



**THURBER** ENGINEERING LTD.

**PRELIMINARY  
FOUNDATION INVESTIGATION AND DESIGN REPORT  
MOIRA RIVER BRIDGE REPLACEMENT  
HIGHWAY 401 WIDENING EA  
SITE No. 11-162  
BELLEVILLE, ONTARIO  
G.W.P. 4193-15-00**

**GEOCRES NO. 31C-311**

**Latitude: 44.193524°  
Longitude: -77.393161°**

**Report**

to

**WSP**

Date: July 5, 2021  
File: 11566



## TABLE OF CONTENTS

### **PART 1: FACTUAL INFORMATION**

|     |  |   |
|-----|--|---|
| 1.0 | INTRODUCTION .....                         | 1 |
| 2.0 | SITE DESCRIPTION .....                     | 2 |
| 3.0 | SITE INVESTIGATION AND FIELD TESTING.....  | 3 |
| 4.0 | LABORATORY TESTING .....                   | 4 |
| 5.0 | DESCRIPTION OF SUBSURFACE CONDITIONS ..... | 4 |
| 5.1 | Pavement Structure .....                   | 5 |
| 5.2 | Embankment Fill.....                       | 5 |
| 5.3 | Limestone Bedrock.....                     | 6 |
| 5.4 | Groundwater Conditions .....               | 7 |
| 6.0 | MISCELLANEOUS .....                        | 8 |

### **PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS**

|      |  |    |
|------|--|----|
| 7.0  | GENERAL .....  | 10 |
| 8.0  | FOUNDATION DESIGN.....                                   | 11 |
| 8.1  | Foundation Alternatives .....                            | 11 |
| 8.2  | Spread Footings on Bedrock .....                         | 12 |
| 8.3  | Caissons Socketted in Bedrock at Pier .....              | 13 |
| 8.4  | Frost Cover.....   | 14 |
| 9.0  | APPROACH EMBANKMENTS .....                               | 14 |
| 10.0 | ABUTMENT WALL BACKFILL AND LATERAL EARTH PRESSURES ..... | 15 |
| 11.0 | EXCAVATION AND GROUNDWATER CONTROL.....                  | 16 |
| 12.0 | SCOUR PROTECTION AND EROSION CONTROL.....                | 17 |
| 13.0 | TEMPORARY PROTECTION SYSTEMS .....                       | 18 |
| 14.0 | INVESTIGATION FOR DETAIL DESIGN .....                    | 18 |
| 15.0 | CLOSURE .....  | 19 |



## **APPENDICES**

|            |   |
|------------|---|
| Appendix A | Record of Borehole Sheets                   |
| Appendix B | Geotechnical Laboratory Test Results        |
| Appendix C | Rock Core Photographs                       |
| Appendix D | Select Site Photographs                     |
| Appendix E | Borehole Locations and Soil Strata Drawings |



**PRELIMINARY  
FOUNDATION INVESTIGATION AND DESIGN REPORT  
MOIRA RIVER BRIDGE REPLACEMENT  
HIGHWAY 401 WIDENING EA  
SITE No. 11-162  
BELLEVILLE, ONTARIO  
G.W.P. 4193-15-00**

**GEOCRES NO. 31C-311**

**PART 1: FACTUAL INFORMATION**

**1.0 INTRODUCTION**

This report presents the factual findings obtained from a preliminary foundation investigation carried out by Thurber Engineering Ltd. (Thurber) for the preliminary (EA) design of the proposed Moira River Bridge replacement located at the crossing of Highway 401 over Moira River in the geographic township of Thurlow, Municipality of Belleville, Ontario.

The purpose of the investigation was to explore the subsurface conditions at selected locations at the site and, based on the data obtained, provide a borehole location plan, records of boreholes, stratigraphic profiles, and a written description of the subsurface conditions. A model of the subsurface conditions was developed to describe the geotechnical conditions influencing the preliminary design of the structure foundations.

Thurber was retained by WSP to carry out this foundation investigation under the Ministry of Transportation Ontario (MTO) Assignment Number 4015-E-0036. The entire project includes preliminary design for Highway 401 widening from Wallbridge-Loyalist Road Interchange easterly to approximately 5 km east of Highway 62 interchange, replacement and rehabilitation of several structures within this section of highway, and preliminary design for a new Highway 62 Norris Whitney twin bridge and structural rehabilitation of the existing bridge.

There is no Geocres information directly related to this site. In preparation of this report, reference has been made to information on subsurface conditions contained in existing



foundation reports for locations close to this site, archive design drawings and a memorandum prepared for this site. The titles of these documents are:

- Draft Preliminary Investigation and Design Memorandum, Moira River Bridge (Site 11-162), prepared by Thurber Engineering, File 19-4406-20, dated January 7, 2015 (Reference 1).
- Foundation Investigation - Thurlow Twp. Bridge No. 2 prepared by Racey, McCallum and Associates Consulting Engineers, Report No. S-500-501/55/T-61-1, dated March 17, 1955, Geocres No. 31C00-028 (Reference 2).
- Foundation Investigation - Thurlow Twp. Bridge No. 5 prepared by Racey, McCallum and Associates Consulting Engineers, Report No. S-500-501/55/T-62-1, dated March 22, 1955, Geocres No. 31C00-027 (Reference 3).
- Archive drawings – Thurlow Township Bridge No. 5, Moira River Crossing, Department of Highways, Ontario, dated April 1955, and Highway 401, Moira River Bridge, General Arrangement I, McCormick Rankin Corporation, dated March 2000 (Reference 4).

