FOUNDATION INVESTIGATION AND DESIGN REPORT
STURGEON RIVER BRIDGE REPLACEMENT
HIGHWAY 599
DISTRICT OF THUNDER BAY, ONTARIO
G.W.P. 6109-10-00, SITE NO: 48W-8

Geocres Number: 52J-9

Report to:

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the Sturgeon River Bridge on Highway 599 in the District of Thunder Bay, Ontario.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, records of boreholes, stratigraphic profile and cross-sections, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained in the course of the investigation.

Thurber carried out the investigation as a sub-consultant to WSP Canada Inc, under the Ministry of Transportation Ontario (MTO) Agreement Number 6010-E-0012.

2 SITE DESCRIPTION

The Sturgeon River Bridge is located on Highway 599, approximately 100 km north of Ignace, which is situated along Highway 17 between Dryden and Thunder Bay, Ontario. The existing bridge is a single span structure approximately 18.6 m long and 9.8 m wide, supported on rock-filled timber crib abutments. Each of the timber crib abutments is approximately 6.0 m high and 3.0 m by 12.8 m in plan.

The Sturgeon River meanders in a north-westerly direction out of Sturgeon Lake. The lands immediately surrounding the bridge site consist of forested areas.

Photographs in Appendix C show the general nature of the site and the existing bridge.

The site lies within the Wabigoon Subprovince of the Superior Province of the Canadian Shield. The area is underlain by granitic igneous and metamorphic rocks of the Early Precambrian, covered by a thin discontinuous layer of drift.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out between September 10 and 16, 2011, and comprised drilling and sampling of six boreholes, identified as Boreholes SRB-01 to SRB-06.

Boreholes SRB-02 to SRB-05 were drilled adjacent to the existing bridge abutments and terminated in bedrock at depths of 8.5 m to 11.4 m following recovery of approximately 3.0 m of rock core. Boreholes SRB-01 and SRB-06 were drilled at the bridge approaches: Borehole SRB-01 was terminated at 7.6 m depth, and Borehole SRB-06 was terminated upon auger refusal at 5.5 m depth. The approximate locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix G.

The borehole locations were marked in the field and utility clearances were obtained prior to drilling. The coordinates and ground surface elevations for the boreholes were estimated from topographic plans provided by WSP Canada Inc.

A truck mounted CME 75 drill rig was used to advance the boreholes using a combination of NW casing/ wash boring techniques and NQ rock coring equipment. Soil samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). All rock cores were logged and the Total Core Recovery (TCR), Rock Quality Designation (RQD) and Fracture Indices (FI) were determined.

The drilling and sampling operations were supervised on a full time basis by a member of Thurber's technical staff. The supervisor logged the boreholes and processed the recovered soil and rock samples for transporting to Thurber's laboratory for further examination and testing.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Groundwater conditions observed after completion of drilling may have been affected by water introduced into the boreholes during wash boring and coring operations. Standpipe piezometers were installed in two boreholes to monitor the groundwater level after drilling. The piezometers were subsequently decommissioned on October 28, 2012 and the boreholes without piezometers were backfilled in general accordance with MOE Regulation 903. Completion details of the piezometers and boreholes are summarized in Table 3.1.

Table 3.1 – Borehole Completion Details

Foundation Unit	Boreholes	Piezometer Tip Depth/ Elevation (m)	Completion Details
North Approach	SRB-01	None installed	Borehole backfilled with holeplug to 0.15 m then asphalt to surface.
North Abutment	SRB-02	None installed	Borehole backfilled with holeplug from 8.7 m to 0.15 m then asphalt to surface.
	SRB-03	6.1/ 401.7	Borehole backfilled with cuttings from 11.4 m to 6.1 m, filter sand from 6.1 m to 4.3 m, holeplug from 4.3 m to 0.30 m, concrete from 0.30 m to 0.15 m, then asphalt to surface.
South Abutment	SRB-04	None installed	Borehole backfilled with holeplug from 9.4 m to 0.30 m, concrete to 0.15 m, then asphalt to surface.
	SRB-05	7.0/ 401.3	Borehole backfilled with cuttings from 8.5 m to 7.0 m, filter sand from 7.0 m to 5.2 m, holeplug from 5.2 m to 0.30 m, concrete from 0.30 m to 0.15 m, then asphalt to surface.
South Approach	SRB-06	None installed	Borehole backfilled with holeplug from 5.5 m to 0.30 m, concrete to 0.15 m, then asphalt to surface.

4 LABORATORY TESTING

All recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. Selected samples were also subjected to grain size distribution analyses (sieve and hydrometer). The results of this testing program are summarized on the Record of Borehole sheets included in Appendix A and on the figures presented in Appendix B.

Bedrock core samples were subjected to geological logging. Point load tests were carried out on selected samples of intact bedrock in the laboratory to evaluate the unconfined compressive strength (UCS) of the bedrock. The UCS values of the rock assessed from the point load test data are reported on the borehole logs in Appendix A.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets included in Appendix A. Details of the encountered soil stratigraphy are presented in these sheets and on the "Borehole Locations and Soil Strata" drawing included in Appendix G. An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole sheets governs any interpretation of the site conditions. It must be recognized that soil conditions may vary between and beyond borehole locations.

The site stratigraphy typically comprises an asphalt surface overlying granular embankment fill, underlain by native sand and gravel deposits, locally sand and silt. Bedrock was encountered below the sand and gravel layer. More detailed descriptions of the individual strata are presented below.

5.1 Asphalt

Asphalt was encountered on the roadway surface in all boreholes drilled. The asphalt was 25 to 40 mm thick.

5.2 Embankment Fill

Cohesionless embankment fill typically consisting of sand with trace to some gravel and silt was encountered beneath the asphalt in all boreholes. Locally the fill graded to sandy gravel and gravelly sand. Cobbles and boulders were encountered in the fill, as indicated on the borehole logs. The base of the granular fill was encountered at depths of 4.3 to 4.6 m (Elev. 403.2 to 403.9).

SPT N-values recorded in the fill typically ranged from 11 to 31 blows/0.3 m penetration, indicating a generally compact condition. N-values of 8 and 9 blows/0.3 m were obtained in the lower portion of the fill in Boreholes SRB-04 and SRB-05, indicating a loose condition. N-values of 70 blows/0.3 m to 50 blows for no penetration were recorded on cobbles and boulders in the fill in five boreholes.

Moisture contents ranged between 6% and 20%.

Samples of fill underwent laboratory grain size analysis testing, the results of which are summarized below. These results are also presented on the Record of Borehole sheets included in Appendix A and on the grain size distribution curves shown on Figures B1 and B2 of Appendix B.

Soil Particles	Sand Fill	Sandy Gravel to Gravelly Sand Fill
Gravel %	2 to 17	36 to 70
Sand %	61 to 85	26 to 55
Silt & Clay %	3 to 22	4 to 9

5.3 Sand, Gravel, and Sand and Silt

Native cohesionless deposits were encountered below the embankment fill in all boreholes. These deposits comprised sand with some gravel and silt at the north abutment and south approach, gravel with trace to some sand at the south abutment, and sand and silt at the north approach. Occasional cobbles and locally boulders were encountered in the sand/gravel, notably above the underlying bedrock in Borehole SRB-03.

The thickness of the native deposits ranged between 0.9 and 4.1 m, with a lower boundary at depths of 5.5 to 8.4 m (Elev. 403.0 to 399.4). Borehole SRB-01 was terminated in the sand at 7.6 m depth (Elev. 400.2).

SPT N-values in the sand and gravel varied widely from 7 to 33 blows/0.3 m, indicating a loose to dense relative density. Several SPT N-values of 50 blows for less than 0.15 m of penetration were recorded, at the bedrock surface in Borehole SRB-02, on a boulder in Borehole SRB-03, and in very dense sand in Borehole SRB-06.

Moisture contents in the sand and gravel deposits ranged from 5% to 15%.

Samples of the deposit underwent laboratory grain size analysis testing, the results of which are summarized below. These results are also presented on the Record of Borehole sheets included in Appendix A, and on the grain size distribution curves shown on Figures B3 and B4 of Appendix B.

Soil Particles	**Sand to Sand and Silt**	**Gravel**
Gravel %	4 to 16	89 to 91
Sand %	51 to 65	8 to 10
Silt & Clay %	9 to 45	1

5.4 Bedrock

Bedrock was proven in Boreholes SRB-02 to SRB-05 by recovery of 2.5 to 3.0 m of bedrock core. Auger refusal on probable bedrock was encountered in Borehole SRB-06. The depth and elevations of the bedrock surface are summarized in Table 5.1.

The bedrock recovered in the core samples was described as white and black granodiorite with pink intrusions.

The Total Core Recovery (TCR) of all samples was 100%. The measured Rock Quality Designation (RQD) ranged from 88% to 100%, indicating good to typically excellent rock quality. The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m core, was 2 or less.

The unconfined compressive strength of the rock interpreted from point load tests conducted on the recovered cores ranged from 142 to 211 MPa, indicating a very strong rock.

Table 5.1 – Depths and Elevations of Bedrock

Borehole	Top of Bedrock	
	Depth (m)	Elevation
SRB-02	6.2	401.6
SRB-03	8.4	399.4
SRB-04	6.4	401.8
SRB-05	5.5	402.8
SRB-06	5.5	403.0*

* Probable bedrock, not proven by coring.

5.5 Water Levels

Where possible, water levels were monitored in the open boreholes during drilling operations. Wash boring and rock coring methods were used to advance the boreholes and therefore water levels recorded during or upon completion of drilling may not reflect natural groundwater levels. Standpipe piezometers were installed in two boreholes to monitor the groundwater level after completion. The water levels observed in the open boreholes upon completion and measured in the piezometers are summarized in Table 5.2.

Table 5.2 – Water Level Measurements

Borehole	Date	Water Level		Comment
		Depth (m)	Elev. (m)	
SRB-01	September 16, 2011	4.5	403.3	In open borehole
SRB-02	September 14, 2011	4.8	403.0	In open borehole
SRB-03	September 10, 2011	4.8	403.0	In open borehole
	September 16, 2011	4.2	403.6	In piezometer
	December 1, 2011	4.9	402.9	In piezometer
	October 28, 2012	4.3	403.5	In piezometer
SRB-04	September 10, 2011	4.3	403.9	In open borehole
SRB-05	September 11, 2011	4.5	403.8	In open borehole
	September 16, 2011	4.6	403.7	In piezometer
	December 1, 2011	4.9	403.4	In piezometer
	October 28, 2012	4.1	404.2	In piezometer
SRB-06	September 11, 2011	4.6	403.9	In open borehole

The above values are short-term readings and seasonal fluctuations of the groundwater and river level are to be expected. In particular, the water levels may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

The preliminary GA drawing provided by WSP Canada Inc indicates a water level at Elev. 404.3 in the Sturgeon River (undated) and a normal high water level at Elev. 404.6. Archive design drawings of the existing bridge indicate a water level at Elev. 402.6 in March 1960.

6 MISCELLANEOUS

Borehole locations were selected and established in the field by Thurber Engineering Ltd. The coordinates and the ground surface elevations for the boreholes were established based on topographic survey information provided by WSP Canada Inc.

Thurber obtained utility clearances for the borehole locations prior to drilling.

Eastern Ontario Diamond Drilling of Hawkesbury, Ontario supplied a truck mounted CME-75 drill rig and conducted the drilling, sampling and in-site testing operations. The drilling operations were supervised by Mr. George Azzopardi of Thurber.

Overall supervision of the field program was conducted by Mr. Mark Farrant P.Eng. Interpretation of the data and preparation of the report were carried out by Ms. Rocio Palomeque Reyna and Ms. Mei Cheong. P.Eng.

The report was reviewed by Mr. Murray Anderson, P.Eng. and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 GENERAL

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical recommendations for the design of a new bridge to replace the existing Sturgeon River bridge on Highway 599 in the District of Thunder Bay, Ontario.

The existing bridge is a single span structure supported on rock-filled timber crib abutments, which was originally constructed in 1961 and rehabilitated in 1980. The bridge is approximately 18.6 m long and 9.8 m wide. Each of the timber crib abutments is approximately 6.0 m high and 3.0 m by 12.8 m in plan, with design founding levels at Elev. 400.8 and 401.2 at the north and south abutments, respectively.

The preliminary General Arrangement drawing provided by WSP Canada Inc. indicates that the proposed replacement bridge will be a single span structure consisting of precast concrete box girders supported on steel pipe piles with a precast concrete header beam. The new bridge will be approximately 23.0 m long and 9.9 m wide. It is understood that each abutment will be carried by six piles and the load demand on each pile will be approximately 1380 kN at ULS and 935 kN at SLS.

In lieu of a conventional abutment, sheet piles will be installed immediately behind the pipe piles to retain the approach embankments. The road grade will be raised by 230 mm at the north abutment and 156 mm at the south abutment. Bridge replacement will be staged to maintain one lane of traffic throughout construction.

The discussions and recommendations presented in this report are based on the information provided by WSP Canada Inc and on the factual data obtained in the course of the investigation.

8 STRUCTURE FOUNDATIONS

The stratigraphy at the site typically consists of granular embankment fill extending to depths of 4.3 to 4.6 m, overlying cohesionless deposits of sand and gravel, which overlie granodiorite bedrock. The fill and native soils contain cobbles and boulders. The bedrock surface was encountered at depths of 6.2 and 8.4 m (Elev. 401.6 and 399.4) at the north abutment, and at depths of 6.4 and 5.5 m (Elev. 401.8 and 402.8) at the south abutment.

Piezometric readings indicate groundwater levels at depths varying between 4.1 and 4.3 m below existing road grade, at Elev. 402.9 to 404.2. The river water level indicated on the preliminary GA drawing is at Elev. 404.3 (undated), and the normal high water level is at Elev. 404.6.

Based on the subsurface conditions at the site, consideration was given to the following types of foundations to support the new bridge structure:

- Spread footings on native soils, engineered fill or bedrock
- Augered caissons (drilled shafts)
- Driven steel H-piles
- Drilled-in pipe piles

A comparison of the foundation alternatives based on advantages and disadvantages of each is included in Appendix D.

