



THURBER ENGINEERING LTD.

**PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
NAMAKAN RIVER BAILEY BRIDGE REPLACEMENT
FLANDERS ROAD
DISTRICT OF RAINY RIVER, ONTARIO
LATITUDE: 48.446705°, LONGITUDE: -92.391113°**

G.W.P. No. 6853-14-00, SITE No. 45-265/B

GEOCRES Number: 52C-57

Report

to

HATCH Corporation

Date: May 1, 2018
File: 18356



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PART 1: FACTUAL INFORMATION

1. INTRODUCTION

This report presents the factual data obtained from a preliminary foundation investigation carried out by Thurber Engineering Ltd. (Thurber) for the proposed replacement of the Namakan River Bailey Bridge on Flanders Road in the District of Rainy River, Ontario.

The purpose of this investigation was to explore the subsurface conditions at the bridge location and, based on the data obtained, to provide a borehole location plan, stratigraphic profile, records of boreholes, laboratory test results, and a written description of the subsurface conditions.

Thurber was retained by Hatch Corporation (Hatch) to carry out this foundation investigation under the Ministry of Transportation Ontario (MTO) Agreement Number 6016-E-0031.

2. SITE DESCRIPTION

The site is located on Flanders Road, approximately 50.9 km south of Highway 11 in the District of Rainy River, Ontario. Two bridges separated by an island carry Flanders Road over Namakan River at this location; a Bailey Bridge crossing the north channel of the river and an Acrow Bridge crossing the south channel. The distance between the bridge abutments on the island is approximately 30 m. Namakan River flows east to west towards Namakan Lake approximately 40 km west of the site.

The archive plan and profile drawing for the Namakan River Bridges prepared by the Ministry of Natural Resources (Rev. 1 dated July 17, 1980) indicates that the Bailey Bridge is 45.7 m long and about 6.0 m wide with a deck width of 4.3 m. The Bailey Bridge is a three-span steel structure with timber deck, supported on rock-filled timber crib abutments and two timber crib piers. The piers are situated in the water.



The existing bridge deck is at approximate Elevation 345.1 m. The water level of Namakan River beneath the bridge was measured at Elevation 340.44 m in November 2015.

The lands surrounding the bridge site predominantly consist of heavily forested areas with occasional lakes, swamps, rivers, and creeks. Local topography is hummocky and knobby and is generally of medium relief. Photographs of the bridge and surrounding area are presented in Appendix E. Bedrock outcrops are visible upstream and downstream along both banks of the river. Fractured bedrock, cobbles and boulders are present at the abutments.

Based on published geological information, the bridge lies within an area of mainly shallow or exposed bedrock with thin layers of till, sand or peat overlying the bedrock. Based on local geological maps the bedrock in the area is identified as metasedimentary.

3. INVESTIGATION PROCEDURES

The site investigation and field testing program for this project was carried out on September 26, 2017, and consisted of drilling, sampling and coring of two boreholes, designated Boreholes 17-01 and 17-02, to depths of approximately 8.0 and 6.2 m (Elevations 337.5 m and 339.4 m). One borehole was drilled from the existing road grade adjacent to each abutment.

The approximate locations of the boreholes are shown on the Borehole Locations and Soil Strata Drawing included in Appendix G.

Utility clearances were obtained prior to the start of drilling. The ground surface elevations for the boreholes were estimated from the cross sections and topographic drawings provided to Thurber by Hatch. The coordinate system MTM NAD 83, Zone 16 was used for these boreholes.

A truck mounted drill rig was used to advance the boreholes using NW casing and wash boring techniques. Soil samples were obtained in the boreholes at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). HQ coring equipment was used to penetrate cobbles and boulders encountered in the approach fills. Bedrock was proved by NQ core size in both boreholes.

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 samples for transport to Thurber's laboratory for further examination and testing. All rock cores were logged, and the Total Core Recovery (TCR), Solid Core Recovery (SCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined.



Groundwater conditions were observed in the open boreholes throughout the drilling operations and in the open boreholes upon completion of drilling. The boreholes were backfilled in general accordance with Ontario Regulation 903, as amended.

Completion details of the boreholes are summarized in Table 3.1.

Table 3.1 – Borehole Completion Details

Borehole Number	Coordinates (MTM) NAD 83, Zone 16		Ground Surface Elevation (m)	Borehole Termination Depth (Elevation), m	Completion Details
	Northing (m)	Easting (m)			
17-01	5,367,715.9	349,824.5	345.5	8.0 (337.5)	Temporary standpipe installed. After removal of standpipe, the borehole was backfilled with bentonite holeplug to surface.
17-02	5,367,759.7	349,845.8	345.6	6.2 (339.4)	Borehole backfilled with bentonite holeplug to surface.

4. LABORATORY TESTING

All recovered soil samples were subjected to visual identification and to natural moisture content determination. Selected samples were also subjected to grain size distribution analyses (sieve and/or hydrometer). Selected bedrock core specimens were also subjected to point load strength index testing. The results of this laboratory testing program are shown on the Record of Borehole sheets included in Appendix A and on the figures included in Appendix B.

In order to assess the potential for sulphate attack on concrete foundations, as well as the potential for corrosion associated with the structure, a sample of the embankment fill was collected and submitted to SGS Canada Inc., a CALA accredited analytical laboratory in Lakefield, Ontario, for analytical testing of corrosivity parameters and sulphate content. The results of the analytical testing are presented in Appendix B.