## **2.0 SITE DESCRIPTION**

The site is located on Highway 401 between Highway 62 and Highway 37 in Belleville, Ontario.

Available information indicates that the existing bridge was constructed in 1956 and consists of a three-span steel plate I-girder bridge with a total length of 78.9 m (22.7 m , 33.5 m , 22.7 m). The bridge has an overall width of 36 m and carries six lanes of traffic on Highway 401 over the Moira River. The road grade on the bridge is approximately 4.5 m above the river channel. The bridge underwent structural rehabilitation in 2000.

The natural terrain in the vicinity of the bridge is generally flat. Bedrock is exposed at various locations along the flat, wide river channel. Lands to the northwest of the bridge are typically of commercial usage. The river channel is approximately 68 m wide and the river is flowing in a southerly direction. The floodplain is mainly vegetated with grass, shrubs and some trees.

Selected photographs of the site, taken during the course of this investigation in 2020 and during a site visit conducted in 2018, are presented in Appendix D.



The project area is situated within the physiographic region known as the Napanee Plain. The Napanee Plain is characterized by a thin veneer of glacial till underlain at relatively shallow depths by limestone bedrock of the Simcoe Group. Thick glacial sediments are present in the deep river and stream valleys in the region. There are a few scattered drumlins in this area.

### **3.0 SITE INVESTIGATION AND FIELD TESTING**

The current borehole investigation and field testing program were carried out between November 11 and 12, 2020, and consisted of drilling and sampling two (2) boreholes, designated as Boreholes MR20-1 and MR20-2. Borehole MR20-1 was drilled on the Highway 401 platform near the northwest corner of the existing west abutment, and Borehole MR20-2 near the southeast corner of the existing east abutment. Boreholes MR20-1 and MR20-2 were terminated at 5.3 m and 6.0 m depth (Elevations 89.6 and 88.4), respectively. Both boreholes were advanced into limestone bedrock by coring 3.0 m to 3.1 m. The approximate locations of the boreholes are shown on the Borehole Location Plan and Stratigraphic Drawing in Appendix E. The records of borehole sheets are provided in Appendix A. In agreement with WSP and MTO, a borehole was not advanced in the river near the piers, as originally proposed, due to logistics such as environmental issues related to working within the river floodplain as well as the need to drill through the bridge deck.

WSP surveyed the as-drilled boreholes in the field and provided Thurber with the borehole coordinates and ground surface elevations. It is understood that the horizontal and vertical accuracy of the survey results meet the MTO terms of reference requirements of 0.5 m and 0.1m, respectively.

Lane closures and traffic control were implemented during drilling of the boreholes for the investigation. Prior to commencement of drilling, utility clearances were obtained for both borehole locations.

The boreholes were advanced using a track-mounted drill rig with hollow stem augers. Soil samples were obtained at selected intervals using a 50 mm outside diameter split-spoon sampler driven in conjunction with the Standard Penetration Test (SPT) in general accordance with ASTM D1586. NQ rock coring equipment was used to recover core samples of the underlying bedrock in both boreholes.



All rock cores were logged, and the Total Core Recovery (TCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined.

The field investigation was supervised on a full-time basis by a member of Thurber's technical staff who marked/staked the boreholes in the field, arranged for the clearance of subsurface utilities, supervised the drilling, sampling and in-situ testing operations, logged the boreholes and processed the recovered soil and rock samples for transport to Thurber's Toronto area laboratory for further examination and testing.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Boreholes were backfilled upon completion of drilling in general conformance with O.Reg. 903 as amended by O.Reg.128/03. The asphalt surface was reinstated in the boreholes upon completion.

#### **4.0 LABORATORY TESTING**

The recovered soil samples were subjected to visual identification (VI) and natural moisture content determination. Selected samples were subjected to grain size distribution analyses (sieve and/or hydrometer) testing. Geotechnical laboratory testing results are summarized on the Record of Borehole sheets included in Appendix A and are presented on the figures included in Appendix B.

Bedrock core samples were subjected to geological logging. Point load tests were carried out on selected samples of intact limestone upon arrival at the laboratory to assist in evaluation of the compressive strength of the bedrock. Detailed results of point load tests on the selected rock core samples are included in Appendix B and summarized results on the Record of Borehole sheets in Appendix A. Rock core photos are presented in Appendix C.

#### **5.0 DESCRIPTION OF SUBSURFACE CONDITIONS**

Details of the encountered soil and rock stratigraphy are presented on the Record of Borehole sheets included in Appendix A, and on the Borehole Locations and Soil Strata drawings in Appendix E. A general 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 and anticipated that soil conditions may vary



between and beyond the borehole locations. As noted earlier, no boreholes were drilled near the pier locations.

In general, the subsurface stratigraphy near the abutments consists of a pavement structure and typically compact cohesionless embankment fill overlying limestone bedrock at 2.3 m to 2.9 m depths. The groundwater level during drilling was noted at approximately 2.1 m to 2.7 m depths below the existing ground.

More detailed descriptions of the individual stratum are presented below.

### **5.1 Pavement Structure**

Pavement structure consisting of approximately 175 mm to 250 mm of asphalt overlying granular (sand and gravel fill) road base was encountered in both boreholes advanced through the Highway 401 platform. The granular fill ranged in thickness from 0.5 m to 1.2 m.

SPT 'N' values recorded in the sand and gravel fill ranged from 11 to 21 blows per 0.3 m of penetration indicating a compact condition. The moisture contents measured on samples of the road base fill ranged from 4 percent to 8 percent.