8.1 Spread Footings

Native sands and gravels were encountered at depths of 4.3 to 4.6 m on site, approximately 0.6 m above to 0.5 m below the water levels measured in the piezometers. Excavation for construction of footings on the native sands and gravels would extend through cohesionless soils close to or below the river level and may require installation of a sheet pile cofferdam to support the excavation sidewalls and control base boiling. Further, the geotechnical resistance in the native soils is highly variable, and sub-excavation of the native deposits to greater depths may be required to penetrate loose zones.

Supporting the structure on spread footings founded on engineered fill would also require excavation of the existing fill and loose native materials close to or below the river level. Cofferdam construction may be required to enable excavation and placement of the engineered fill to above the water level.

Bedrock was encountered at depths of 0.6 to 4.2 m below the water levels measured in the piezometers. Extending footings down through cohesionless soils below the water level to found on the bedrock is not considered to be practical.

Based on the above issues, the use of spread footings is not recommended at this site and design recommendations were not developed further.

8.2 Augered Caissons (Drilled Shafts)

Caissons supporting the structure at this site would extend through existing granular embankment fill and deposits of cohesionless sands and gravels containing cobbles and boulders, to found on/in bedrock. Construction of caissons through the cohesionless deposits below the groundwater table would require use of a permanent steel liner extended into bedrock to support the caisson sidewalls and provide a seal against groundwater inflow. Proper sealing of the liner in bedrock may be difficult to achieve. Further, boulders may interfere with caisson installation.

For these reasons, the use of caisson foundations is not recommended at this site.

8.3 Driven Steel H-Piles

The use of driven steel H-piles to support the proposed bridge is considered feasible provided the existing rock-filled cribs are removed prior to pile driving. It is recommended that the piles be driven to bedrock. The anticipated pile tip elevations and geotechnical resistance values recommended for HP 310x110 steel H-piles driven to refusal on bedrock are presented in Table 8.1. The anticipated pile lengths, based on the pile cut-off elevations shown on the preliminary design drawings, are also provided.

Table 8.1 – Recommended Pile Resistance Values for Driven H-Piles

Foundation Element	Boreholes	Estimated Pile Tip Elevation	Pile Cut-off Elevation	Anticipated Pile Length (m)
North Abutment	SRB-02	401.6	406.3	4.7
	SRB-03	399.4	406.3	6.9
South Abutment	SRB-04	401.8	406.7	4.9
	SRB-05	402.8	406.7	3.9

A factored geotechnical resistance at ULS of 2,000 kN per pile is recommended for design of HP 310x110 piles driven to refusal on bedrock. The geotechnical reaction at SLS will not govern design of piles on bedrock.

Since the bedrock surface is variable, the actual pile tip elevation and length of pile required may vary from those indicated in the table. An NSSP alerting the Contractor to the possibility of piles encountering refusal at varying depths is provided in Appendix E.

The existing rock-filled timber cribs should be completely removed prior to installation of the piles. It is noted that complete removal of the cribs may require excavation in cohesionless soils below the river water level (with associated stream protection/shoring/dewatering) and backfilling of the excavation. Further comments regarding excavation and dewatering are presented in Section 13.

Oversize materials (e.g. greater than 75 mm nominal diameter) must not be used in any fills through which the piles will be driven.

If the anticipated pile length to bedrock is not sufficient to provide lateral fixity, the piles will need to be socketed into the bedrock. In this case, pile installation should include advancing a socket at least 1.5 m below the bedrock surface, inserting the pile to the base of the socket, and then backfilling around the pile with concrete. For a HP 310x110 steel H-pile, a rock socket diameter of 610mm is required. The socket depth may need to be greater than 1.5 m to satisfy structural requirements such as lateral loads and maximum shear and moment demand on each pile.

Since the elevation of the bedrock surface is variable across the site and there is evidence of cobbles and boulders immediately above the bedrock, it is critical to determine in the field during inspection of rock socket installation that the entire depth of socket is formed in sound bedrock and not partly in cobbles and boulders and partly in bedrock. This issue is addressed in an NSSP included in Appendix E.

8.3.1 H-Pile Installation

Pile installation must be in accordance with OPSS 903.

The tips of all driven H-piles must be fitted with rock points from an approved manufacturer such as Titus Steel (Rock Injector for H-piles) or approved equivalent. Rock points are recommended for tip protection and to reduce the potential for slipping of the pile tip along the sloping bedrock surface while driving.

For piles installed to the tolerances shown in Clause 903.07.05.01 of the Specification, the foundation drawing should include the note “Piles to be driven to bedrock”.

If the proposed bridge design requires that the deviation at the top of the pile be limited to tight tolerances, a driving template or other means may be required to achieve the specified maximum deviation.

Cobbles and boulders were encountered within the existing embankment fill and underlying native soils. Rock fill is also present on the embankment slopes. The cobbles, boulders and rock fill may interfere with pile installation and some piles may meet refusal on boulders above the bedrock surface. The Contract Documents should contain a NSSP alerting Bidders to the presence of the cobbles, boulders and rock fill, and the need to remove, dislodge or otherwise penetrate these obstructions to advance the piles to bedrock while meeting the specified deflection tolerances. Suggested wording for an NSSP addressing these issues is included in Appendix E.

For rock socketed piles, the method of installation of the piles is the responsibility of the Contractor. The Contractor’s drilling method must be capable of dislodging, removing or penetrating obstructions such as cobbles and boulders in the overburden soils. Care must be exercised while drilling the socket within the bedrock; the drilling methodology must be capable of excavating the bedrock to the specified socket dimensions without disturbing or fracturing the bedrock forming the sidewalls and base of the socket. Blasting to facilitate rock removal is not permitted.

The drilling method must also maintain sidewall stability of the drilled hole and allow cleaning of the socket without cohesionless soils running into the socket. During and subsequent to installation, the drilled hole and socket will be partially filled with water and it may not be practical to dewater the socket prior to concreting. Tremie concreting will be required for concreting these piles. A NSSP addressing these issues is included in Appendix E.

8.3.2 Lateral Resistance for Piles

The geotechnical lateral resistance acting on a pile in cohesionless soils may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

$$k_s = n_h \; z / D \qquad (\mathrm{kN/m^3})$$

$$p_{ult} = 3 \; \gamma \; z \; K_p \qquad (\mathrm{kPa})$$

where z = depth of embedment of pile in metres
D = pile width in metres
n_h = coefficient related to soil density
γ = unit weight, $\mathrm{kN/m^3}$
K_p = passive earth pressure coefficient

The parameters recommended for use with the above equations are provided in Table 8.2.

The above equations and recommended parameters may be used to analyze the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance.

The spring constant, K, for analysis may be obtained by the expression, $K = k_s \; L \; D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction ($\mathrm{kN/m^3}$), D is the pile width (m) and L is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance on any one segment of pile, P_{ult}, may be obtained from the expression, $P_{ult} = p_{ult} \; L \; D$. This represents the ultimate load at which the pile fails and will not support any additional load at greater displacements. It is recommended however that the total lateral resistance by one pile be limited to 120 kN at factored ULS and 50 kN at SLS.

Table 8.2 – Parameters for Lateral Pile Resistance

Abutment	Elevation	n_h (kN/m^3)	K_p	Unit Weight (kN/m^3)	Soil Conditions
North	406.3 to 405.5	3,500[1]	2.0[1]	21	Compact sand fill
	405.5 to 404.5	5,000	3.3	21	Compact sand fill
	404.5 to 403.5	4,000	3.3	11[2]	Compact sand fill
	403.5 to bedrock	5,000	3.5	11[2]	Compact to dense sand
South	406.7 to 405.5	3,500[1]	2.0[1]	21	Compact sand fill
	405.5 to 404.5	3,000	3.0	21	Loose sand fill
	404.5 to 403.7	2,500	3.0	11[2]	Loose sand fill
	403.7 to bedrock	4,000	3.7	11[2]	Loose to compact gravel

1 Values reduced due to sloping ground.
2 Buoyant unit weight below the water level.

For lateral soil/pile group interaction analysis, the modulus of subgrade reaction (k_s) in the soil may have to be reduced based on pile spacing. Where a pile group is oriented ***perpendicular*** to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Pile Spacing Perpendicular to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
4 D*	1.00
1 D*	0.50

* D is the width of the pile, and spacing is measured centre to centre

Where a pile group is oriented ***parallel*** to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Pile Spacing Parallel to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
8 D	1.00
6 D	0.70
4 D	0.40
3 D	0.25

Intermediate values may be obtained by interpolation.

8.4 Drilled-in Pipe Piles

The replacement bridge may be supported on drilled-in steel pipe piles socketed into bedrock and filled with concrete. It is recommended that the piles be advanced a minimum 1.0 m into bedrock to confirm that any fractured rock and cobbles/ boulders at the bedrock surface are penetrated and to fix the pile tip in place. Due to the presence of cobbles, boulders and rock fill in the embankment material, driven pipe piles are not recommended.

The capacity of concrete-filled pipe piles socketed into bedrock will be dictated by the structural resistance of the composite pile section and will not be governed by the geotechnical resistance of the bedrock. The factored geotechnical resistance recommended for selected pipe pile sections are presented in Table 8.3.

Table 8.3 – Factored Geotechnical Resistance of Drilled-in Pipe Piles

Pipe Pile Section		Factored Geotechnical Resistance (kN)
Outer Diameter (mm)	**Wall Thickness (mm)**	
324	12.7	2,000
406	12.7	2,800
508	12.7	4,000
610	12.7	5,500

The resistance values presented above assume a steel yield strength of 245 MPa and a concrete compressive strength of 35 MPa. The resistances have been reduced to account for the possibility that residual crushed rock may remain in the rock socket. The depth of the socket may need to be greater than 1.0 m to address the lateral resistance requirement, base fixity requirement and shear and moment demand for each pile.

8.4.1 Lateral Socket Resistance

The ultimate passive force that can be mobilized by the embedded portion of a pipe pile socket within rock is constant with depth and is given by:

$$P_p = 6\ c\ D\ L$$

Where c = 2,000 kPa (equivalent Mohr-Coulomb cohesion based on Hoek and Brown rock mass classification

L = Depth of socket in rock, m

D = Socket diameter, m

Lateral resistance of the section of pipe pile located above the bedrock surface may be computed using the parameters in Section 8.3.2.

8.4.2 Drilled-in Pipe Pile Installation

Installation of pipe piles must follow OPSS 903 specifications.

The method of installation of the pipe piles is the responsibility of the Contractor. One option for installing pipe piles is to drill them in using a concentric drilling method such as the Symmetrix system. The Contractor’s drilling method must be capable of dislodging, removing or penetrating obstructions such as cobbles, boulders or rock fill, the existing timber cribs, and overburden soils. Care must be exercised while drilling into the bedrock; the drilling methodology must be capable of advancing the pile without disturbing or fracturing the bedrock at the base of the pile. Blasting to facilitate rock removal is not permitted.

Since the rock cutting shoe at the tip of a pipe pile will be slightly larger in diameter than the outside diameter of a pipe pile, there will be a small gap between the rock socket wall and the pipe pile. It is recommended that the annular space between the pipe pile and socket wall be grouted to the bedrock surface to achieve fixity.

During and subsequent to installation, the pipe pile will be partially filled with water and it may not be practical to dewater the pipe prior to concreting. Tremie concreting will be required for concreting these pipe piles.

A NSSP addressing the above issues is included in Appendix E.

8.5 Downdrag

Downdrag on piles is not considered to be an issue at this site.

8.6 Recommended Foundation

From a geotechnical perspective and based on the subsurface conditions, steel pipe piles drilled into the bedrock are the recommended foundation option for supporting the proposed bridge structure. Consideration may also be given to H-piles driven to bedrock provided the existing rock-filled cribs are completely removed or the piles are located behind the cribs. Socketing of the H-piles into bedrock may be required to satisfy structural requirements.

8.7 Frost Cover

The design depth of frost penetration at this site is 2.6 m. The base of all buried pile caps, if employed, must be provided with a minimum of 2.6 m of earth cover as protection against frost action.

9 SHEET PILE WALLS

Installation of steel sheet pile walls adjacent to the pile foundations is proposed in lieu of conventional abutment walls. The sheet piles will provide containment and resistance to lateral earth pressures from the approach fill.

The alignment of the proposed sheet pile walls should be carefully selected to avoid the existing abutment cribs. Alternatively, the cribs should be completely removed prior to installation of the piles. Oversize materials (e.g. greater than 75 mm nominal diameter) must not be permitted in fill used to backfill the crib excavations through which the piles will be driven.

Driving of the sheet piles through the existing approach fill and native soils may encounter cobbles, boulders or zones of rock fill. The Contract Documents should contain a NSSP alerting the Bidders to the possibility of some sheet piles meeting refusal on the cobbles, boulders or rock fill, and the need to remove or penetrate these obstructions. Suggested text for the NSSP is included in Appendix E. Any visible obstructions such as boulders and rock protection along the sides of the embankment should be removed prior to driving the sheet piles.

Sheet piles should be provided with sheet pile tip protection to minimize any tip damage.

Design of the permanent sheet pile walls must consider environmental conditions such as road salts or fluctuating water levels that may cause corrosion and reduce the service life of the structure. The ground in front of the sheet pile should be properly compacted and re-graded following removal of the existing timber cribs, and protected from river erosion so that the sheet piles do not lose lateral support.

Backfill behind the sheet pile walls should be in accordance with OPSS 902 and should consist of Granular A, Granular B Type II or Granular B Type III material. All granular material should meet the specifications of OPSS.PROV 1010. Compaction equipment to be used adjacent to sheet pile walls should be restricted in accordance with OPSS 501.

Lateral earth pressures acting on the sheet pile walls may be assumed to be distributed triangularly and to be governed by the characteristics of the backfill behind the sheet piles and the underlying soils. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

$$p_h = K(\gamma h + q)$$

Where: p_h = horizontal pressure on the wall at depth h (kPa)

K = earth pressure coefficient (see Table 9.1)

γ = unit weight of retained soil (see Table 9.1)

h = depth below top of fill where pressure is computed (m)

q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the sheet piles are dependent on the material used as backfill. Typical values are shown in Table 9.1.