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 on the Record of Borehole sheets and on the Borehole Locations and Soil Strata Drawing included in Appendix G. A general description of the stratigraphy, based on the conditions encountered in the boreholes, is given in the following



paragraphs. However, the factual data presented on the Record of Borehole sheets takes precedence over this general description and must be used for interpretation of the site conditions. It must be recognized and expected that soil conditions may vary between and beyond the borehole locations.

In general, the subsurface conditions encountered in the boreholes consisted of sand and gravel fill overlying a layer of fragmented rock and bedrock. Descriptions of the individual strata are presented below.

5.1 Sand and Gravel Fill

Sand and gravel fill, containing some silt, occasional cobbles, and possible zones of rock fill was encountered at the ground surface in both boreholes. At depths of 1.2 to 2.3 m and 3.5 to 3.8 m at the south abutment, coring equipment was required to penetrate rock fragments in the fill. The fill extended to depths of 4.6 and 2.4 m (Elev. 340.9 and 343.2) at the south (Borehole 17-01) and north (Borehole 17-02) abutments, respectively.

Measured SPT 'N' values within the fill ranged from 16 to 59 blows per 0.3 m of penetration, indicating a compact to very dense state. 'N' values of 100 blows for less than 0.1 m of penetration were recorded where cobbles or rock fragments were encountered. The measured moisture contents in the fill ranged from 9 to 15%.

The results of grain size analyses conducted on selected samples of the sand and gravel fill are provided on the Record of Borehole sheets in Appendix A, and illustrated in Figure B1 of Appendix B. The results are summarized as follows:

Soil Particle	Sand and Gravel Fill
Gravel %	35 to 49
Sand %	42 to 52
Silt & Clay %	9 to 13

5.2 Rock Fragments

A 0.5 to 0.6 m thick layer of broken rock fragments was encountered at the base of the fill in both boreholes. Rock coring equipment was used to penetrate the fragmented rock layer.



5.3 Bedrock

Bedrock was contacted below the fill and rock fragments in both boreholes, at the following depths and elevations:

Table 5.1 - Depths and Elevations of Top of Bedrock

Borehole	Top of Bedrock	
	Depth (m)	Elevation (m)
17-01 (South Abutment)	5.1	340.4
17-02 (North Abutment)	3.0	342.6

Bedrock was proved by coring about 3 m in both boreholes. The bedrock was described as grey, slightly weathered greywacke, with near-horizontal fractures as well as fractures near 60 degrees to the horizontal. Photographs of the recovered rock core are presented in Appendix C.

Total Core Recovery (TCR) in the bedrock ranged from 89 to 100%. The Rock Quality Designation (RQD) determined from the recovered cores generally ranged from 49 to 94%, indicating a fair to excellent rock quality. One RQD value of 8% was recorded for the first core run in Borehole 17-02, indicating very poor quality.

Average unconfined compressive strengths (UCS) of the rock ranged between 147 and 190 MPa, indicating the rock is very strong. The estimated rock strength values are interpreted from point load tests that were conducted on rock cores recovered from the boreholes.

5.4 Groundwater Conditions

A temporary standpipe was installed in Borehole 17-01 upon completion of coring, and water was measured at a depth of 4.2 m (Elev. 341.3) about 4.5 hours later. The standpipe was decommissioned immediately thereafter. Water was measured at 2.9 m depth (Elev. 342.6) in Borehole 17-02 upon completion of coring, however this water is believed to represent water added to the borehole during coring operations.

The groundwater level should be assumed to reflect the water level in the river. The river water level was measured at Elevation 340.44 m in November 2015, as shown on drawings provided by HATCH.



Groundwater levels are short-term observations and seasonal fluctuations of the river water and groundwater levels are to be expected. In particular, the water levels may be at a higher elevation during spring and after periods of significant or prolonged precipitation.

6. CORROSIVITY AND SULPHATE TEST RESULTS

A sample of the sand and gravel fill from Borehole 17-02 was submitted for analytical testing of corrosivity parameters and sulphate. The results of the analytical tests are shown in Table 6.1. The laboratory certificates of analysis are presented in Appendix D.

Table 6.1 – Analytical Test Results

Parameter	Test Results
	Borehole 17-02 SS1 (0.0 to 0.6 m depth)
Sulphide %	<0.02
Chloride (µg/g)	58
Sulphate (µg/g)	75
pH	5.98
Conductivity (µS/cm)	215
Resistivity (ohms.cm)	4700
Redox Potential (mV)	382

7. MISCELLANEOUS

Thurber obtained subsurface utility clearances prior to drilling. The northing and easting coordinates and ground surface elevations were estimated based on field measurements relative to the topographic plans provided by Hatch.

Downing Drilling of Hawkesbury, Ontario supplied and operated the drilling, sampling and in-situ testing equipment for the field investigation. The field investigation was supervised on a full time basis by Mr. Amir Fereidouni of Thurber. Overall supervision of the field program was provided by Mr. Cory Zanatta, EIT. of Thurber.

Geotechnical laboratory testing was carried out in Thurber's geotechnical laboratory. Analytical laboratory testing was carried out by SGS Canada Inc.



Interpretation of the field data and preparation of this report was carried out by Mr. Cory Zanatta, EIT and Murray Anderson, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

8. GENERAL

This section of the report provides an interpretation of the geotechnical data in the factual report, and presents preliminary geotechnical recommendations to assist selection and preliminary design of the foundation system for a replacement bridge to carry Flanders Road over Namakan River in the District of Rainy River, Ontario.