### **5.2 Embankment Fill**

Embankment fill was encountered below the pavement structure in both boreholes, and consisted of layers of dark brown, brown and grey silty sand, sand and gravel, and sand containing trace to some clay with clay pockets. The thickness of the embankment fill was 1.6 m and 1.4 m in Boreholes MR20-1 and MR20-2, respectively. The depth to the base of the embankment fill was at 2.3 m and 2.9 m (Elevations 92.6 and 91.5) in MR20-1 and MR20-2, respectively.

The SPT 'N' values recorded in the embankment fill typically ranged from 12 to 21 blows per 0.3m of penetration indicating a compact condition. An SPT 'N' value of 3 blows per 0.3 m of penetration, indicating a very loose zone, was measured in Borehole MR20-02 below Elevation 92.9. The natural moisture contents measured on samples of the cohesionless fill ranged from 6 percent to 15 percent.





The results of grain size analyses conducted on samples of the silty sand fill and sand fill are provided on the Record of Borehole sheets in Appendix A, and illustrated on Figures B1 and B2 of Appendix B. The results are summarized as follows:

| Soil Particle | Embankment<br>Silty Sand Fill<br>(Percent) | Embankment<br>Sand Fill<br>(Percent) |
|---------------|--|--------------------------------------|
| Gravel        | 8 to 15                                    | 0                                    |
| Sand          | 47 to 57                                   | 72                                   |
| Silt          | 23 to 28                                   | 15                                   |
| Clay          | 10 to 12                                   | 13                                   |

### 5.3 Limestone Bedrock

The soils described above were found to be underlain by bedrock consisting of limestone with shale interbeds of the Simcoe Group. This limestone is typically fossiliferous, argillaceous and laminated, and varies from medium to thickly bedded. The recovered rock cores are grey to dark grey in colour and described as horizontally bedded and highly to moderately weathered within the upper 2 m, becoming moderately weathered with depth. These cores contain frequent shale interbeds typically ranging between 10 mm and 60 mm in thickness with an occasional interbed up to 90 mm. Rock core photos are presented in Appendix C.

Depth and elevations of the top of bedrock encountered in the present investigation are shown in Table 5.1.

**Table 5.1 – Depth and Elevation of Top of Bedrock**

| Approximate Location | Borehole | Depth to weathered Bedrock (m) | Top of Weathered Bedrock Elevation (m) |
|----------------------|----------|--------------------------------|--|
| West Abutment        | MR20-01  | 2.3 *                          | 92.6                                   |
| East Abutment        | MR20-02  | 2.9 *                          | 91.5                                   |

\* Proved by coring below augered depth.

Bedrock cores were recovered using NQ size coring equipment. TCR in the bedrock was 100 percent in all four runs.



RQD values ranged from 50 to 81 percent indicating a poor to good rock quality in three of the runs, except for Run 2 of Borehole MR20-02 where 100 percent was measured indicating an excellent rock quality. FI of the rock, expressed as fractures per 0.3 m of core, typically ranged from 0 to 5, and greater than 10 where the value corresponded to a rubble/broken zone in Run 1 of Borehole MR20-01.

Unconfined compressive strengths interpreted from point load tests conducted on selected rock cores typically varied from 62 MPa to 138 MPa, indicating a strong to very strong rock. There are zones within the bedrock where the unconfined compressive strengths ranged from 29 to 43MPa, indicating a medium strong rock. A low value of about 4 MPa was estimated for a shaley limestone interbed in Run 1 of Borehole MR20-02. Results of the point load tests are included in Appendix B.

#### 5.4 Groundwater Conditions

Groundwater levels in the boreholes were observed during the drilling operations and measured upon completion of drilling. Water levels noted in the open boreholes are presented in Table 5.2 below.

**Table 5.2- Groundwater Level Measurements**

| Borehole | Date              | Groundwater Level |               | Comments      |
|----------|-------------------|-------------------|---------------|---------------|
|          |                   | Depth (m)         | Elevation (m) |               |
| MR20-01  | November 12, 2020 | 2.1 *             | 92.8          | Open borehole |
| MR20-02  | November 11, 2020 | 2.7 *             | 91.7          | Open borehole |

\* Measured shortly after coring was completed with water added into the borehole; readings may be affected by the coring water.

The General Arrangement (GA) drawings provided by WSP dated October 2020 indicate that the water levels at Moira River are noted to be at the following elevations:

- River level in February 2007 – Elevation 91.78
- 2-year river level – Elevation 93.07



The values shown in Table 5.2 are very short-term readings and the water levels are expected to be governed by the river level. Seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after periods of significant or prolonged precipitation, and snowmelts.

## **6.0 MISCELLANEOUS**

Thurber staked and/or marked the borehole locations in the field and obtained utility clearances prior to drilling. WSP surveyed the boreholes in the field and provided the borehole coordinates and ground surface elevations.

Downing Drilling from Hawkesbury, Ontario supplied and operated the drilling and sampling equipment for the field program.

Full time supervision of the field activities was carried out by Mr. George Azzopardi, C.Tech. Overall supervision of the field program was performed by Mr. Stephane Loranger, C.E.T. of Thurber.

Interpretation of the field data and preparation of the report were carried out by Ms. Rocio Palomeque Reyna, P.Eng. The report was reviewed by Dr. Sydney Pang, P.Eng. and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.



THURBER ENGINEERING LTD.