Table 9.1 – Earth Pressure Coefficients (K)

Condition	**Earth Pressure Coefficient (K)**			
	OPSS Granular A or OPSS Granular B Type II **$\phi = 35°$, $\gamma = 22.8$ kN/m^3**		**OPSS Granular B Type I, Granular B Type III, or Existing Sand and Gravel Fill** **$\phi = 32°$, $\gamma = 21.2$ kN/m^3**	
	Horizontal Surface Behind Wall	**Sloping Surface Behind Wall (2H:1V)**	**Horizontal Surface Behind Wall**	**Sloping Surface Behind Wall (2H:1V)**
Active (Unrestrained Wall)	0.27	0.38*	0.31	0.46*
At rest (Restrained Wall)	0.43	-	0.47	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-

* For wing walls.

The use of a material with a high friction angle and low active pressure coefficient (Granular A, Granular B Type II) is preferred as it results in lower earth pressures acting on the wall.

The factors in Table 9.1 are "ultimate" values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.16 in the Commentary to the Canadian Highway Bridge Design Code.

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I and Type III or 1.7 m for Granular A or Granular B Type II.

10 SEISMIC CONDITIONS

The following seismic parameters should be used for design:

Velocity Related Seismic Zone	0
Zonal Velocity Ratio	0.0
Acceleration Related Seismic Zone	0
Zonal Acceleration Ratio	0.0
Peak Horizontal Acceleration	0.011g

The soil profile type at this site has been classified as Type I. Therefore, according to Clause 4.4.6.1 Table 4.4 of the CHBDC, a Site Coefficient "S" (ground motion amplification factor) of 1.0 should be used in seismic design.

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading. The coefficients of horizontal earth pressure for seismic loading presented in Table 10.1 may be used:

Table 10.1 – Earth Pressure Coefficients for Earthquake Loading

Condition	**Earth Pressure Coefficient (K)**	
	OPSS Granular A or OPSS Granular B Type II $\phi = 35°, \gamma = 22.8$ **kN/m³**	**OPSS Granular B Type I, Granular B Type III, or Existing sand and gravel fill** $\phi = 32°, \gamma = 21.2$ **kN/m³**
Active (K_{AE})*	0.28	0.32
Passive (K_{PE})	3.7	3.2
At Rest (K_{OE})**	0.45	0.50

* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.

** After Woods

The potential for liquefaction of the foundation soils was assessed using the Seed and Idriss (1971) method for cohesionless soils. Using the method, it is estimated that under the existing conditions, the foundation soils at the abutments are not prone to liquefaction.

11 APPROACH EMBANKMENTS

The existing approach embankments are approximately 4.5 m high above the riverbed. The existing side slopes are inclined at about 1.5H:1V to 2H:1V, are covered by rock fill, and generally appear to be performing well.

The proposed bridge replacement works will include a grade raise up to 230 mm, cutting the forward slopes in front of the abutments to an inclination no steeper than 2H:1V, and provision of rock protection.

Embankment construction should be carried out in accordance with OPSS 206. It is recommended that any additional fill placed on the existing sloped embankment surfaces to accommodate the minor grade raise consist of rock fill. Existing vegetation should be removed from the embankment slopes prior to placement of new rock fill.

The foundation soils governing settlement and stability of the approach embankments generally consist of compact to very dense native sands and gravels. Considering the foundation conditions and the performance of the existing embankment slopes, stability of the embankments at the current inclinations is not a concern provided erosion protection is maintained.

A slope stability analysis was conducted to assess the stability of the forward slopes with the proposed sheet pile wall abutment configuration. The analyses were carried out using the Morgenstern-Price method of slope stability analysis. The geotechnical model and results of the stability analyses are shown on Figures F1 and F2 of Appendix F. The results of the analyses indicate that adequate factors of safety exceeding 1.3 are achieved if the sheet piles are driven to or below Elevation 401.6 at the north abutment and Elevation 403.5 at the south abutment. Deeper penetration may be required to resist earth pressures.

The proposed 230 mm grade raise will induce immediate (elastic) settlement in the existing granular embankment fill and the underlying native sand and gravel deposits. The settlement under the proposed grade raise is expected to be in the order of 5 to 10 mm and occur essentially as the fill is placed.

12 EROSION PROTECTION

Erosion protection measures such as rock protection must be maintained and/or enhanced along any surfaces that may be in contact with river flow.

A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion, in general accordance with OPSS 804.

13 EXCAVATION AND GROUNDWATER CONTROL

All excavation must be carried out in accordance with OPSS 902 and the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the existing embankment fill and native soils within the probable depth of excavation at this site may be classed as Type 3 soils above the water level and Type 4 soils below the water level.

Any excavation below the river or groundwater level without prior dewatering is not recommended since the inflow of groundwater will make it difficult to maintain a dry, sound base on which to work. If complete removal of the existing crib abutments is planned (ie., to accommodate driving of new H-piles or sheet piles), a shoring and dewatering system may be required depending upon the river level at the time of construction. This may include installation of a sheet pile wall cofferdam extended down to bedrock to support the sidewalls and minimize the potential for upwelling of the excavation base. Sealing of the sheet pile cofferdam at the bedrock surface may be problematic however due to the sloping bedrock surface and cobbles/boulders in the overlying native soils. Groundwater inflow, base instability and loss of soil may remain an issue, and therefore excavation below the water level is not recommended at this site.

The design of any dewatering and roadway protection systems that may be required is the responsibility of the Contractor. All shoring systems should be designed by a Professional Engineer experienced in such designs, who will determine an appropriate support system.

Bridge replacement will be carried out in stages to maintain one traffic lane operational at all times. Roadway protection will be required to facilitate staging. Roadway protection should be provided in accordance with OPSS 539 and designed for Performance Level 2.

14 CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to:

- The existing rock-filled timber cribs should be completely removed prior to driving of piles.
- If the existing timber cribs are to be completely removed, excavation in cohesionless soils and backfilling of the excavation below the river water level (with shoring and dewatering) may be required depending upon the river level at the time of construction. Installation and sealing of a sheet pile cofferdam at the bedrock surface may be problematic however due to the sloping bedrock surface and cobbles/boulders in the overlying native soils.
- Cobbles and boulders were encountered within the existing embankment fill and underlying native soils. Provision must be made to remove, dislodge or otherwise penetrate these obstructions to advance the foundation piles to bedrock while meeting the specified deflection tolerance, and to install sheet piles.
- If driven H-piles are employed, rock points are recommended to reduce the potential for slipping of the pile tip along the bedrock surface while driving.
- Since the bedrock surface is variable, the actual pile tip elevation and length of pile required may vary.

15 CLOSURE

Engineering analysis and preparation of the report were carried out by Ms. Mei Cheong, P.Eng.

The report was reviewed by Mr. Murray R. Anderson, P.Eng. and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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Sept 15/14
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Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTUAL CLASSIFICATION OF SOILS

CLASSIFICATION	PARTICLE SIZE	VISUAL IDENTIFICATION
Boulders	Greater than 200mm	same
Cobbles	75 to 200mm	same
Gravel	4.75 to 75mm	5 to 75mm
Sand	0.075 to 4.75mm	Not visible particles to 5mm
Silt	0.002 to 0.075mm	Non-plastic particles, not visible to the naked eye
Clay	Less than 0.002mm	Plastic particles, not visible to the naked eye

2. COARSE GRAIN SOIL DESCRIPTION (50% greater than 0.075mm)

TERMINOLOGY	PROPORTION
Trace or Occasional	Less than 10%
Some	10 to 20%
Adjective (e.g. silty or sandy)	20 to 35%
And (e.g. sand and gravel)	35 to 50%

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

DESCRIPTIVE TERM	UNDRAINED SHEAR STRENGTH (kPa)	APPROXIMATE SPT(1) ‘N’ VALUE
Very Soft	12 or less	Less than 2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	Greater than 200	Greater than 30

NOTE: Hierarchy of Soil Strength Prediction

1) Laboratory Triaxial Testing
2) Field Insitu Vane Testing
3) Laboratory Vane Testing
4) SPT value
5) Pocket Penetrometer

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

DESCRIPTIVE TERM	SPT “N” VALUE
Very Loose	Less than 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	Greater than 50

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE	SS Split Spoon Sample	WS Wash Sample	AS Auger (Grab) Sample
	TW Thin Wall Shelby Tube Sample	TP Thin Wall Piston Sample	
	PH Sampler Advanced by Hydraulic Pressure	PM Sampler Advanced by Manual Pressure	
	WH Sampler Advanced by Self Static Weight	RC Rock Core	SC Soil Core

$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$

▼	Water Level
C_{pen}	Shear Strength Determination by Pocket Penetrometer

(1) SPT ‘N’ Value Standard Penetration Test ‘N’ Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.

(2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to “A” size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

UNIFIED SOILS CLASSIFICATION

MAJOR DIVISIONS		GROUP SYMBOL	TYPICAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	GW	Well-graded gravels or gravel-sand mixtures, little or no fines.
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines.
		GM	Silty gravels, gravel-sand-silt mixtures.
		GC	Clayey gravels, gravel-sand-clay mixtures.
	SAND AND SANDY SOILS	SW	Well-graded sands or gravelly sands, little or no fines.
		SP	Poorly-graded sands or gravelly sands, little or no fines.
		SM	Silty sands, sand-silt mixtures.
		SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS	SILTS AND CLAYS $W_L < 50\%$	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays. ($W_L < 30\%$).
		CI	Inorganic clays of medium plasticity, silty clays. ($30\% < W_L < 50\%$).
		OL	Organic silts and organic silty-clays of low plasticity.
	SILTS AND CLAYS $W_L > 50\%$	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
		CH	Inorganic clays of high plasticity, fat clays.
		OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.
CLAY SHALE			
SANDSTONE			
SILTSTONE			
CLAYSTONE			
COAL			

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

Fresh (FR)	No visible signs of weathering.
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.

DISCONTINUITY SPACING

Bedding	**Bedding Plane Spacing**
Very thickly bedded	Greater than 2m
Thickly bedded	0.6 to 2m
Medium bedded	0.2 to 0.6m
Thinly bedded	60mm to 0.2m
Very thinly bedded	20 to 60mm
Laminated	6 to 20mm
Thinly Laminated	Less than 6mm

SYMBOLS

▨	CLAYSTONE
┅	SILTSTONE
░	SANDSTONE
■	COAL
▩	BEDROCK

STRENGTH CLASSIFICATION

Rock Strength	**Approximate Uniaxial Compressive Strength**		**Field Estimation of Hardness***
	(MPa)	**(psi)**	
Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

TERMS

Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length
Solid Core Recovery:(SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run
Rock Quality Designation:(RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a % of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index:(FI)	Frequency of natural fractures per 0.3m of core run.

Ministry of Transportation
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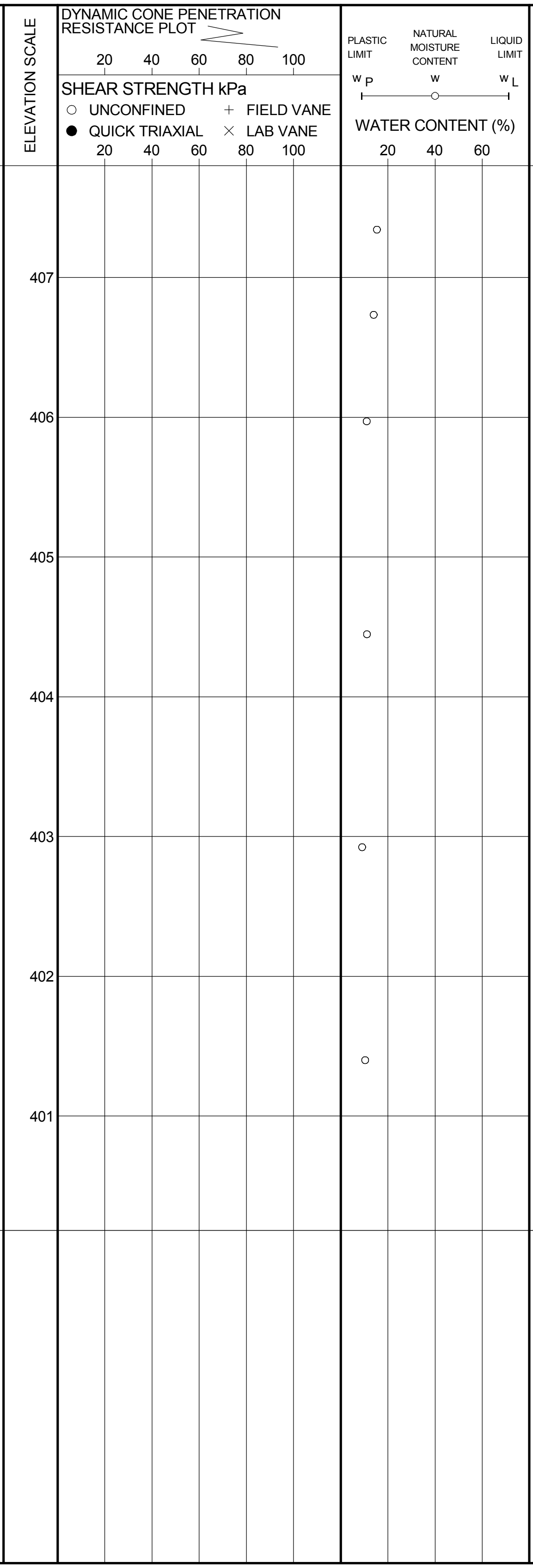
RECORD OF BOREHOLE No SRB-01

1 OF 1

METRIC

W.P.	6082-09-01	LOCATION	Sturgeon River Bridge N 5 549 165.5 E 653 619.4	ORIGINATED BY	GA
HWY	599	BOREHOLE TYPE	Casing/Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2011.09.16 - 2011.09.16	CHECKED BY	RPR

SOIL PROFILE		SAMPLES				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	γ kN/m³	GR	SA	SI	CL
407.8										
0.0	**ASPHALT:** (40mm)									
	SAND, some gravel, trace silt Compact to Very Dense Brown Wet (FILL)	1	SS	28						
	Occasional cobbles	2	SS	25			15	80	5 (SI+CL)	
		3	SS	70						
	Boulder (300mm) at 2.3m	4	SS	50/ 0.00						
	Boulder (275mm) at 3.1m	5	SS	50/ 0.075						
403.4 4.4	**SAND** and **SILT**, trace clay, trace gravel Dense to Compact Grey Wet				▽					
		6	SS	33			4	51	36	9
		7	SS	22						
400.2 7.6	END OF BOREHOLE AT 7.6m. BOREHOLE OPEN TO 7.6m, WATER OBSERVED AT 4.5m UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH HOLEPLUG FROM 7.6m TO 0.15m, THEN ASPHALT TO SURFACE.									