Replacement of the existing three-span Bailey bridge is being considered. It is anticipated that the replacement bridge will be a single span structure with new abutments located behind the existing timber crib abutments, however the design concept has not been developed.

The discussions and recommendations presented in this report are based on the information provided by HATCH and MTO and on the factual data obtained during the course of the investigation.

The interpretation and recommendations are intended for the use of the design consultant and the Ministry of Transportation (MTO), and shall not be relied upon by any other parties including the construction contractor, or used for any purposes other than development of the project design. Comments on construction methodology and equipment, where presented, are provided only to highlight those aspects that could affect the design of the project. Contractors must make their own assessment of the factual information presented in Part 1 of the report, and the implications on equipment selection, construction methodology, and scheduling.

8.1 Existing Structure

The archive "Plan and Profile View" and "Crib Details" drawings from April 1979 indicate that the existing three span Bailey bridge is supported on rock filled timber cribs constructed on bedrock.



A ledge was to be excavated in the rock on which the cribs were to be constructed, and concrete was to be used to level the founding surface under the piers constructed in the river. The bottom timbers were to be bolted to the rock using 19 mm diameter by 1.2 m long rock bolts. Rock fill is shown as backfill to the crib abutments.

Road grade at the bridge abutments is approximate Elev. 345.5. The water level in Namakan River was reported to be Elev. 340.4 in November 2015.

The Bailey bridge is the northern of two bridges carrying Flanders Road over Namakan River at this location, separated by an island in the middle of the river. The Acrow Bridge crosses the south channel of the river, separated from the south abutment of the Bailey bridge by a distance of approximately 30 m.

9. REPLACEMENT STRUCTURE FOUNDATIONS

The subsurface stratigraphy at the site comprises existing embankment fill (sand and gravel with cobbles and rock fragments) and a layer of rock fragments, underlain by bedrock. Bedrock was contacted at depths of 5.1 m (Elev. 340.4) in Borehole 17-01 at the south abutment, and 3.0 m (Elev. 342.6) in Borehole 17-02 at the north abutment. The river water level was reported to be at elevation 340.4 m in November 2015.

Based on the subsurface conditions at the site, consideration was given to supporting the bridge using the following foundation types:

- Spread footings on bedrock
- Spread footings on rock fill and/or engineered granular fill;
- Steel H-piles; and
- Augered caissons (drilled shafts)

A comparison of the technical advantages and disadvantages of the alternative foundation schemes is presented in Appendix F.

Recommendations for design of the feasible foundation alternatives are presented in the following sections together with the corresponding geotechnical design parameters. A preferred foundation scheme from a geotechnical perspective is then recommended.

In general, the existing rock filled timber crib abutments should be removed prior to construction of new abutment foundations unless the bridge span is increased to position new abutments behind the existing cribs.



9.1 Spread Footings on Bedrock

The use of spread footings founded on bedrock is considered feasible to support the replacement structure. For evaluation of the bridge design concept, it is recommended that a geotechnical resistance of 3 MPa be assumed at factored ULS for preliminary design of footings on undisturbed bedrock at approximate Elev. 340.4 and 342.6 at the south and north abutments, respectively. A consequence factor of 1 for a “typical” degree of understanding is assumed. The resistance at SLS will not govern design of footings on bedrock.

The archive drawings and visual site observations indicate that the bedrock surface may vary in short distances. Boreholes and rock probes will be required during detailed design to define the bedrock surface profile within the foundation footprint and determine requirements for preparation of the bedrock surface to receive footing concrete. Where sloping or fractured bedrock is identified, it may be necessary to excavate the bedrock or place mass concrete to provide a sound, level founding surface, and/or dowel the footing into bedrock to improve resistance to sliding.

The specified resistance value is for a vertical, concentric load. Where eccentric or inclined loads are applied, the resistance used in design must be reduced in accordance with the CHBDC Clause 6.7.3 and Clause 6.7.4.

The lateral resistance developed along the base of concrete footings founded on sound clean bedrock may be computed using an ultimate friction coefficient of 0.7.

Excavation and backfilling for the footings must be in accordance with OPSS 902.

9.2 Spread Footings on Rock Fill or Engineered Fill

Supporting the replacement bridge on spread footings/bearing pads placed directly on the existing embankment fill was considered. However, this option is not recommended in view of the potential variability in the composition and compactness of the existing fill, the potential for voids to exist in zones of rock fill, and the potential loss of support in the event that the existing crib abutments are removed or fail.

Supporting the replacement bridge on footings/pads placed on new rock fill and/or engineered granular fill is considered feasible. Placement of the rock/engineered fill should include removal of the existing timber crib abutments and existing embankment material, and placement of new material directly on bedrock.



If employed, granular engineered fill must consist of OPSS Granular “A” placed in 150 mm lifts and compacted to 100% of its SPMDD at $\pm 2\%$ of optimum moisture content. The fill should extend laterally beyond the edge of footing a distance of at least 1.0 m plus the thickness of the engineered fill. All timber elements, boulders, and other deleterious material should be removed from the footprint of the new foundations prior to placement of the engineered fill.

Compaction of rock backfill and granular material should be carried out in accordance with OPSS.PROV 206 and OPSS.PROV 501. The top surface of the rock fill must be chinked with rock fragments to fill any voids, and covered by a minimum 450 mm thick layer of Granular A material placed as a bedding and levelling pad.