Rocío Palomeque Reyna, P.Eng.  
Geotechnical Engineer



Sydney Pang, P.Eng.  
Associate, Senior Foundation Engineer



P.K. Chatterji, P.Eng.  
Review Principal, Designated MTO Contact



**PRELIMINARY  
FOUNDATION INVESTIGATION AND DESIGN REPORT  
MOIRA RIVER BRIDGE REPLACEMENT  
HIGHWAY 401 WIDENING EA  
SITE No. 11-162  
BELLEVILLE, ONTARIO  
G.W.P. 4193-15-00**

**GEOCRES NO. 31C-311**

**PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS**

**7.0 GENERAL**

This report presents interpretation of the geotechnical data in the factual report and provides preliminary foundation recommendations in support of preliminary design of the new Moira River Bridge and its approach fills to replace the existing structure to facilitate widening of Highway 401 in Belleville, Ontario.

Available information indicates that the existing bridge was constructed in 1956 and consists of a three-span steel plate I-girder bridge of about 79 m in length and 36 m in width, and carries six traffic lanes of Highway 401 over the Moira River. The road grade on the bridge is approximately 4.5 m above the river channel. It is understood that the existing structure is supported on two abutments and two piers founded on bedrock. The abutment footings are typically 2.1 m wide and the pier footings are approximately 1.8 m wide. The approach fills are up to the order of 4 m in height. Conversion to integral abutments and structural rehabilitation were completed in 2002.

Based on a preliminary GA drawing provided by WSP dated May 2021, the existing bridge will be replaced by a new bridge. The WBL portion will be located to the north of the existing bridge, and the new Highway 401 centreline will also be shifted north. The new bridge will be a two-span structure supported on two abutments and one pier to be founded on bedrock. There will be two equal spans each of 48 m in length for an overall length of 96 m between abutments. The overall width of the bridge deck will be in the order of 52 m or greater. The top of the proposed footings will be at approximate Elevations 92.7 and 92.1 for the west and east abutments, respectively.



The centre pier is shown to be supported on spread footings, but subsequent information from WSP indicates that caissons socketted into bedrock are also being considered.

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

## **8.0 FOUNDATION DESIGN**

In general, the subsurface stratigraphy encountered near the abutments on both sides of the existing bridge consists of a pavement structure and typically compact, cohesionless embankment fill overlying limestone bedrock at 2.3 m to 2.9 m depths. The very short term groundwater level was observed in the boreholes at approximately 2.1 m and 2.7 m depths below the existing ground.

### **8.1 Foundation Alternatives**

Based on the limited subsurface information, consideration may be given to supporting the new bridge using the following foundation types:

- Spread footings on bedrock
- Augered caissons (drilled shafts) socketted into bedrock

Discussions on feasible foundation alternatives are presented in the following paragraphs.

#### Spread Footings on Bedrock

Spread footings founded on bedrock are considered feasible to support the new bridge abutments and piers, given the shallow depths to bedrock at this site. In the river floodplain and flow channel, the footings will likely be constructed within cofferdam enclosures with dewatering measures. Temporary protection will also be required at highway grade.

#### Caissons (Drilled Shafts)

Drilled shafts socketted into bedrock can be considered as an alternative to spread footings. This is a feasible foundation option for the piers which can be designed to connect with the superstructure columns without pier caps. Depending on the construction methodology, this may



result in lesser requirements of dewatered cofferdams. There is currently no subsurface information at the pier.

Driven piles are not considered to be a feasible foundation system for this bridge due to the presence of shallow bedrock. If piles are to be used, it is possible that structural considerations require socketting into the bedrock (augered piles) rendering it less cost effective than other available options. As such, foundation recommendations for piles are not provided at this time.

### Recommended Foundations

From a foundation technical and cost effectiveness perspective, the preliminary foundation alternative for the Moira River bridge are spread footings founded on bedrock for the east and west abutments. Both spread footings and caissons can be considered for the centre pier. Borehole data is required at the proposed pier locations for selecting appropriate foundation alternatives.

## **8.2 Spread Footings on Bedrock**

For preliminary design, spread footings are feasible for the support of the proposed structure founded on bedrock at depths and elevations shown in Table 5.1. Boreholes were not drilled at the pier locations and therefore bedrock elevations are unknown.

### **8.2.1 Vertical Resistance**

Spread footings bearing on undisturbed limestone bedrock at the elevations quoted above may be designed for the following geotechnical resistance:

- Factored geotechnical resistance of 2,500 kPa at Ultimate Limit States (ULS)

The SLS condition will not govern design of footings founded on bedrock.

The values of the Factored Geotechnical Resistance at ULS were assessed assuming a Consequence Factor equal to 1 (Typical), and a Resistance Factor equal to 0.5 (Typical degree of understanding of the subsurface conditions), as per CHBDC 2019.

The bearing resistance is for vertical, concentric loading. In the case of eccentric or inclined loading, the bearing resistance must be adjusted as shown in the CHBDC (2019) Clauses 6.10.2 to 6.10.5.



The above recommendations are based on footings bearing on clean, undisturbed bedrock surface. All shattered and loosened bedrock fragments must be removed from the footprint of the footing and replaced with mass concrete fill of the same class and strength of the footing concrete. Where bedrock is lower than anticipated, the founding subgrade level should be raised using the same mass concrete fill. For sloping and undulating bedrock surface, the footing may step up or down across the width of the structure to accommodate changes in elevations of the top of bedrock.

### **8.2.2 Lateral Resistance**

For preliminary design, the horizontal resistance at the footing-rock interface may be assessed using a value of 0.7 for the ultimate coefficient of friction for concrete placed on undisturbed limestone bedrock.

If the frictional component is insufficient, the horizontal resistance may be increased by dowelling the footing into the rock mass. Dowels are considered to be comparatively short steel bars that may be assumed to provide only shear resistance. If vertical resistance in tension is required, rock anchors may be included in the design.