+³, ×³: Numbers refer to Sensitivity

20
15 ⊕ 5
10
(%) STRAIN AT FAILURE

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THURBER

RECORD OF BOREHOLE No SRB-02

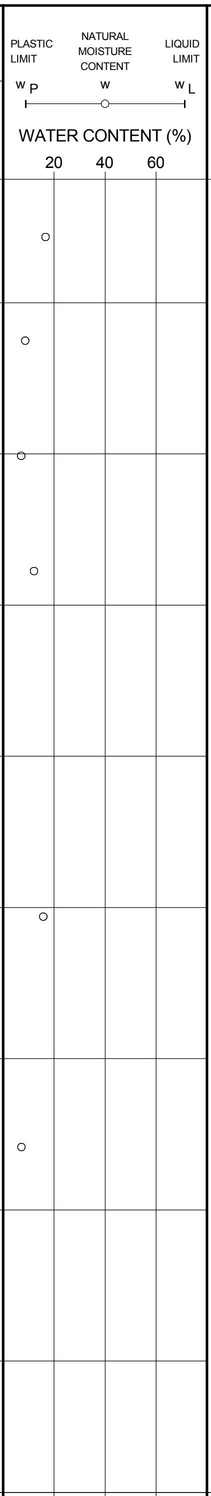
1 OF 1

METRIC

W.P.	6082-09-01	LOCATION	Sturgeon River Bridge N 5 549 160.3 E 653 612.7	ORIGINATED BY	GA
HWY	599	BOREHOLE TYPE	Coring/NW Casing	COMPILED BY	AN
DATUM	Geodetic	DATE	2011.09.14 - 2011.09.14	CHECKED BY	RPR

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	FI	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
407.8 0.0	**ASPHALT:** (40mm)							
	SAND, trace gravel, trace silt, occasional cobbles Compact Brown Moist (FILL)	1	SS	24				
						407		
		2	SS	18				10 84 6 (SI+CL)
		3	SS	19		406		
		4	SS	12				
						405		
	Boulders from 2.9m to 3.7m	5	SS	50/ 0.00				
						404		
403.2 4.6	**SAND**, some gravel, some silt Dense Grey Wet	6	SS	30	∇	403		16 65 19 (SI+CL)
						402		
401.6 6.2	**BEDROCK**, granodiorite, white and black with pink intrusions	7	SS	50/ 0.075			FI 0 0	RUN #1 TCR=100% SCR=100% RQD=100% UCS=177MPa (Average)
		1	RUN			401	0 0 0	
	Horizontal joints at 8.1m, 8.3m and 8.6m	2	RUN			400	0 0	RUN #2 TCR=100% SCR=100% RQD=100% UCS=164MPa (Average)
399.1		3	RUN				1 1	RUN #3 TCR=100% SCR=100% RQD=100% UCS=145MPa (Average)
8.7	END OF BOREHOLE AT 8.7m. BOREHOLE OPEN TO 8.7m, WATER OBSERVED AT 4.8m UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH HOLEPLUG FROM 8.7m TO 0.15m, THEN ASPHALT TO SURFACE.							

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB VANE
20 40 60 80 100



UNIT WEIGHT γ kN/m 3

+ 3, × 3: Numbers refer to Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

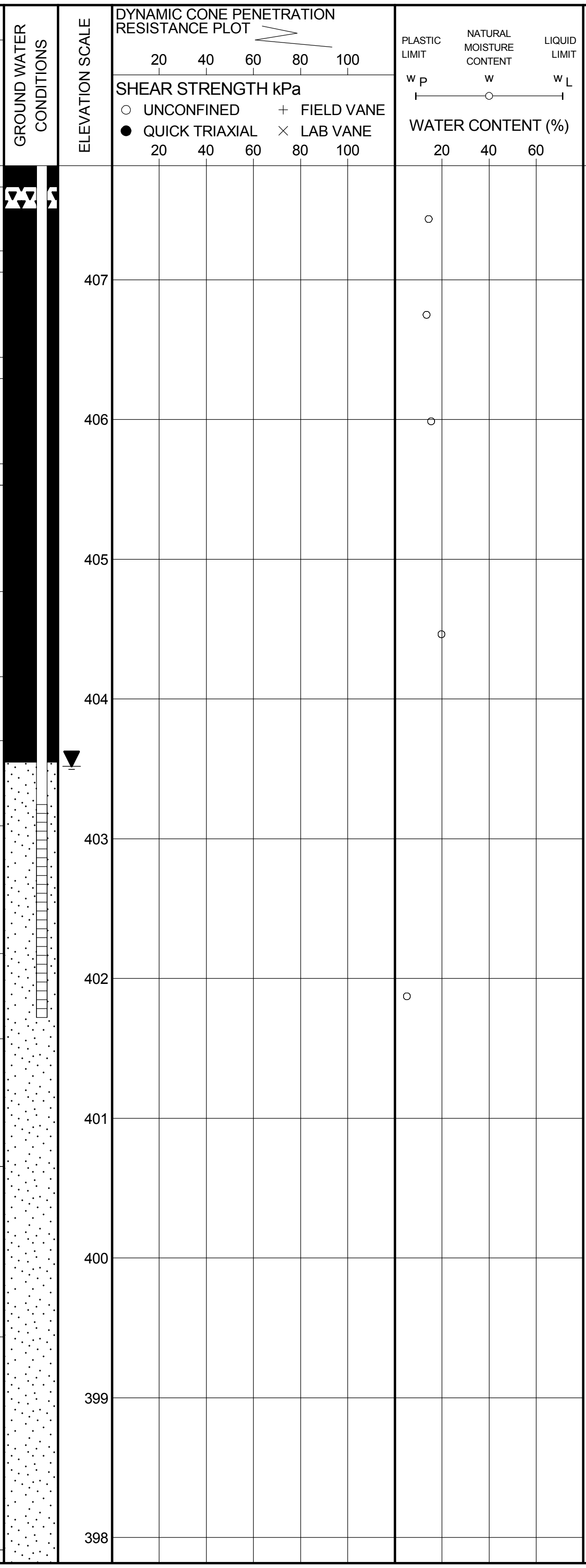
RECORD OF BOREHOLE No SRB-03

1 OF 2

METRIC

W.P. 6082-09-01
LOCATION Sturgeon River Bridge N 5 549 157.2 E 653 617.3
ORIGINATED BY GA
HWY 599
BOREHOLE TYPE NW Casing/NQ Coring
COMPILED BY AN
DATUM Geodetic
DATE 2011.09.10 - 2011.09.10
CHECKED BY RPR

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m 3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
407.8						
0.0 0.0	**ASPHALT:** (40mm)					
	SAND, some gravel to gravelly, trace silt Compact to Very Dense Brown Wet (FILL) Occasional cobbles	1	SS	31		13 84 3 (SI+CL)
		2	SS	27		
		3	SS	15		
	Boulder (200mm) at 2.3m	~~4~~	~~SS~~	~~50/~~ 0.00		
	Boulder (300mm) at 2.6m					
		5	SS	17		36 55 9 (SI+CL)
403.5 4.3	**SAND**, some gravel, some silt Compact Brown Wet	6	SS	11		
	Grey	7	SS	19		
	Boulder (225mm) at 7.2m	~~8~~	~~SS~~	~~50/~~ 0.00		
	Cobbles from 7.6m to 8.2m				FI	
399.4 8.4	**BEDROCK**, granodiorite, white and black with pink intrusions				0	RUN #1 TCR=100% SCR=100% RQD=93% UCS=142MPa (Average)
		1	RUN		2	
					1	
	Vertical joint from 8.6m to 8.9m				0	
	Sub-vertical joint at 9.1m				0	



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+ 3 , × 3 : Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No SRB-03

2 OF 2

METRIC

W.P.	6082-09-01	LOCATION	Sturgeon River Bridge N 5 549 157.2 E 653 617.3	ORIGINATED BY	GA
HWY	599	BOREHOLE TYPE	NW Casing/NQ Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2011.09.10 - 2011.09.10	CHECKED BY	RPR

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
	Continued From Previous Page										
	BEDROCK, granodiorite, white and black with pink intrusions		2	RUN			397			0 0 0 0 0	RUN #2 TCR=100% SCR=100% RQD=100% UCS=195MPa (Average)
396.4 11.4	END OF BOREHOLE AT 11.4m. BOREHOLE OPEN TO 11.4m AND WATER LEVEL AT 4.8m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Sep.16/11 4.2 403.6 Dec.01/11 4.9 402.9 Oct.28/12 4.3 403.5										

+ 3 , × 3 : Numbers refer to Sensitivity

20
15 5
10
(%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No SRB-04

1 OF 2

METRIC

W.P.	6082-09-01	LOCATION	Sturgeon River Bridge N 5 549 138.1 E 653 604.4	ORIGINATED BY	GA
HWY	599	BOREHOLE TYPE	NW Casing/NQ Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2011.09.10 - 2011.09.10	CHECKED BY	RPR

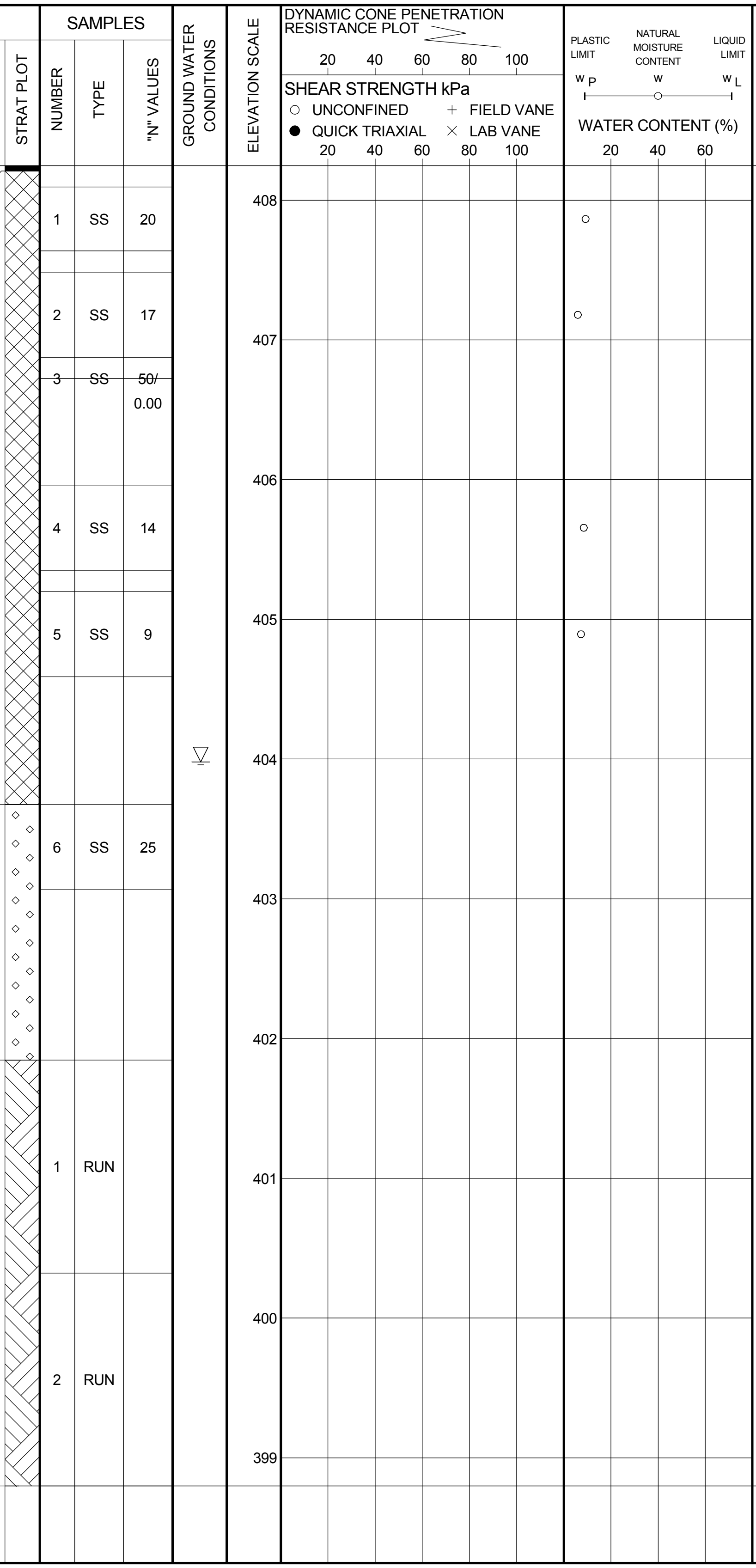
SOIL PROFILE

ELEV DEPTH	DESCRIPTION
408.2 0.0	**ASPHALT:** (40mm)
0.0	Sandy **GRAVEL** to Gravelly **SAND**, trace silt Compact Brown Wet (FILL) Occasional cobbles Boulder (200mm) at 1.5m
403.7 4.6	**GRAVEL**, trace to some sand, trace silt Compact Brown to Grey Wet Occasional cobbles
401.8 6.4	**BEDROCK**, granodiorite, white and black Horizontal joint at 6.4m Sub-horizontal joint at 9.1m Horizontal joint at 9.3m
398.8 9.4	END OF BOREHOLE AT 9.4m. BOREHOLE OPEN TO 9.4m AND WATER LEVEL AT 4.3m.

