 1

METRIC

W. P.

LOCATION	Rosseau River Bridge East Footing, MTM Z10 (N5010995.5 E298222)
----------	---

ORIGINATED BY NT

DIST HWY 141

BOREHOLE TYPE D-56, Dynamic Cone Penetration

COMPILED BY NT

DATUM Geodetic

DATE 2016/07/15 - 2016/07/15

CHECKED BY SG

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

Brampton, Ontario

RECORD OF BOREHOLE No PB2-16

1 OF 1

METRIC

W. P. _____ LOCATION Rosseau River Bridge East Footing, MTM Z10 (N5011000.5 E298229.2) ORIGINATED BY RB
 DIST HWY 141 BOREHOLE TYPE D-56, Dynamic Cone Penetration COMPILED BY NT
 DATUM Geodetic DATE 2016/07/15 - 2016/07/15 CHECKED BY SG

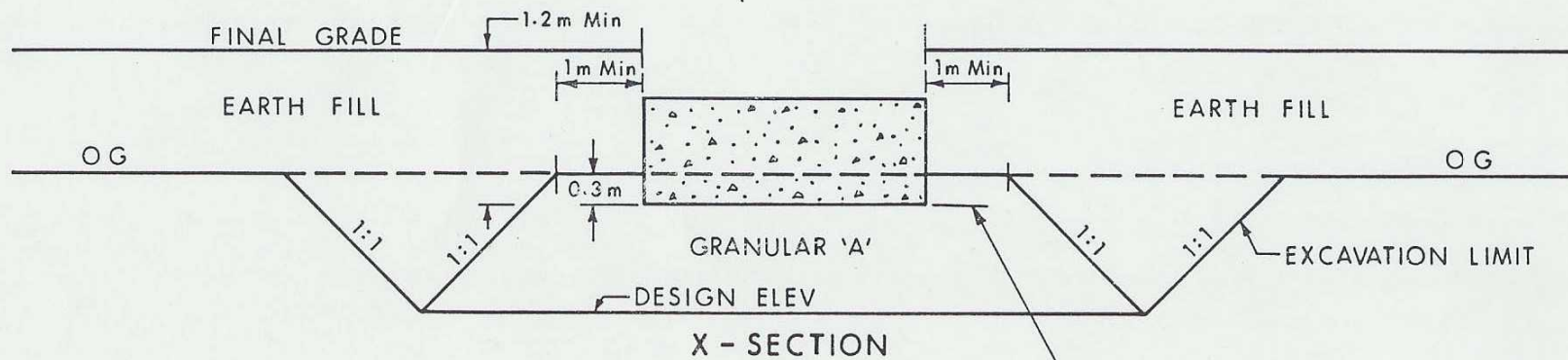
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20	40	60	80	100	W _p	W		
256.0	Ground Surface															
255																
254.2																
1.8	-Rod Bent END OF DCPT Rod Bent, Possible Boulder															

EXP RECORD OF BOREHOLE 5015-E-0007 ASSIG.3-BH LOGS.GPJ ONTARIO MOT.GDT 7/22/16

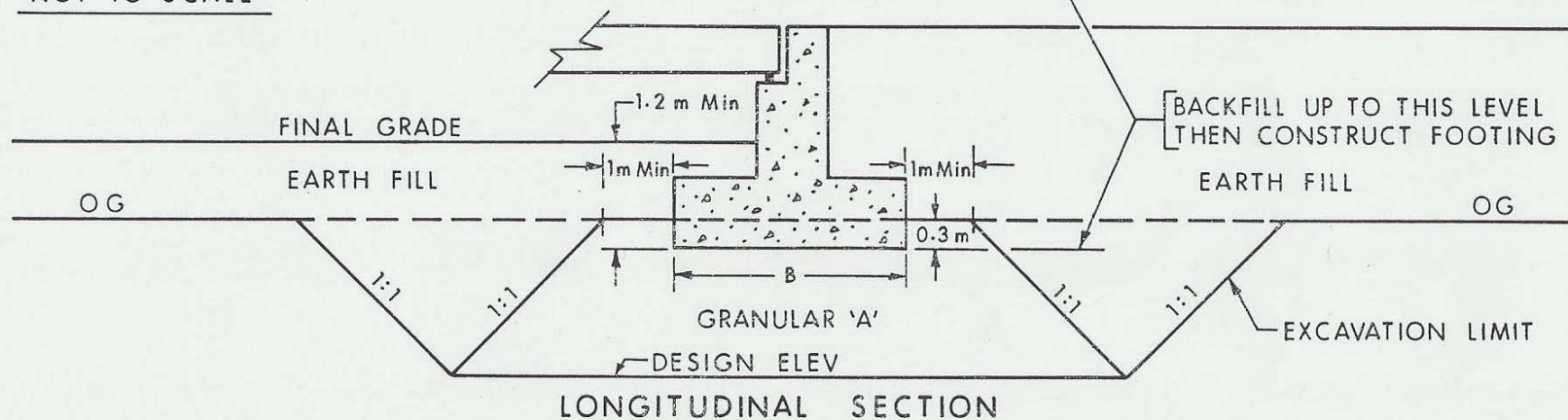


Appendix D

Granular A Core Drawing



NOT TO SCALE



NOTES:

- 1- EXCAVATE TO DESIGN ELEVATION UNDER AREA OF COMPACTED GRANULAR 'A'.
- 2- BACKFILL GRANULAR 'A' & EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT MTO STANDARDS.
- 3- CONSTRUCT CONCRETE FOOTING.
- 4- PLACE REMAINDER OF GRANULAR 'A' & EARTH FILL AS REQUIRED.