## **8.3 Caissons Socketted in Bedrock at Pier**

Caissons (drilled shafts) socketted into bedrock may be considered to support the pier in the river. There is currently no borehole information available for the pier. Upon the request of WSP, the following provides preliminary foundation recommendations for caisson design based on information obtained on land at the abutment boreholes. This is only a guide and has to be subjected to review and revision once additional borehole information for the pier from the investigation for detail design (Section 14) is available.

### **8.3.1 Axial Resistance**

It should be assumed that caisson geotechnical resistance is derived from rock sockets. The limestone bedrock encountered in the abutment boreholes is in a highly to moderately weathered state with random shale interbeds. It is assumed that the limestone at the pier location in the river would be more weathered and has more fractures. A minimum rock socket depth of 3 m is recommended for axial geotechnical resistance. The upper 1 m of bedrock has been neglected for estimating geotechnical resistance. It is important to note that the required depth of rock socket should be determined during detail design to satisfy base fixity, lateral load resistance and any other structural requirements. WSP advised that consideration is currently being given to a





single row of 1.8 m or 2.1 m diameter caissons for supporting the pier.

As a preliminary guide, a 1.8 m diameter and 3 m long socket within limestone at the pier may be designed using an axial geotechnical resistance of 10,000 kN at ULS. A 2.1 m diameter and 3 m long socket within limestone may be designed using an axial geotechnical resistance of 13,000 kN at ULS. The caisson base must be adequately cleaned to ensure mobilization of end bearing resistance.

The SLS condition does not govern design of caissons socketted into bedrock.

### **8.3.2 Lateral Resistance**

Lateral bridge loadings at the pier can be geotechnically resisted by the augered caissons through passive pressure developed along the embedded portion of the caissons below the bedrock surface.

The design of laterally loaded augered caissons must take into account such factors as the relative rigidity of the foundation elements to the surrounding rock, the fixity condition at the head of the caissons, the structural capacity of the caissons to withstand bending moments and shear, the rock resistance that can be mobilized and the maximum tolerable lateral deflection at the top of caisson.

Currently there is insufficient information for lateral resistance design at the pier caissons. Once more information is available during detail design, foundation recommendations for caisson lateral resistance design will be provided.

## **8.4 Frost Cover**

The design depth of frost penetration at this site is 1.4 m. However, frost penetration is not a design issue for footings bearing on bedrock or mass concrete fill placed on bedrock.

## **9.0 APPROACH EMBANKMENTS**

Based on currently available information, the existing embankment fills are up to the order of about 4 m in height with design slope inclinations of 2H : 1V. The west and east approach embankments will be widened to the north, parallel to the highway, to accommodate the new and



wider structure. These widenings will require placement of new fill. There is currently no subsurface information for the embankment widening areas.

It is recommended that all new slopes be designed to match the existing slope configuration. The forward and approach slopes should be designed with an inclination of 2H : 1V or flatter.

No global stability issues are anticipated for the approach fills at this site given the compact cohesionless fill overlying shallow bedrock, and provided that all surficial vegetation, organics and topsoil, soft/loosened or wet soils and debris are removed from the widening areas prior to fill placement.

It is recommended that all exposed slope surfaces be vegetated and seeded in accordance with current MTO practice. Erosion protection measures must be provided for the slopes.

Based on available information, the new fill will be in the order of 3 m to 4 m in height. Foundation settlement of the cohesionless subgrade due to the new fill is expected to take place as the fill is placed and be completed by the end of construction. The magnitude of post construction settlement due to compression of the embankment fill itself depends on the type of materials to be used, but is anticipated to be less than 25 mm.

## **10.0 ABUTMENT WALL BACKFILL AND LATERAL EARTH PRESSURES**

Backfill to the abutment walls should consist of free-draining granular material conforming to OPSS.PROV 1010 Granular A or B Type II specifications. 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 2019 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)
- $\gamma$  = 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.

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

**Table 10.1 – Lateral Earth Pressure Coefficients**

| Loading Condition          | Earth Pressure Coefficient (K)  |                               |   |                               |
|----------------------------|---|-------------------------------|---|-------------------------------|
|                            | OPSS Granular A or<br>Granular B Type II<br>$\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$ |                               | OPSS Granular B Type I<br>$\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$ |                               |
|                            | Horizontal<br>Backfill  | Sloping Backfill<br>(2H : 1V) | Horizontal<br>Backfill  | Sloping Backfill<br>(2H : 1V) |
| Active (Unrestrained Wall) | 0.27  | 0.40                          | 0.31  | 0.48                          |
| At-rest (Restrained Wall)  | 0.43  | 0.62                          | 0.47  | 0.70                          |
| Passive                    | 3.7   | -                             | 3.2   | -                             |

## 11.0 EXCAVATION AND GROUNDWATER CONTROL

All excavations must be carried out in accordance with OPSS.PROV 902 and the Occupational Health and Safety Act (OHSA). For the purposes of assessing excavation slope and temporary support requirements in compliance with the OHSA, the embankment fills are classified as Type



3 soils above water level and Type 4 soils below water level. River sediments are considered as Type 4 soils.

Excavation for pier construction could involve removal of river sediments, cobbles, boulders and loose rock fragments. Excavation for abutment construction could involve similar materials within the bridge widening areas, and embankment fills within the footprint of the existing bridge. Rock excavation for footing construction should be minimized. Minor rock excavation may be required at the foundation locations in order to prepare the founding surface. Where required, rock excavation should be carried out using methods that will avoid disturbing the bedrock below the founding elevation. Blasting should not be used for excavating bedrock.