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	20	
2	SS	17	70 26 4 (SI+CL)
3	SS	50/ 0.00	
4	SS	14	
5	SS	9	
6	SS	25	89 10 1 (SI+CL)
1	RUN		RUN #1 TCR=100% SCR=100% RQD=90% UCS=172MPa (Average)
2	RUN		RUN #2 TCR=100% SCR=100% RQD=88% UCS=191MPa (Average)

FI: 2, 0, 0, 0, 0, 0, 0, 0, 1, 2



UNIT WEIGHT γ kN/m³

Continued Next Page

+ ³, × ³ : Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No SRB-04

2 OF 2

METRIC

W.P. 6082-09-01
LOCATION Sturgeon River Bridge N 5 549 138.1 E 653 604.4
ORIGINATED BY GA
HWY 599
BOREHOLE TYPE NW Casing/NQ Coring
COMPILED BY AN
DATUM Geodetic
DATE 2011.09.10 - 2011.09.10
CHECKED BY RPR

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m 3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION Continued From Previous Page	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
	BOREHOLE BACKFILLED WITH HOLEPLUG FROM 9.4m TO 0.3m, CONCRETE FROM 0.3m TO 0.15m, THEN ASPHALT TO SURFACE.										

ONTMT4S 0840.GPJ 2012TEMPLATE(MTO).GDT 7/31/14

+ 3 , × 3 : Numbers refer to Sensitivity

20
15 ⊕ 5
10
(%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

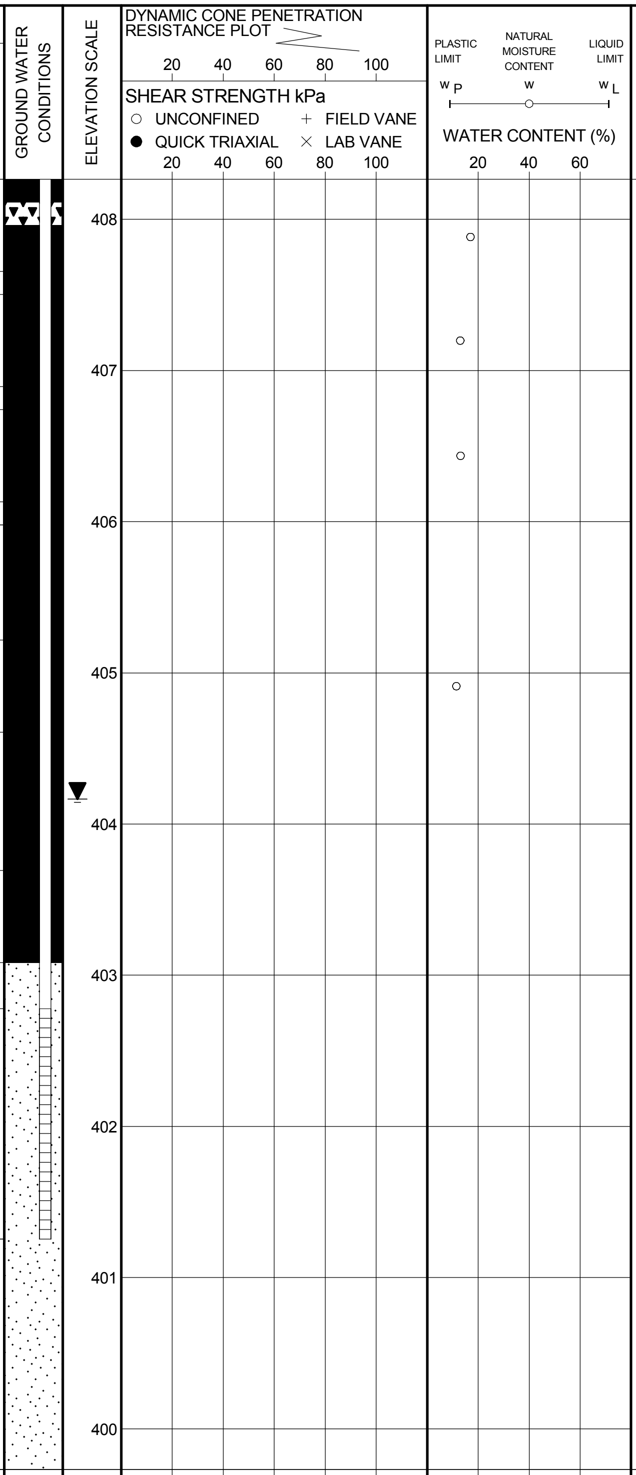
RECORD OF BOREHOLE No SRB-05

1 OF 2

METRIC

W.P. 6082-09-01
LOCATION Sturgeon River Bridge N 5 549 141.2 E 653 599.8
ORIGINATED BY GA
HWY 599
BOREHOLE TYPE NW Casing/NQ Coring
COMPILED BY AN
DATUM Geodetic
DATE 2011.09.11 - 2011.09.11
CHECKED BY RPR

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	SAMPLES: NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
408.3						
0.0	**ASPHALT:** (40mm)					
0.0	**SAND**, some silt, trace to some gravel Compact Brown Wet (FILL)	1	SS	20		
		2	SS	23		2 85 13 (SI+CL)
		3	SS	18		
	Occasional cobbles	~~4~~	~~SS~~	~~50/~~ 0.00		
	Gravelly	5	SS	8		
403.7 4.6	**GRAVEL**, trace to some sand Loose Brown to Grey Wet	6	SS	7		91 8 1 (SI+CL)
402.8 5.5	**BEDROCK**, granodiorite, white and black Horizontal joint at 5.5m	1	RUN		FI 1 0 0 0 0	RUN #1 TCR=100% SCR=100% RQD=97% UCS=169MPa (Average)
	Horizontal joint at 7.2m Sub-horizontal joint at 7.4m and 7.7m	2	RUN		1 1 1 0 0	RUN #2 TCR=100% SCR=100% RQD=100% UCS=211MPa (Average)
399.7 8.5	END OF BOREHOLE AT 8.5m. BOREHOLE OPEN TO 8.5m AND WATER LEVEL AT 4.5m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.					



WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEV. (m)
Sep.16/11	4.6	403.7

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SRB-05 2 OF 2 METRIC

W.P.	6082-09-01	LOCATION	Sturgeon River Bridge N 5 549 141.2 E 653 599.8	ORIGINATED BY	GA
HWY	599	BOREHOLE TYPE	NW Casing/NQ Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2011.09.11 - 2011.09.11	CHECKED BY	RPR

SOIL PROFILE			SAMPLES								
ELEV DEPTH	DESCRIPTION Continued From Previous Page	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w L WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m 3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Dec.01/11 4.9 403.4 Oct.28/12 4.1 404.2										

+ 3 , × 3 : Numbers refer to Sensitivity

20
15 ◐ 5 (%) STRAIN AT FAILURE
10

Ministry of Transportation
Ontario

THURBER

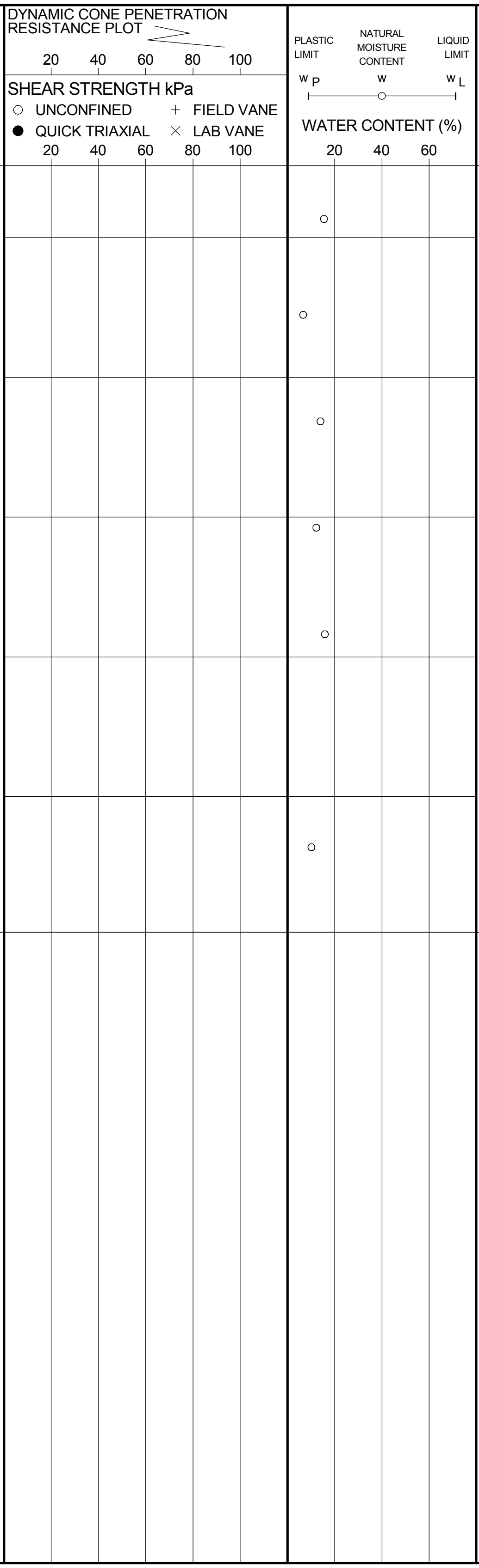
RECORD OF BOREHOLE No SRB-06

1 OF 1

METRIC

W.P. 6082-09-01
LOCATION Sturgeon River Bridge N 5 549 132.9 E 653 597.8
ORIGINATED BY GA
HWY 599
BOREHOLE TYPE NW Casing
COMPILED BY AN
DATUM Geodetic
DATE 2011.09.11 - 2011.09.11
CHECKED BY RPR

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	GR	SA	SI	CL
408.5									
0.0	**ASPHALT:** (25mm)								
0.0	**SAND**, some gravel, some silt Compact Brown Damp (FILL)	1	SS	23					
		2	SS	18					
	Wet	3	SS	16		17	61	22 (SI+CL)	
		4	SS	17					
		5	SS	11					
403.9 4.6	**SAND**, some silt Very Dense Brown Wet	6	SS	50/ 0.150	▽				
403.0 5.5	END OF BOREHOLE AT 5.5m UPON REFUSAL ON PROBABLE BEDROCK. BOREHOLE OPEN TO 5.5m AND WATER LEVEL AT 4.6m. BOREHOLE BACKFILLED WITH HOLEPLUG FROM 5.5m TO 0.3m, CONCRETE FROM 0.3m TO 0.15m, THEN ASPHALT TO SURFACE.								



UNIT WEIGHT γ kN/m³

+³, ×³: Numbers refer to Sensitivity

20
15 ⊕ 5 (%) STRAIN AT FAILURE
10

Appendix B

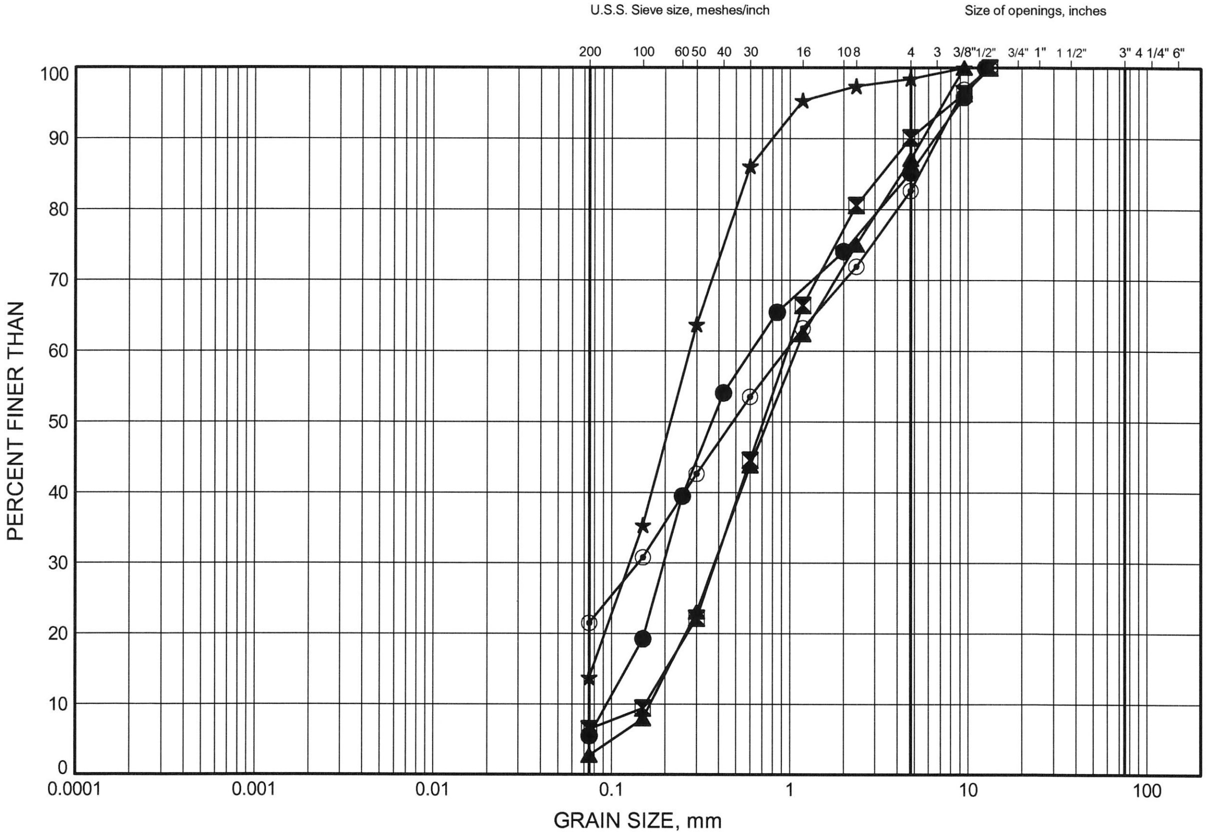
Laboratory Test Results

Sturgeon River Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND FILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SRB-01	1.07	406.73
⧗	SRB-02	1.07	406.75
▲	SRB-03	0.46	407.36
★	SRB-05	1.07	407.20
⊙	SRB-06	1.83	406.69