Provided a minimum footing width of 2 m is maintained, footings bearing on an engineered fill pad constructed on bedrock may be designed for the following values:

- Factored geotechnical resistance at ULS 900 kPa
- Factored geotechnical resistance at SLS (25mm settlement) 350 kPa

The lateral resistance of footings founded on engineered fill may be computed using an unfactored friction coefficient of 0.6 for cast-in-place concrete and 0.5 for pre-cast concrete.

Erosion and scour protection must be provided for granular engineered fill and any other soil surfaces that may be in contact with the river flow. In general, this should involve placement of a layer of appropriately sized rock fill as protection over granular slopes.

9.3 Steel H-Pile Foundations

Bedrock was contacted at relatively shallow depths of 5.1 m (Elev. 340.4) in Borehole 17-01 at the south abutment, and 3.0 m (Elev. 342.6) in Borehole 17-02 at the north abutment. In addition, cobbles and rock fragments are present within the existing embankment fill. The use of driven steel H-piles is not recommended for these conditions.

Considering the need to maintain traffic during construction and the relatively short separation distance available to maintain alignment with the Acrow bridge to the south, the use of socketed pile foundations may be of benefit to expedite staging operations and minimize excavation/roadway protection requirements.

If H-piles are employed, installation should involve either excavation or coring through the sand and gravel fill, coring a minimum 1.5 m long socket into rock, inserting the pile, and grouting the annular space in the socket with concrete. The actual length of socket will need to be determined



by additional investigation and coring. A socket diameter approximately 200 mm larger than the largest dimension (corner to corner) of the pile will be required.

For preliminary design purposes, a factored geotechnical resistance of 2,000 kN per pile is recommended for steel HP 310x110 piles socketed at least 1.5 m into bedrock. The SLS resistance will not govern design. Downdrag on the piles is not an issue at this site.

Boreholes and rock probes will be required during detailed design to define the bedrock surface profile along the abutment alignment. Coring will be required to assess the quality of the bedrock and confirm the geotechnical resistance.

9.4 Caissons / Drilled Shafts

Considering the need to maintain traffic during construction, the use of augered caissons may be preferred over footings as a means to minimize roadway protection requirements. The use of augered caissons would however require augering/coring through sand and gravel fill with cobbles and rock fragments, and potentially socketing into bedrock.

For evaluation of the bridge design concept, it is recommended that a geotechnical resistance of 3 MPa be assumed at factored ULS for preliminary design of caissons end-bearing on undisturbed bedrock at approximate Elev. 340.4 and 342.6 at the south and north abutments, respectively. The resistance at SLS will not govern design of caissons bearing on bedrock.

9.5 Frost Cover

The design depth of frost penetration at this site is 2.3 m as per Ontario Provincial Standard Drawing (OPSD) 3090.100. Frost penetration is not an issue for footings bearing on bedrock. Concrete slab foundations for modular bridges may be founded on an engineered fill pad with a minimum embedment of 0.5 m.

9.6 Recommended Foundation

From a geotechnical perspective, the preferred foundation option to support the replacement bridge comprises spread footings founded on bedrock. Construction of footings on rock fill or engineered fill may also be considered. The use of socketed H-piles or caissons end-bearing on bedrock may be more expeditious subject to staging alignments and roadway protection requirements.

10. ABUTMENT BACKFILL AND LATERAL EARTH PRESSURES

Backfill to the abutments should consist of rock backfill or free-draining granular material conforming to OPS Granular A or B Type II specifications. Rock backfill must be restricted to a maximum dimension of 250 mm. The rock backfill and granular material should be placed to the extents shown in OPSD 3101.200 or 3121.150. Compaction should be carried out in accordance with OPSS.PROV 206 and OPSS.PROV 501.

Earth pressures acting on the structure may be assumed to impose a triangular distribution governed by the characteristics of the backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

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

Where:

- p = horizontal earth pressure on the wall at depth h (kPa)
- K = earth pressure coefficient (see table below)
- γ = unit weight of retained soil (see table below)
- h = depth below top of fill where pressure is computed (m)
- q = value of any surcharge (kPa)

The earth pressure coefficients are dependent on the material used as backfill. Recommended unfactored values are shown in Table 10.1. The at-rest coefficients should be employed for restrained walls. Active pressures should be used for any wingwalls or unrestrained walls.

Table 10.1 – Lateral Earth Pressure Coefficients

Loading Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		Rock Backfill $\phi = 42^\circ, \gamma = 19 \text{ kN/m}^3$	
	Horizontal Backfill	Sloping Backfill (2H:1V)	Horizontal Backfill	Sloping Backfill (2H:1V)
Active (Unrestrained Wall)	0.27	0.39*	0.20	0.25*
At-rest (Restrained Wall)	0.43	-	0.33	-
Passive	3.7	-	5.0	-

* For wing walls.

The parameters in the table correspond to full mobilization of active and passive earth pressures, and require certain relative movements between the wall and adjacent soil to produce these



conditions. The values to be used in design can be assessed from Figure C6.16 of the Commentary to the CHBDC 2014.

In accordance with Clause 6.12.3 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 or 1.7 m for Granular A or Granular B Type II.

The design of the abutment walls must incorporate measures such as weep holes and/or subdrains to permit drainage of the backfill and avoid the potential build-up of hydrostatic pressures behind the walls.