Temporary excavation for footing construction will extend below the groundwater and river levels, and must be carried out in conjunction with temporary protection and cofferdams. Dewatering will be required to facilitate footing construction in the dry.

The design of the dewatering system that will be required is the responsibility of the Contractor. This design must take into account the maximum river level likely to occur during construction. For pier construction, a steel cofferdam set on bedrock might be appropriate. The cofferdams will be required to be seated on the exposed bedrock and exclude the river flow. Some form of concrete plug may need to be tremied in place at the bottom of the excavation prior to dewatering inside the cofferdam. Pumping from filtered sumps should be required in all cases.

Hydro-vac methods may be considered for cleaning the excavation base. Attempts should be made to facilitate pier footing construction in the dry. If complete dewatering inside the cofferdams is found to be impractical, pumped tremie concrete may be required to construct the pier footings.

## **12.0 SCOUR PROTECTION AND EROSION CONTROL**

Erosion and scour protection should be provided for the forward slopes at the river floodplain and surrounding the pier foundations. Such protection measures should be designed by qualified and experienced river hydraulics professionals.

Typically, rock protection should be provided over all surfaces with which the river water is likely to be in contact.



### **13.0 TEMPORARY PROTECTION SYSTEMS**

Temporary protection (shoring) systems will be required for construction of the new abutments in general accordance with OPSS.PROV 539. It is recommended that Performance Level 2 be specified.

Due to shallow bedrock, sheetpiles and driven H-piles do not appear to be suitable for use as temporary protection. A soldier pile and lagging system with H-piles socketted within bedrock should be feasible.

The selection and design of suitable temporary protection systems are the responsibilities of the Contractor. All shoring systems must be designed by a Professional Engineer experienced in such designs.

### **14.0 INVESTIGATION FOR DETAIL DESIGN**

There is no GEOCRETS information available for this site. The subsurface conditions depicted by the two boreholes of this preliminary investigation is insufficient and incomplete to be used for detail design of the new works. It will be necessary to carry out additional site investigation and field testing to support the preparation of foundation design recommendations for detail design of the replacement bridge and its approach fills. In particular, boreholes must be advanced for the new piers in the river and within the bridge widening areas. Although bedrock is reportedly visible in the river, the nature, quality and strength amongst other characteristics of the bedrock above and below founding depth are unknown. This information affects the design and construction of the foundations, and therefore must be obtained in support of detail design.

For detail design, it is recommended that Guidelines for MTO Foundation Engineering Services (Version 2.0 October 2020) be followed. For this bridge replacement, the minimum requirements are summarized as follows:

- 2 BHs at each foundation element advancing to a minimum of 3 m below refusal.
- Where bedrock is encountered, a minimum of 50 percent of the boreholes shall be cored for a minimum depth of 3 m.



- 1 BH at each bridge approach embankment within 20 m of the abutment, advancing to 3m into a competent stratum or 10 m below the base of the fill. Where bedrock is encountered, no coring is required.

The two boreholes advanced for this preliminary investigation can be incorporated into the detail investigation program. In order to address the widening areas within the floodplain and locations within the existing bridge footprint, and to satisfy the minimum requirements above, the borehole configuration for detail design should be as follows:

- 2 BHs at each abutment area within existing bridge for a total of 4 BHs.
- 1 BH at each abutment widening area in the floodplain for a total of 2 BHs.
- 2 BHs at the centre pier location for a total of 2 BHs.
- 1 BH at each approach area within existing bridge for a total of 2 BHs.
- 1 BH at each approach widening area in floodplain for a total of 2 BHs.

There should be a total of 12 boreholes, or 10 new boreholes.

## **15.0 CLOSURE**

Engineering analysis and preparation of this foundation design report were carried out by Ms. R. Palomeque Reyna, P.Eng. The report was reviewed by Dr. Sydney Pang, P.Eng. and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.



THURBER ENGINEERING LTD.



Rocío Palomeque Reyna, P.Eng.  
Geotechnical Engineer



Sydney Pang, P.Eng.  
Associate, Senior Foundation Engineer



P.K. Chatterji, P.Eng.  
Review Principal, Designated MTO Contact



## **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 <sup>(1)</sup> '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           |

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


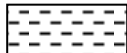



 Water Level  
 Shear Strength Determination by Pocket Penetrometer

- (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<br>W <sub>L</sub> < 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.<br>(W <sub>L</sub> < 30%). |
|                      |   | CI           | Inorganic clays of medium plasticity, silty clays.<br>(30% < W <sub>L</sub> < 50%).   |
|                      |   | OL           | Organic silts and organic silty-clays of low plasticity.  |
|                      | SILTS AND CLAYS<br>W <sub>L</sub> > 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>  |                   |
|---------------------------------------|---|---|-------------------|
| <b>Fresh (FR)</b>                     | No visible signs of weathering.   |   |                   |
| <b>Fresh Jointed (FJ)</b>             | Weathering limited to the surface of major discontinuities.   |  | CLAYSTONE         |
| <b>Slightly Weathered (SW)</b>        | Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material. |  | SILTSTONE         |
| <b>Moderately Weathered (MW)</b>      | Weathering extends throughout the rock mass, but the rock material is not friable.                            |  | SANDSTONE         |
| <b>Highly Weathered (HW)</b>          | Weathering extends throughout the rock mass and the rock is partly friable.                                   |  | COAL              |
| <b>Completely Weathered (CW)</b>      | 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>                        |  |                       |             |              |  |
|-------------------------------------|--|-----------------------|-------------|--------------|--|
| Total Core Recovery: (TCR)          | Core recovered as a percentage of total core run length.   | Weak                  | 5.0 to 25.0 | 750 to 3,500 | Can be peeled by a pocket knife with difficulty                                |
| Solid Core Recovery: (SCR)          | Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run. | Very Weak             | 1.0 to 5.0  | 150 to 750   | Can be peeled by a pocket knife, crumbles under firm blows of geological pick. |
| 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.        | Extremely Weak (Rock) | 0.25 to 1.0 | 35 to 150    | Indented by thumbnail  |
| 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 MR 20-01