GRAIN SIZE DISTRIBUTION - THURBER 0840.GPJ 5/14/14

Date May 2014

W.P. 6082-09-01

THURBER

Prep'd AN

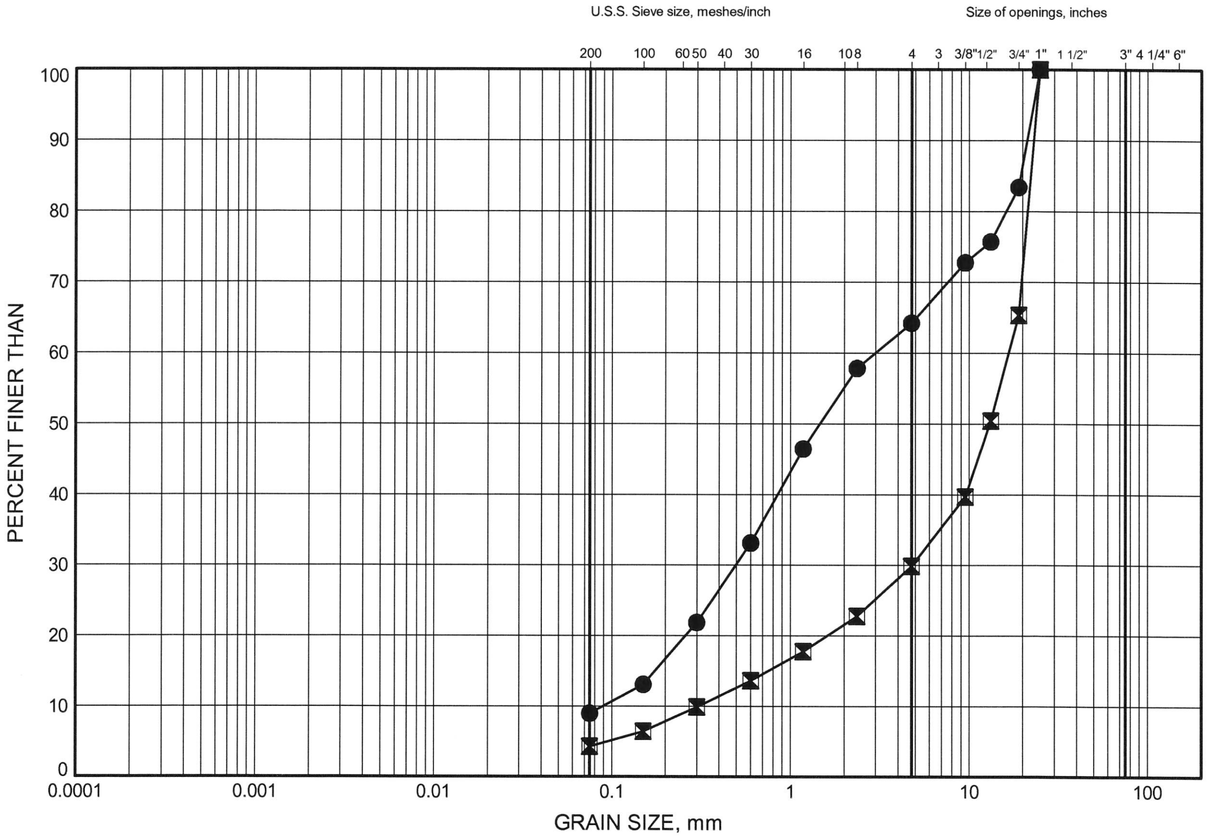
Chkd. MRA

Sturgeon River Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B2

GRAVELLY SAND TO SANDY GRAVEL FILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SRB-03	3.35	404.46
☒	SRB-04	1.07	407.18

Date May 2014

W.P. 6082-09-01

THURBER

Prep'd AN

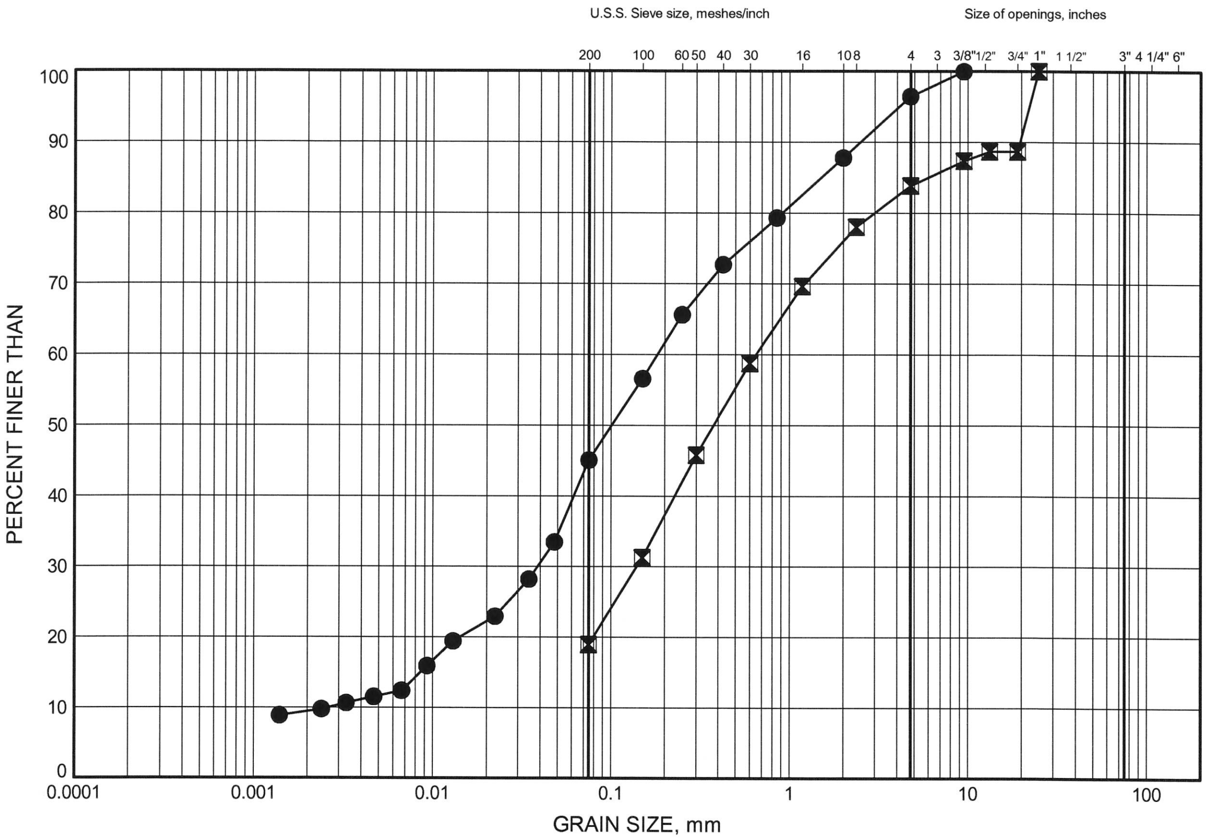
Chkd. MRA

Sturgeon River Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B3

SAND TO SAND & SILT



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SRB-01	4.88	402.92
☒	SRB-02	4.88	402.94

Date May 2014

W.P. 6082-09-01

THURBER

Prep'd AN

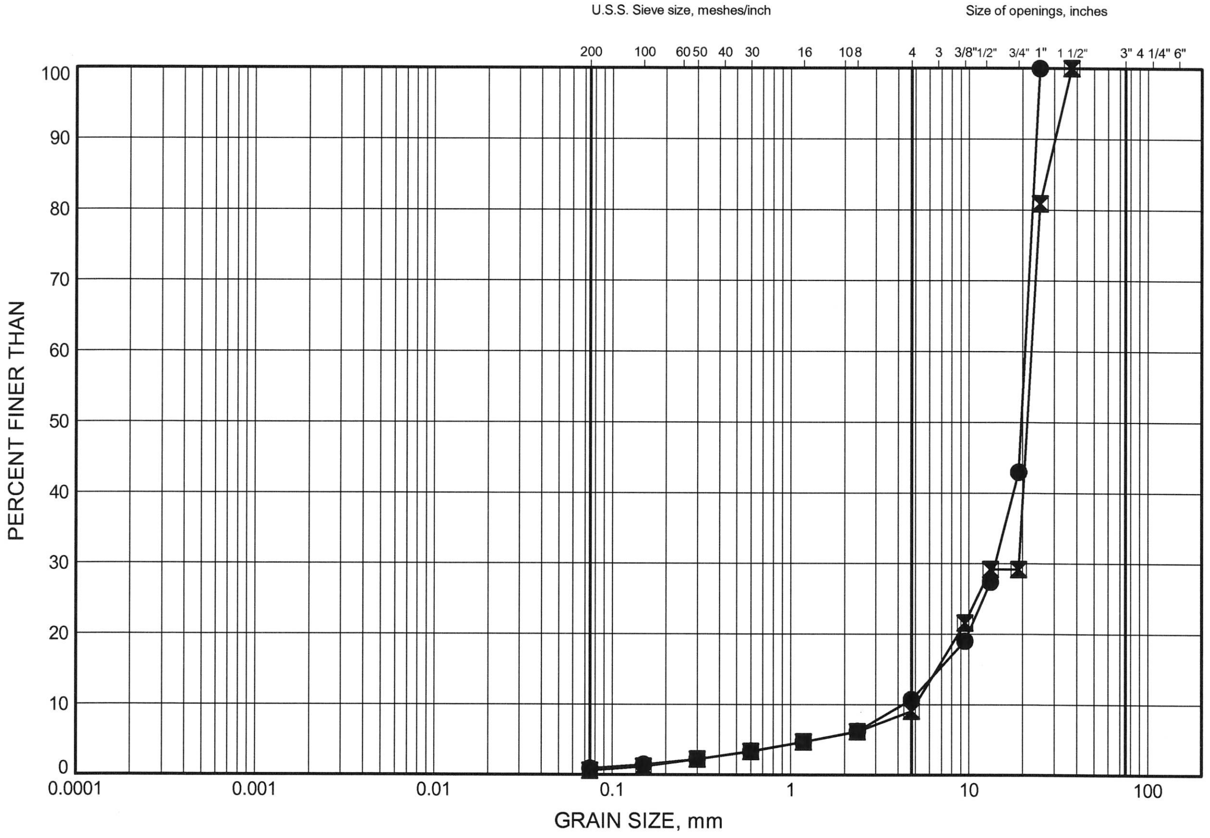
Chkd. MRA

Sturgeon River Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B4

GRAVEL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SRB-04	4.88	403.37
☒	SRB-05	4.88	403.39

Date May 2014

W.P. 6082-09-01

THURBER

Prep'd AN

Chkd. MRA

Appendix C

Site Photographs

Photograph 1 – North approach, looking south

Photograph 2 – South approach, looking north

Photograph 3 – South crib abutment

Photograph 4 – Riverbed on east side of bridge

Appendix D

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES

Spread Footings	Augered Caissons	Steel H-Piles	Drilled-in Pipe Piles
Advantages: i. Generally less costly construction than deep foundation elements.	***Advantages:*** i. High geotechnical resistance available for caissons founded on bedrock. ii. Construction of caissons could continue in freezing weather. iii. Excavation requirements are minimized.	***Advantages:*** i. High geotechnical resistance available for H-piles driven to bedrock. ii. Installation of piles could continue in freezing weather. iii. Excavation and dewatering requirements are minimized. iv. Pile base inspection not required.	***Advantages:*** i. High geotechnical resistance available for pipe piles socketed into bedrock. ii. Excavation and dewatering requirements are minimized. iii. Liner is not required to support excavation sidewalls. iv. Less vibration and disturbance than driven piles. v. Cleaning and inspection of the socket base is not required.
Disadvantages: i. Higher unit cost than footings. ii. Variable geotechnical resistance on native soils iii. Requires removal of existing crib abutments and excavation in cohesionless soils below water level. iv. Bedrock not present within a reasonable excavation depth. v. Temporary shoring and dewatering required for construction of footings on native soils or placement of engineered fill. vi. Temporary excavation for footing construction may have environmental impact on creek.	***Disadvantages:*** i. Higher unit cost than footings. ii. Measures will be required to provide sidewall support during drilling through cohesionless materials. iii. Base instability in the cohesionless material, or difficulty in obtaining a seal in bedrock socket. Tremie concrete may be required. iv. Potential difficulty in cleaning and inspection of socket base. v. Cobbles and boulders may slow caisson installation.	***Disadvantages:*** i. Higher unit cost than footings on bedrock. ii. H-piles may encounter refusal at varying depths on cobbles and boulders in existing embankment fill or native soil. iii. Variable depth to bedrock. iv. Socketing into bedrock may be required if depth to bedrock is inadequate. v. Piles must be positioned to avoid existing crib abutment, or crib must be completely removed.	***Disadvantages:*** i. Possibly higher unit cost compared to other foundation options such as footings. ii. Specialized installation. iii. Piles must be socketed into very strong bedrock. iv. Concreting or grouting of the annular space within the pile socket is required.
NOT RECOMMENDED	**NOT RECOMMENDED**	**FEASIBLE**	**RECOMMENDED**

Appendix E

List of OPSS, OPSD and SPs, and Suggested Text for NSSPs

1. List of Special Provisions and OPSS Documents Referenced in this Report

- OPSS 206
- OPSS 501
- OPSS 539
- OPSS 804
- OPSS 902
- OPSS 903
- OPSS.PROV 1010

2. Suggested text for NSSP on “Construction of Driven H-piles”

Installation of H-piles shall be in accordance with OPSS 903 and the following.

Cobbles and boulders are present within the existing embankment fill and native soils on site, and boulders and rock fill are present on the embankment slopes. The cobbles, boulders and rock fill may interfere with pile installation and some piles may meet refusal on boulders or rock fill above the bedrock surface. The Contractor must be prepared to remove, dislodge or otherwise penetrate these obstructions to advance the piles to bedrock while meeting the specified deflection tolerances.

If the piles meet refusal at a depth less than the anticipated depth, the QVE must terminate driving before the pile is damaged due to over-driving. The QVE must immediately bring it to the attention of the CA. If the CA cannot resolve the issue, it must be referred to the design team for resolution.

3. Suggested text for a NSSP on “Construction of H-Piles in Rock Sockets”

Installation of H-piles in rock sockets shall be in accordance with OPSS 903 and the following.

H-pile installation in rock sockets will require advancing through cohesionless soils below the groundwater table and construction of sockets in the underlying bedrock. Bedrock is present at shallow depths. The Contractor is advised of the following:

- The cohesionless soil above the bedrock is susceptible to disturbance under conditions of unbalanced hydrostatic head, and measures must be employed to maintain sidewall stability during installation of the piles and prevent collapse/washing of cohesionless soils into the rock socket. Selection of the methods and equipment employed to achieve this is the responsibility of the Contractor.
- The installation methods and equipment must be capable of dislodging, removing or otherwise penetrating cobbles and boulders in the soils overlying the bedrock.