11. EXCAVATION AND TEMPORARY PROTECTION SYSTEMS

All excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the sand and gravel embankment fill, and rock fill at this site are classified as Type 3 soils. Excavation for abutment foundation construction is expected to remain above the normal river water level, however pumping from within a sandbag enclosure may be required to provide a relatively dry work site if high river water levels occur during construction.

Temporary roadway protection will be required to maintain a single lane of traffic at all times during construction. Temporary protection systems should be provided in accordance with OPSS.PROV 539 and designed for Performance Level 2 (maximum 25 mm horizontal deflection). The actual pressure distribution acting on the shoring systems is a function of the construction sequence and relative flexibility of the wall and these factors must be considered when designing the shoring system.

Use of a soldier pile and lagging system may be considered for the roadway protection at this site. Socketing of the piles into bedrock will be required in view of the shallow depth to bedrock. The use of driven or vibrated steel sheet piles is not expected to be feasible due to the presence of rock fragments in the fill and the shallow depth to bedrock.

The soil parameters in Table 11.1 may apply for design of the temporary roadway protection system with horizontal backfill.



Table 11.1 –Soil Parameters for Temporary Protection System Design

Soil Parameter	Existing Embankment Fill
Unit weight, γ	21 kN/m ³
K_a	0.28
K_p	3.5

Design of the temporary protection systems is the responsibility of the Contractor. All protection systems should be designed by a Professional Engineer experienced in such designs. The design of the roadway protection system must incorporate traffic loading and surcharge loading due to the construction equipment and operations.

12. EMBANKMENT RESTORATION

The existing Flanders Road embankment is approximately 3.0 m to 5.0 m in height at the bridge location and the existing embankment slopes appear to be stable. Provided that the embankment is reconstructed at the same slope inclination as the existing embankment, but not steeper than 2H:1V in granular/earth fill or 1.25H:1V in rock fill, the restored embankment slope should remain stable.

If widening of the approaches is planned, it is recommended that rock fill be used as widening material and the existing slope inclinations be re-instated. Settlement and stability of rock fill embankments founded on bedrock are not a concern.

Embankment slopes comprising exposed granular material must be provided with erosion protection in accordance with OPSS.PROV 804. Typically, rock protection should be provided over all surfaces with which river water is likely to be in contact. Rock fill or a vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion.

The embankment slopes should be regularly inspected and any areas of material loss potentially impacting the highway platform should be repaired with rock fill.



13. POTENTIAL FOR CORROSION AND SULPHATE ATTACK

The results of the analytical testing conducted on a sample of the sand fill from the site indicates a negligible potential for corrosion or sulphate attack on buried concrete foundations.

The potential for soil corrosion on metal is considered to be moderate, and appropriate protection measures commensurate with the above are recommended. The corrosive effects of road salts should also be considered, and appropriate protection measures are recommended if metal structural elements are used.

14. DETAILED INVESTIGATION REQUIREMENTS

Further subsurface investigation, analysis and design should be carried out during detailed design to confirm the soil and bedrock conditions at the location of the structure foundation elements and approaches. This investigation should include:

- Drilling of boreholes and rock probes within the footprint of the abutment foundations to delineate the bedrock surface at the foundation locations;
- Drilling of boreholes along the alignment of potential roadway protection systems, to determine the depth to bedrock and the thickness and composition of the fill;
- Coring of the bedrock to determine the rock strength and quality, assess the geotechnical resistance of the bedrock, and confirm rock conditions to be anticipated during socket construction, if employed; and
- Confirmation of the subsurface conditions along any new or widened approach embankments.



15. CLOSURE

Engineering analysis and preparation of this report was carried out by Mr. Cory Zanatta, EIT and Mr. Murray Anderson, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL 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

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


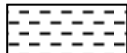



 Water Level
 Shear Strength Determination by Pocket Penetrometer

- (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

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





<u>DISCONTINUITY SPACING</u>		<u>STRENGTH CLASSIFICATION</u>			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
			(MPa)	(psi)	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Very thinly bedded	20 to 60mm				
Laminated	6 to 20mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
Thinly Laminated	Less than 6mm				
<u>TERMS</u>		Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.	Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.	Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage 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.				

RECORD OF BOREHOLE No 17-01

1 OF 1

METRIC

W.P. 18356 LOCATION Namakan River Bridge N 5 367 715.9 E 349 824.5 ORIGINATED BY AHF
 HWY Flanders Road BOREHOLE TYPE HQ/NQ Coring, HW Casing COMPILED BY MP
 DATUM Geodetic DATE 2017.09.26 - 2017.09.26 CHECKED BY CZ

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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa																
								○ UNCONFINED + FIELD VANE																
								● QUICK TRIAXIAL × LAB VANE																
						20	40	60	80	100	WATER CONTENT (%)													
345.5	GROUND SURFACE																GR SA SI CL							
0.0	SAND and GRAVEL , some silt, with cobbles Dense to Very Dense Brown Moist (FILL) 25 to 75mm rock fragments		1	SS	43		345							o			35 52 13 (SI+CL)							
			2	SS	100/ 0.075			344												TCR=29%				
									CORE															
			3	SS	31				343										o				TCR=100%	
			4	SS	100/ 0.100					342											o			
				CORE																				
	5	SS	59	341											o			49 42 9 (SI+CL)						
340.9																								
4.6	ROCK FRAGMENTS		6		SS	100/ 0.075	340													RUN #1 TCR=89% SCR=56% RQD=50% UCS=155MPa (average)				
340.4																								
5.1	BEDROCK (Greywacke), slightly weathered, grey With fractures near horizontal and 60° from horizontal.		1		RUN			339													RUN #2 TCR=89% SCR=49% RQD=49% UCS=185MPa (average)			
			2	RUN		338																		
337.5																								
8.0	END OF BOREHOLE AT 8.0m. WATER LEVEL IN TEMPORARY STANDPIPE AT 4.2m AFTER 4.5 HRS. BOREHOLE BACKFILLED WITH BENTONITE.																							