Ontario

Ministry of
Transportation

ABUTMENT IN EXCAVATED AREA
SHOWING GRANULAR 'A' CORE

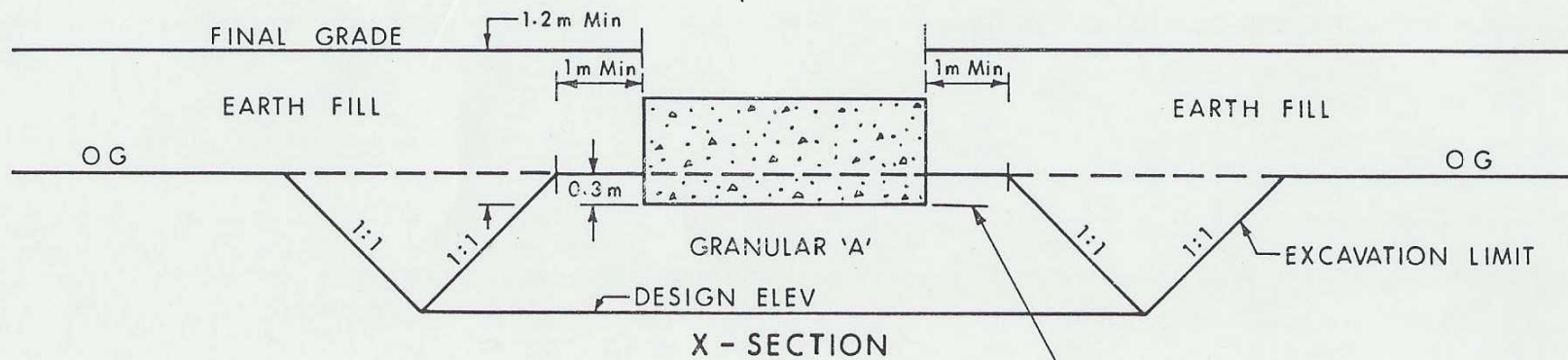
FIG No

W P

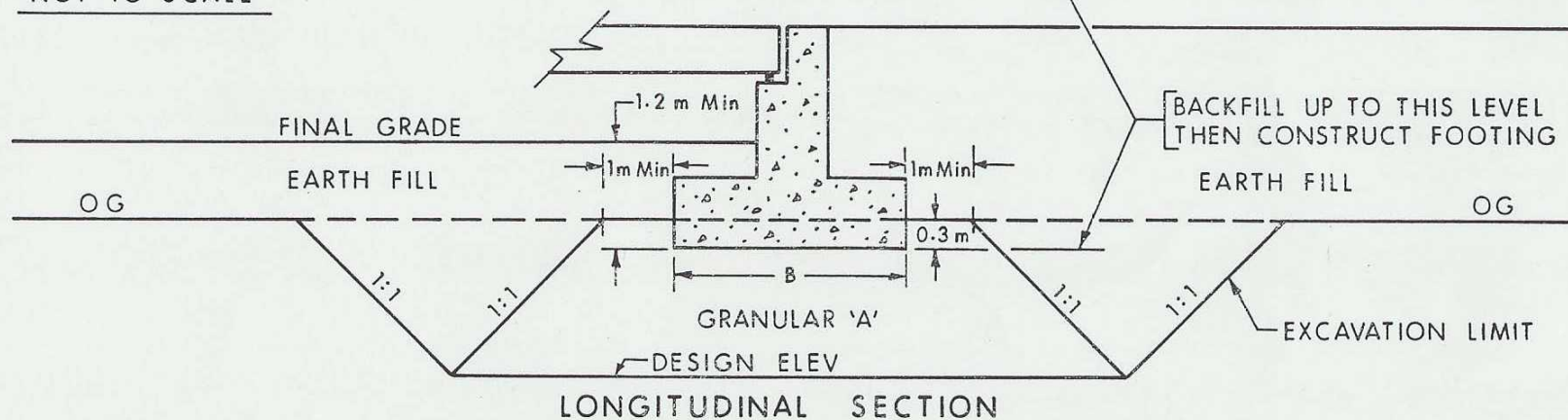


Appendix E

Contractors method statement/approch



NOT TO SCALE



NOTES:

- 1- EXCAVATE TO DESIGN ELEVATION UNDER AREA OF COMPACTED GRANULAR 'A'.
- 2- BACKFILL GRANULAR 'A' & EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT MTO STANDARDS.
- 3- CONSTRUCT CONCRETE FOOTING.
- 4- PLACE REMAINDER OF GRANULAR 'A' & EARTH FILL AS REQUIRED.



Ontario

Ministry of
Transportation

ABUTMENT IN EXCAVATED AREA
SHOWING GRANULAR 'A' CORE

FIG No

W P