- The bedrock consists of very strong granodiorite (granite-type rock). The strength and hardness of this rock must be taken into account when selecting equipment to advance the socket into rock. Equipment supplied to construct or drill the rock socket must be capable of excavating the bedrock to the specified socket dimensions without disturbing or fracturing the bedrock forming the sidewalls and base of the socket. Blasting to facilitate the removal of bedrock is not permitted.
- The rock socket must be formed entirely within the bedrock below the level of any cobbles and boulders. Any length of pile above the bedrock surface will not be considered part of the specified length of rock socket.
- H-piles shall be placed centred into the holes, bearing directly on the sound rock at the bottom of the hole. Piles shall be stabilized in place by temporary supports.
- The annular space between the rock socket wall and H-pile shall be filled with 30 MPa concrete to top of existing ground. The plumbness and alignment of the pile shall be maintained during concreting.

4. Suggested Text for NSSP on “Construction of Drilled-in Pipe Piles”

Installation of drilled-in pipe piles shall be in accordance with OPSS 903 and the following.

Drilled-in pipe pile installation at this site will require penetration through granular embankment fill with cobbles, boulders and rock fill, partially removed rock-filled timber crib abutments, and cohesionless soils below the groundwater table. The piles must also be advanced into the underlying bedrock. The Contractor is advised of the following:

- The installation methods and equipment must be capable of dislodging, removing or otherwise penetrating cobbles or boulders in the embankment fill, as well as the existing timber cribs.
- The bedrock consists of very strong granodiorite (granite-like) rock. The strength and hardness of the bedrock must be taken into account when selecting equipment to advance the pile into rock. Equipment supplied to advance the pile into rock must be capable of penetrating the bedrock to create a clean socket without disturbing or fracturing the bedrock adjacent to the pile. Blasting to facilitate the removal of bedrock is not permitted.
- The rock embedment length must be formed entirely within the bedrock below the level of any cobbles overlying the bedrock. Any length of pile above the bedrock surface will not be considered part of the specified length of rock embedment.

- The annular space between the rock socket wall and pile shall be filled with 30MPa concrete or grout to the top of the bedrock surface. The plumbness and alignment of the pile shall be maintained during concreting.
- During and subsequent to installation, the pipe pile may be partially filled with water and it may not be practical to dewater the pipe prior to concreting. Tremie concreting will be required for concreting these pipe piles.

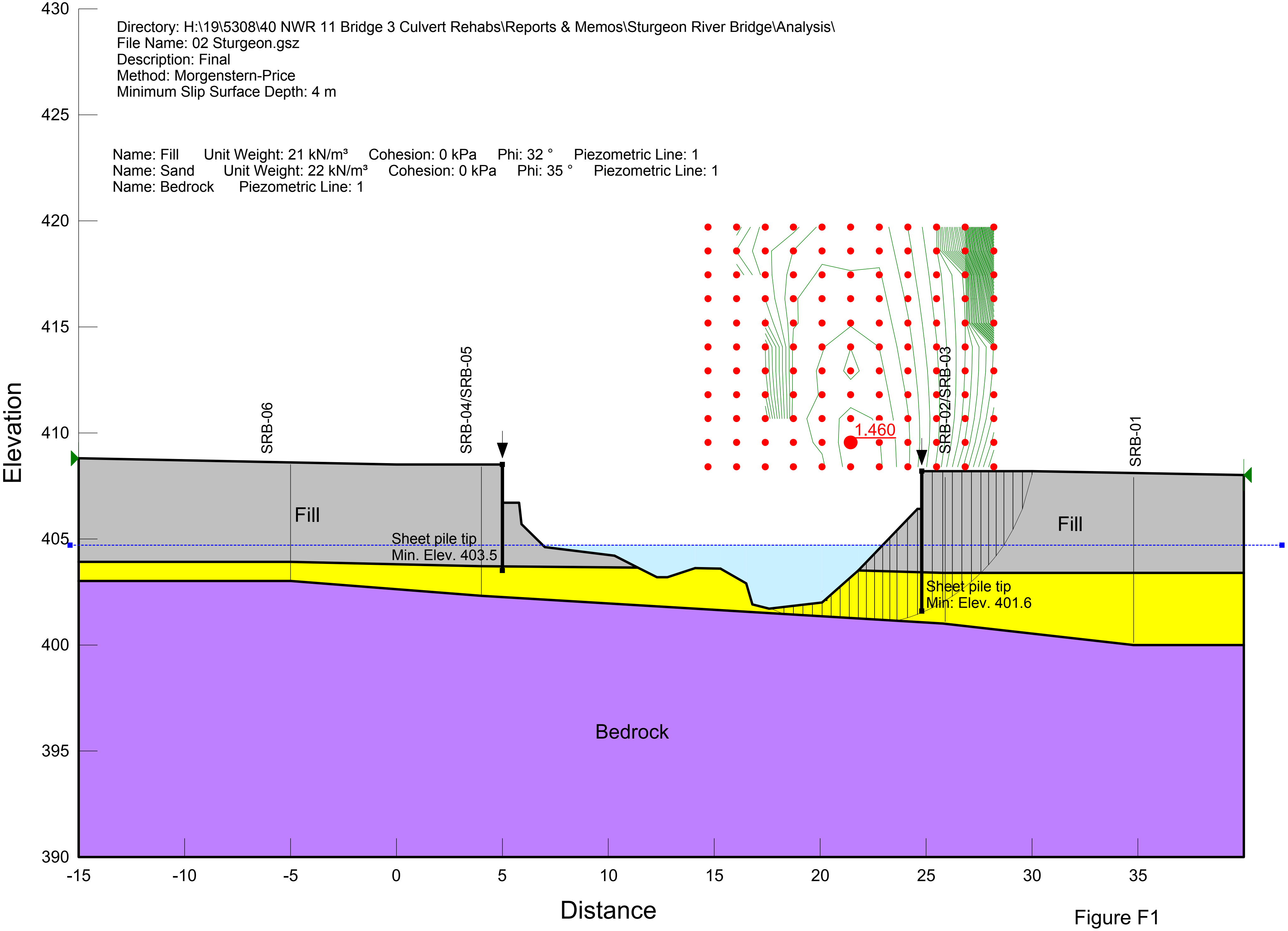
5. Suggested Text for NSSP on Installation of Steel Sheet Piles

Cobbles and boulders are present within the existing embankment fill on site, and boulders and rock fill are present on the embankment slopes. The cobbles and boulders may impede the driving of sheet piles and at some locations the sheet piles may not be able to penetrate the cobbles and boulders and reach the design depth of installation.

The Contractor shall be prepared to remove, drill through and/or penetrate these obstructions and extend the piles to the design depth.

Appendix F

Slope Stability Analyses



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Description: Final
Method: Morgenstern-Price
Minimum Slip Surface Depth: 4 m

Name: Fill	Unit Weight: 21 kN/m³	Cohesion: 0 kPa	Phi: 32 °	Piezometric Line: 1
Name: Sand	Unit Weight: 22 kN/m³	Cohesion: 0 kPa	Phi: 35 °	Piezometric Line: 1
Name: Bedrock	Piezometric Line: 1			

Figure F1

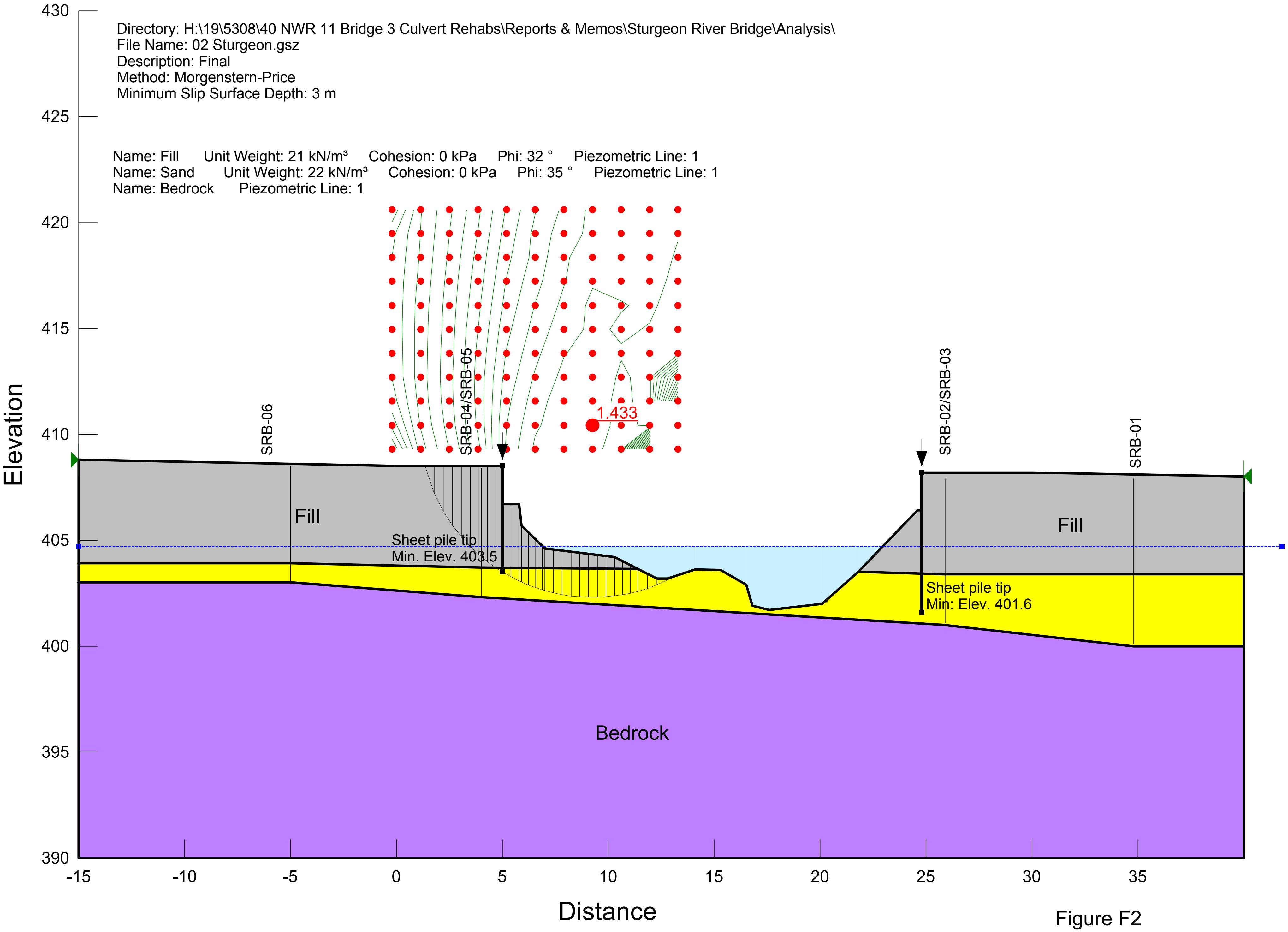
Directory: H:\19\5308\40 NWR 11 Bridge 3 Culvert Rehabs\Reports & Memos\Sturgeon River Bridge\Analysis\
File Name: 02 Sturgeon.gsz
Description: Final
Method: Morgenstern-Price
Minimum Slip Surface Depth: 3 m
Name: Fill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 32 ° Piezometric Line: 1
Name: Sand Unit Weight: 22 kN/m³ Cohesion: 0 kPa Phi: 35 ° Piezometric Line: 1
Name: Bedrock Piezometric Line: 1
1.433
SRB-06
SRB-04/SRB-05
SRB-02/SRB-03
SRB-01
Fill
Fill
Sheet pile tip
Min. Elev. 403.5
Sheet pile tip
Min. Elev. 401.6
Bedrock
Elevation
430
425
420
415
410
405
400
395
390
Distance
-15
-10
-5
0
5
10
15
20
25
30
35