ONTMT4S MTO-18356.GPJ 2017TEMPLATE(MTO).GDT 1/18/18

RECORD OF BOREHOLE No 17-02

1 OF 1

METRIC

W.P. 18356 LOCATION Namakan River Bridge N 5 367 759.7 E 349 845.8 ORIGINATED BY AHF
 HWY Flanders Road BOREHOLE TYPE NQ Coring, NW Casing COMPILED BY MP
 DATUM Geodetic DATE 2017.09.26 - 2017.09.26 CHECKED BY CZ

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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										
345.6	GROUND SURFACE							20	40	60	80	100						
0.0	SAND and GRAVEL , some silt, occasional cobbles Compact to Dense Brown Moist (FILL)		1	SS	37	▽												
			2	SS	16													
			3	SS	22													
343.2			4	SS	100/													
2.4	ROCK FRAGMENTS 5 to 75mm size				0.100													
342.6																		
3.0	BEDROCK (Greywacke), slightly weathered, grey		1	RUN														
			2	RUN														
	With fractures near horizontal and 60° from horizontal.																	
			3	RUN														
339.4																		
6.2	END OF BOREHOLE AT 6.2m. WATER LEVEL AT 2.9m UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE BOREPLUG.																	

ONTMT4S MTO-18356.GPJ 2017TEMPLATE(MTO).GDT 1/18/18



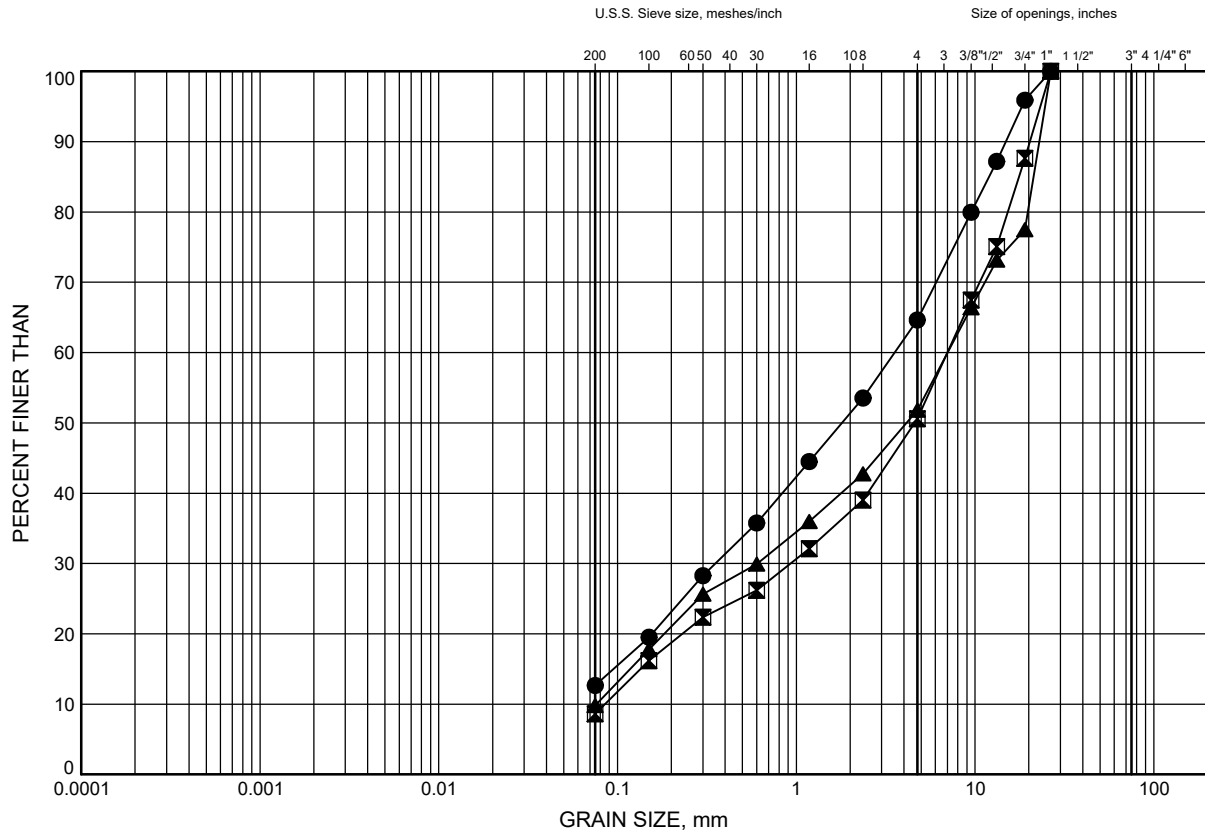
Appendix B

Geotechnical Laboratory Test Results

Namakan River Bridge GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND and GRAVEL (FILL)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	17-01	0.3	345.2
⊠	17-01	4.1	341.4
▲	17-02	1.8	343.8