1 OF 1

METRIC

W.P. 4193-15-00 LOCATION Moira River Bridge, MTM NAD83-9 N 4 895 216.1 E 233 339.6 ORIGINATED BY GA  
DIST Eastern HWY 401 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN  
DATUM Geodetic DATE 2020.11.12 - 2020.11.12 LATITUDE 44.193447 LONGITUDE -77.393956 CHECKED BY RPR

| SOIL PROFILE  |   |            | SAMPLES |      |            | GROUND WATER<br>CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION<br>RESISTANCE PLOT  |  |  |  | UNIT<br>WEIGHT<br><br>γ<br><br>kN/m <sup>3</sup> | REMARKS<br>&<br>GRAIN SIZE<br>DISTRIBUTION<br>(%)<br><br>GR SA SI CL |   |
|---------------|---|------------|---------|------|------------|----------------------------|-----------------|--|--|--|--|--|--|---|
| ELEV<br>DEPTH | DESCRIPTION   | STRAT PLOT | NUMBER  | TYPE | "N" VALUES |                            |                 | SHEAR STRENGTH kPa<br>○ UNCONFINED      + FIELD VANE<br>● QUICK TRIAXIAL    × LAB VANE |  |  |  |  |  | WATER CONTENT (%)<br>w <sub>P</sub> w      w <sub>L</sub> |
| 94.9          | GROUND SURFACE  |            |         |      |            |                            |                 |  |  |  |  |  |  |   |
| 0.0           | ASPHALT: (175mm)  |            |         |      |            |                            |                 |  |  |  |  |  |  |   |
| 0.2           | SAND and GRAVEL<br>Compact  |            | 1       | SS   | 20         |                            |                 |  |  |  |  |  |  |   |
| 94.2          | Brown   |            |         |      |            |                            |                 |  |  |  |  |  |  |   |
| 0.7           | Moist<br>(FILL)   |            | 2       | SS   | 21         |                            |                 |  |  |  |  |  |  |   |
|               | Silty SAND, some clay, trace gravel<br>Compact  |            |         |      |            |                            |                 |  |  |  |  |  |  |   |
| 93.4          | Dark Brown to Brown   |            |         |      |            |                            |                 |  |  |  |  |  |  |   |
|               | Moist<br>(FILL)   |            | 3       | SS   | 14         |                            |                 |  |  |  |  |  |  |   |
| 1.5           | SAND and GRAVEL, trace silt<br>Compact  |            |         |      |            |                            |                 |  |  |  |  |  |  |   |
|               | Brown   |            |         |      |            |                            |                 |  |  |  |  |  |  |   |
| 92.6          | Moist<br>(FILL)   |            |         |      |            |                            |                 |  |  |  |  |  |  |   |
| 2.3           | Coring started at 2.3m  |            |         |      |            |                            |                 |  |  |  |  |  |  |   |
|               | LIMESTONE, highly to moderately<br>weathered, grey to dark grey, with<br>shale interbeds, laminated, horizontally<br>bedded: (Simcoe Group)   |            | 1       | RUN  |            |                            |                 |  |  |  |  |  |  |   |
|               | Highly fractured zone from 2.30m to<br>2.44m  |            |         |      |            |                            |                 |  |  |  |  |  |  |   |
|               | Horizontal fractures at 2.44m, 2.52m,<br>2.59m, 2.60m, 2.64m, 2.78m, 2.84m,<br>3.05m, 3.23m, 3.24m, 3.52m and<br>3.62m  |            |         |      |            |                            |                 |  |  |  |  |  |  |   |
|               | Moderately weathered  |            | 2       | RUN  |            |                            |                 |  |  |  |  |  |  |   |
|               | Shale interbeds:<br>2.57m - 2.59m (20mm)<br>2.81m - 2.83m (20mm)<br>2.96m - 2.97m (10mm)<br>3.21m - 3.23m (20mm)<br>3.93m - 3.95m (20mm)<br>4.26m - 4.27m (10mm)<br>4.41m - 4.43m (10mm)<br>4.72m - 4.73m (10mm)<br>4.75m - 4.80m (50mm)<br>5.17m - 5.22 m (50mm)<br>Horizontal fractures at 3.81m, 3.85m,<br>3.87m, 4.38m, 4.75m, 4.80m, 4.85m,<br>5.25m and 5.26m |            |         |      |            |                            |                 |  |  |  |  |  |  |   |
| 89.6          | END OF BOREHOLE AT 5.3m.<br>BOREHOLE OPEN TO 5.3m AND<br>WATER LEVEL AT 2.1m UPON<br>COMPLETION OF DRILLING.<br>BOREHOLE BACKFILLED WITH<br>BENTONITE HOLEPLUG TO 0.6m,<br>CONCRETE TO 0.2m, THEN<br>ASPHALT COLD PATCH TO<br>SURFACE.  |            |         |      |            |                            |                 |  |  |  |  |  |  |   |
| 5.3           |   |            |         |      |            |                            |                 |  |  |  |  |  |  |   |