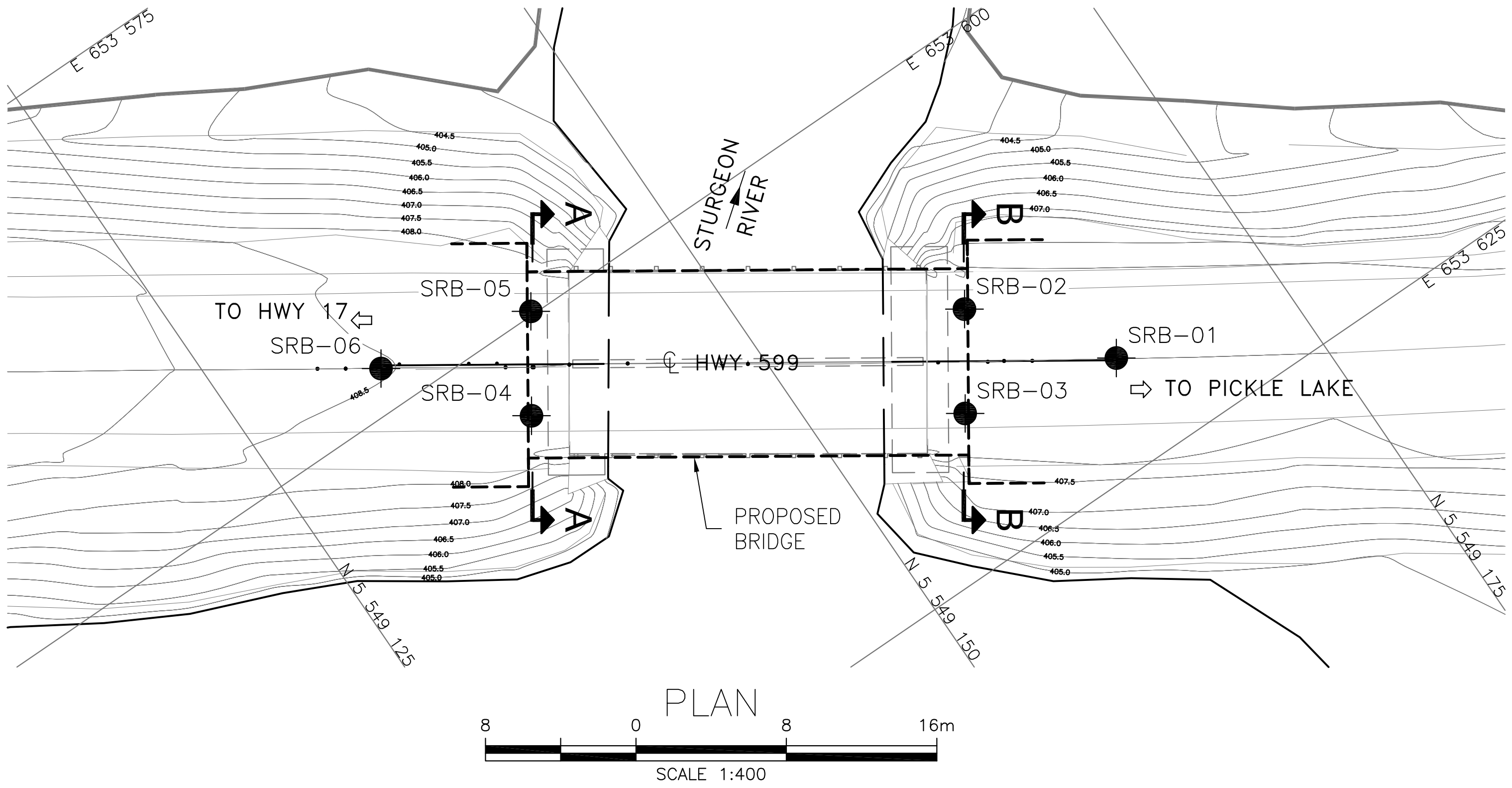
Figure F2

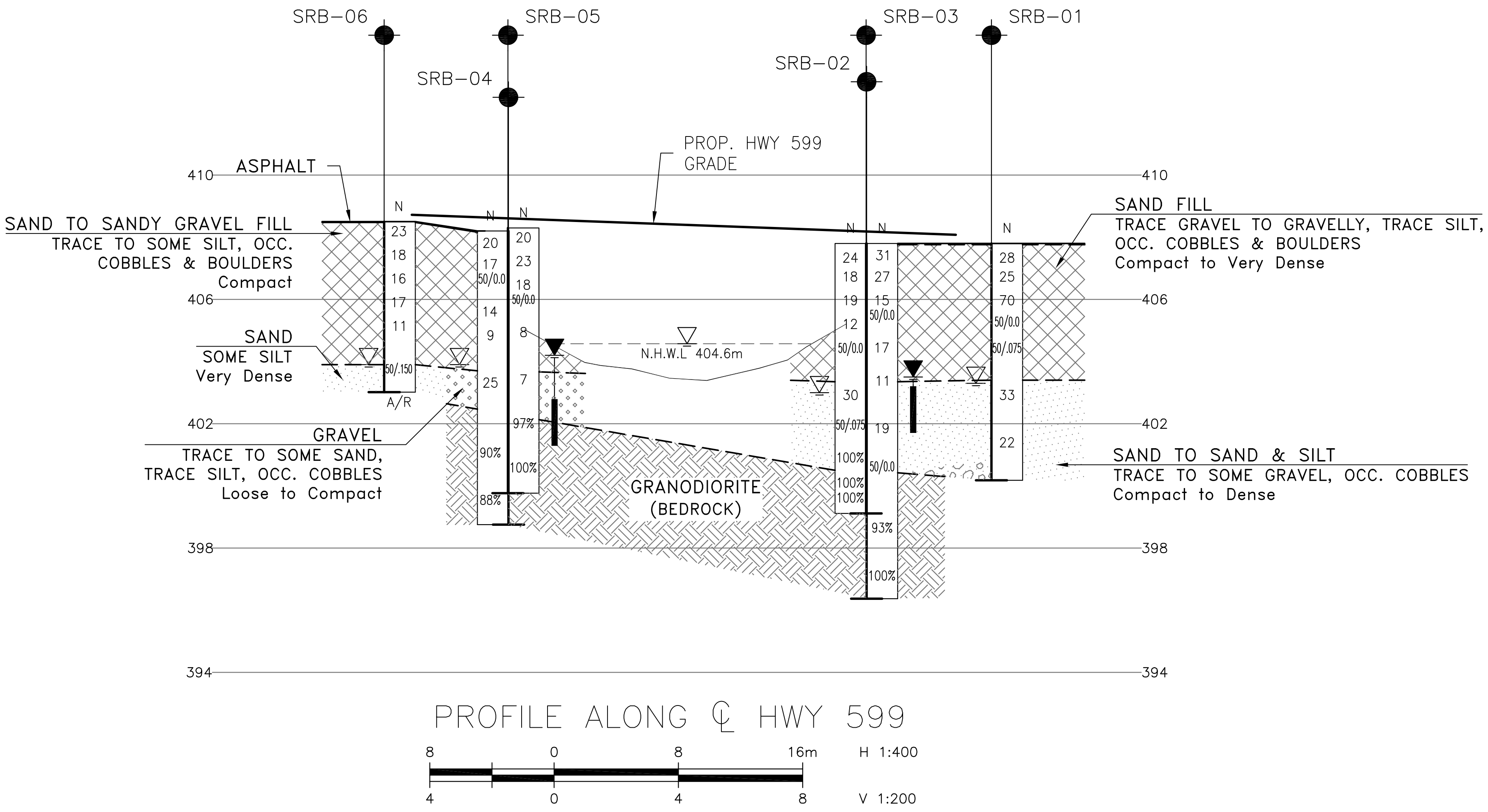
Appendix G

Borehole Locations and Soil Strata Drawings

THURBER

MINISTRY OF TRANSPORTATION, ONTARIO





METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

M. R. ANDERSON
Sept 15/14

P. K. CHATTERJI
Sep 15/14

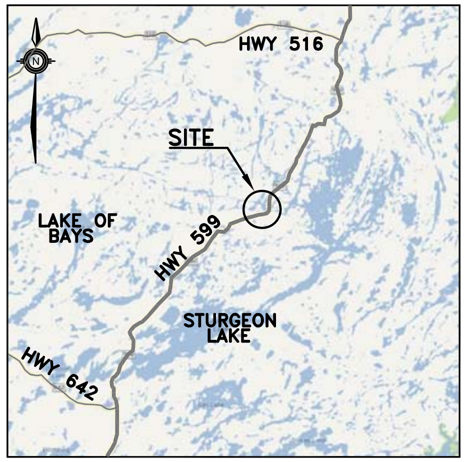
CONT No
WP No 6082–09–01

STURGEON RIVER BRIDGE
REPLACEMENT
HIGHWAY 599
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

WSP
1269 PREMIER WAY, THUNDER BAY (ONTARIO) CANADA, P7B 0A3
TEL: 807 625-6700 | FAX: 807 623-4491 | WWW.WSPGROUP.COM

THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

●	Borehole
⊕	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
▽	Water Level During Drilling
▼	Water Level In Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
SRB-01	407.8	5 549 165.5	653 619.4
SRB-02	407.8	5 549 160.3	653 612.7
SRB-03	407.8	5 549 157.2	653 617.3
SRB-04	408.2	5 549 138.1	653 604.4
SRB-05	408.3	5 549 141.2	653 599.8
SRB-06	408.5	5 549 132.9	653 597.8

-NOTES-

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

2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 52J-9

REVISIONS	DATE	BY	DESCRIPTION

DESIGN MRA	CHK MRA	CODE	LOAD	DATE SEP 2014
DRAWN AN	CHK PKC	SITE 48W–8	STRUCT	DWG 1

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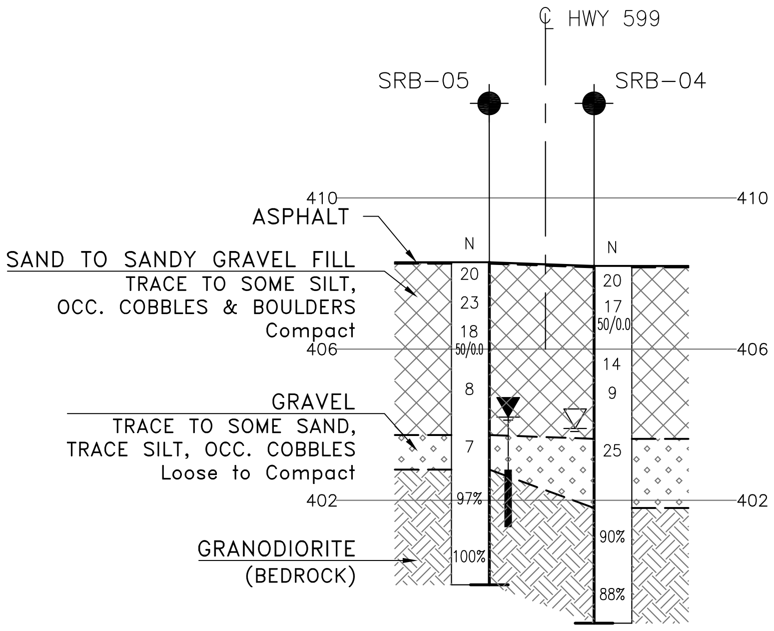
PLOTDATE: 9/17/2014 1:13 PM

MINISTRY OF TRANSPORTATION, ONTARIO

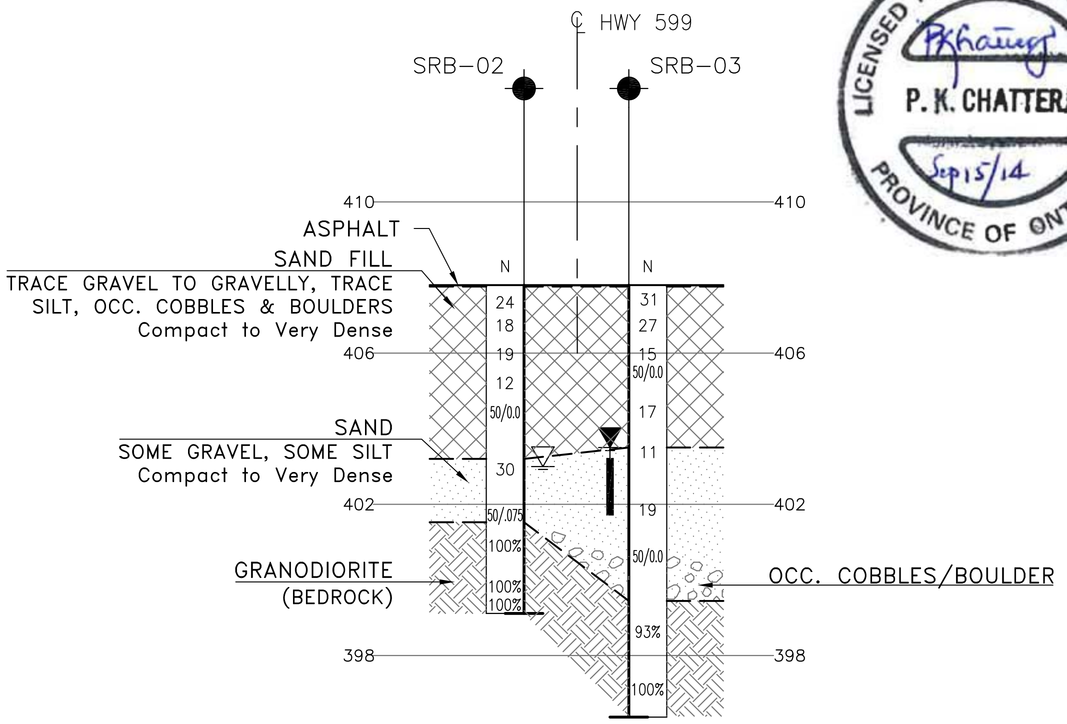
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

LICENSED PROFESSIONAL ENGINEER
M. R. ANDERSON
Sept 15/14
PROVINCE OF ONTARIO

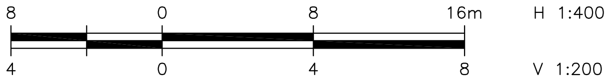
LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Sep 15/14
PROVINCE OF ONTARIO



SECTION ALONG A–A



SECTION ALONG B–B



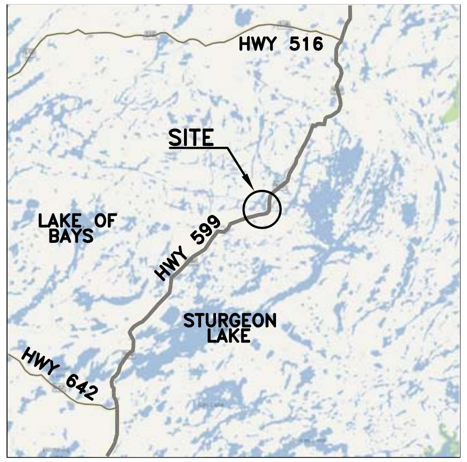
CONT No
WP No 6082–09–01

STURGEON RIVER BRIDGE
REPLACEMENT
HIGHWAY 599
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

WSP
1269 PREMIER WAY, THUNDER BAY (ONTARIO) CANADA, P7B 0A3
TEL: 807 625-6700 | FAX: 807 623-4491 | WWW.WSPGROUP.COM

THURBER ENGINEERING LTD.



KEYPLAN

L E G E N D

●	Borehole
⊕	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
▽	Water Level During Drilling
▼	Water Level In Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
SRB–01	407.8	5 549 165.5	653 619.4
SRB–02	407.8	5 549 160.3	653 612.7
SRB–03	407.8	5 549 157.2	653 617.3
SRB–04	408.2	5 549 138.1	653 604.4
SRB–05	408.3	5 549 141.2	653 599.8
SRB–06	408.5	5 549 132.9	653 597.8

-NOTES-

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

2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 52J-9

REVISIONS	DATE	BY	DESCRIPTION

DESIGN MRA	CHK MRA	CODE	LOAD	DATE SEP 2014
DRAWN AN	CHK PKC	SITE 48W–8	STRUCT	DWG 2

FILENAME: H:\Drafting\19\5308\40\ted0840–BoreholePlan&Profile(SturgeonRiver)–September2014.dwg
PLOTDATE: 9/17/2014 1:13 PM