Date January 2018
W.P. 18356



Prep'd AN
Chkd. MRA



Appendix C

Core Photographs

Namakan River Bridge Replacement
Photographs of Rock Core

Borehole 17-01, 1.2-2.3m, Rock Fragments



Borehole 17-01, 3.5-3.8m, Rock Fragments



Namakan River Bridge Replacement
Photographs of Rock Core

Borehole 17-01, Run 1, Bedrock



Borehole 17-02, Runs 1 to 3, Bedrock





Appendix D

Analytical Laboratory Test Results

RESULTS

Sample Number	5	6
Sample Name	BP-03 SS-5	17-02 SS-1
Sample Matrix	Soil	Soil
Sampled By	Sarah Hashimi	Sarah Hashimi

Parameter	Units	RL	Result	Result
-----------	-------	----	--------	--------

| Internal ref.: ME-CA-[ENV]EWL-LAK-AN-27

Corrosivity Index	none	1	1	1
Soil Redox Potential	mV	-	309	382
Resistivity (calculated)	ohms.cm	-9999	8100	4700

Anions by IC

Method: EPA300/MA300-Ions1.3 | Internal ref.: ME-CA-[ENV]IC-LAK-AN-001

Chloride	µg/g	0.4	3.5	58
Sulphate	µg/g	0.4	130	75

Carbon/Sulphur

Method: ASTM E1915-07A | Internal ref.: ME-CA-[ENV]JARD-LAK-AN-020

Sulphide	%	0.02	< 0.02	< 0.02
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Conductivity

Method: SM 2510 | Internal ref.: ME-CA-[ENV]EWL-LAK-AN-006

Conductivity	uS/cm	2	123	215
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Moisture

Method: CCME Tier 1 | Internal ref.: ME-CA-[ENV]GC-LAK-AN-010

Moisture Content	%	0.1	11.6	6.9
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pH

Method: SM 4500 | Internal ref.: ME-CA-[ENV]EWL-LAK-AN-001

pH	no unit	0.05	6.91	5.98
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Appendix E

Site Photographs

Namakan River Bridge Replacement Site Photographs



Photograph 1 – West side of Namakan River Bridge, looking north



Photograph 2 – East side of Namakan River Bridge, looking north



Appendix F

Foundation Comparison



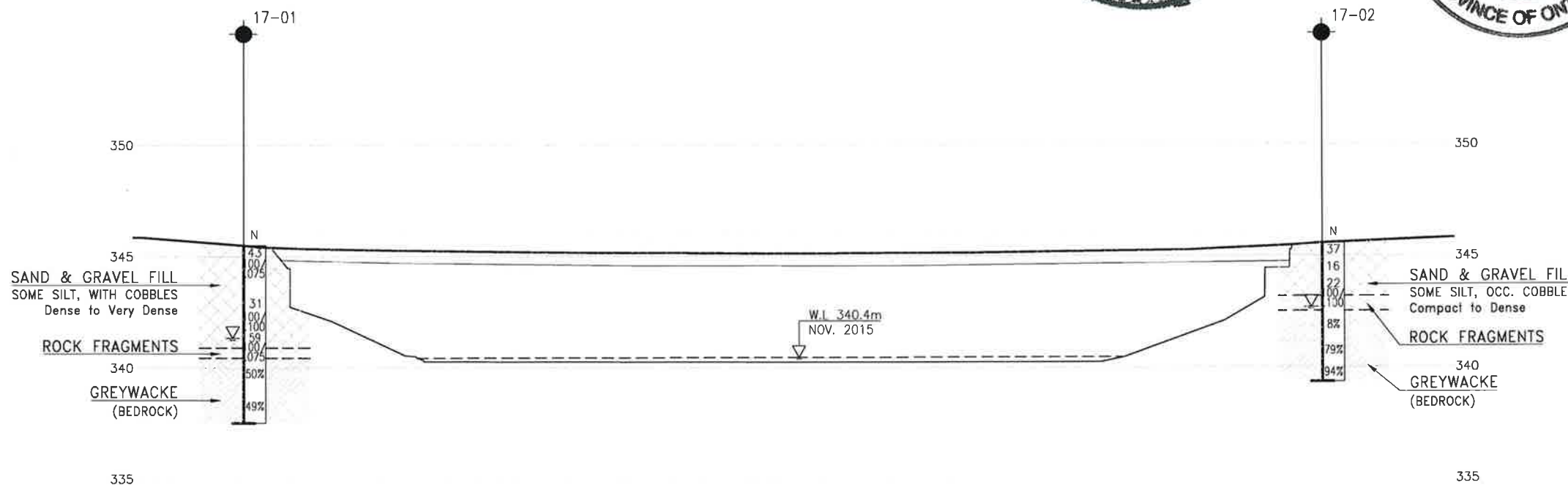
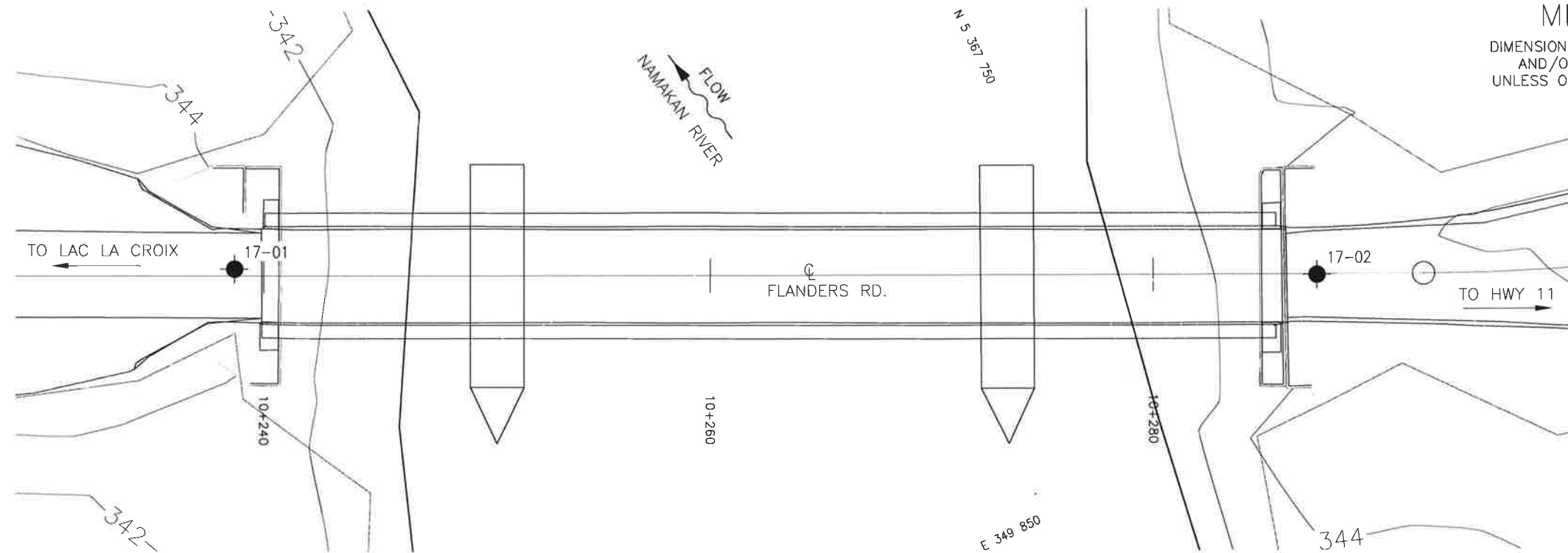
COMPARISON OF FOUNDATION ALTERNATIVES

Footings on Bedrock	Footings on Rock Fill or Engineered Fill	Socketed Piles	Caissons
<p>Advantages:</p> <ul style="list-style-type: none"> i. Ease of construction. ii. High resistance values are available on bedrock at shallow depth. iii. Scour of bedrock of minimal concern. iv. Lower cost than deep foundations. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Does not allow use of integral abutment design. ii. Excavation of existing sand and gravel fill with cobbles/rock fragments required. iii. Bedrock elevation may vary. iv. Roadway protection required. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Ease of construction. ii. Design founding level can be controlled. iii. Generally lower cost than deep foundations. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Additional cost of engineered fill placement. ii. Limited benefit as bedrock is present at relatively shallow depth. iii. Roadway protection required to maintain traffic. iv. Granular fill must be protected from scour and erosion 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Piles will develop high geotechnical resistance on bedrock. ii. May require less excavation than footing construction, reducing roadway protection requirements. iii. Possibly more adaptable to staging alignments / support of temporary bridge. iv. Pile installation may continue in freezing weather. v. Allows use of integral abutments. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit cost than footings. ii. Need to socket piles into very strong bedrock. iii. Augering through existing sand and gravel fill with cobbles/rock fragments required. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High resistance is available for caissons founded on bedrock. ii. May require less excavation than footing construction, reducing roadway protection requirements. iii. Construction could continue in freezing weather. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Much higher cost than shallow footings. ii. Temporary steel liners may be required to install caissons through cohesionless fill. iii. Augering through existing sand and gravel fill with cobbles/rock fragments required iv. Difficulty in cleaning and inspecting bases.
RECOMMENDED	FEASIBLE	FEASIBLE	FEASIBLE



Appendix G

Borehole Locations and Soil Strata Drawing



PROFILE ALONG FLANDERS RD.



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

CONT No
WP No

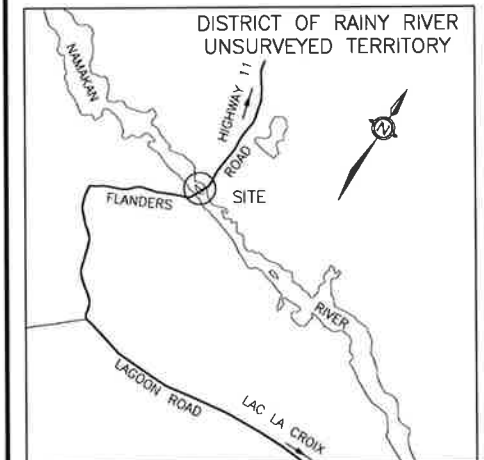
FLANDERS ROAD
NAMAKAN RIVER BRIDGE

BOREHOLE LOCATIONS AND SOIL STRATA

HATCH



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
- Head Artesian Water
- Piezometer
- 90%
- Rock Quality Designation (RQD)
- A/R
- Auger Refusal

NO	ELEVATION	NORTHING	EASTING
17-01	345.5	5 367 715.9	349 824.5
17-02	345.6	5 367 759.7	349 845.8

NOTES

- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCREs No. 52C-57

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	CZ	CHK	MM
DRAWN	AN	CHK	CZ
CODE	LOAD	DATE	MAY 2018
SITE	STRUCT	DWG	1