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to  
Sensitivity

20  
15  
10

(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No MR 20-02

1 OF 1

METRIC

W.P. 4193-15-00 LOCATION Moira River Bridge, MTM NAD83-9 N 4 895 224.3 E 233 466.7 ORIGINATED BY GA  
 DIST Eastern HWY 401 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN  
 DATUM Geodetic DATE 2020.11.11 - 2020.11.11 LATITUDE 44.193532 LONGITUDE -77.392367 CHECKED BY RPR

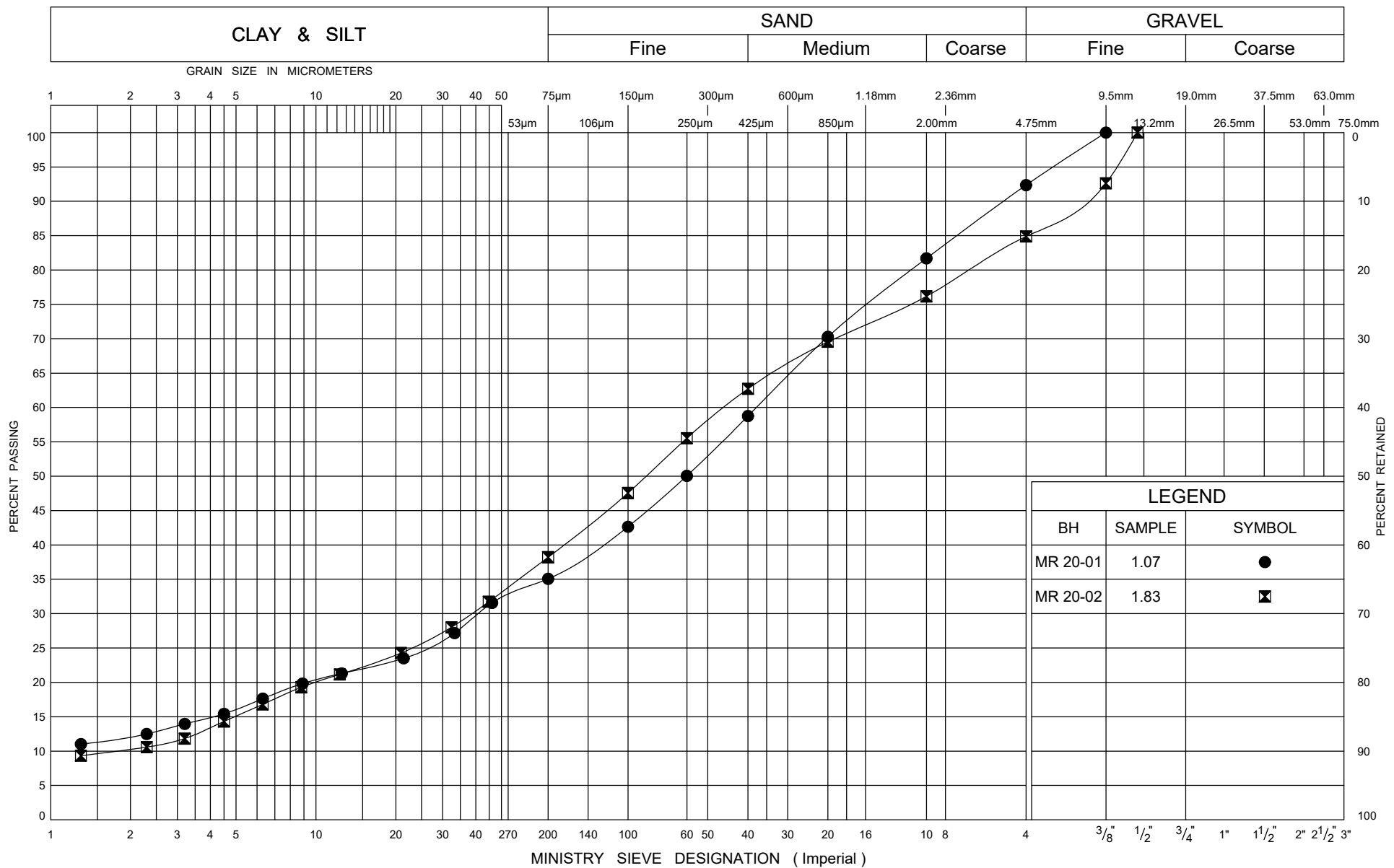
| SOIL PROFILE  |  |            | SAMPLES |      |            | GROUND WATER<br>CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION<br>RESISTANCE PLOT  |    |    |    | PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT |  |  | UNIT<br>WEIGHT<br><br>γ<br><br>kN/m <sup>3</sup> | REMARKS<br>&<br>GRAIN SIZE<br>DISTRIBUTION<br>(%) |    |    |    |   |
|---------------|--|------------|---------|------|------------|----------------------------|-----------------|--|----|----|----|---|--|--|--|---|----|----|----|---|
| ELEV<br>DEPTH | DESCRIPTION  | STRAT PLOT | NUMBER  | TYPE | "N" VALUES |                            |                 | SHEAR STRENGTH kPa<br>○ UNCONFINED      + FIELD VANE<br>● QUICK TRIAXIAL      × LAB VANE |    |    |    | WATER CONTENT (%)<br>w <sub>p</sub> w      w <sub>L</sub>     |  |  |  |   |    |    |    |   |
| 94.4          | GROUND SURFACE   |            |         |      |            |                            |                 | 20   | 40 | 60 | 80 | 100   |  |  |  |   | GR | SA | SI | CL  |
| 0.0<br>94.1   | ASPHALT: (250mm)   |            |         |      |            |                            |                 |  |    |    |    |   |  |  |  |   |    |    |    |   |
| 0.3           | SAND and GRAVEL, trace silt<br>Compact<br>Brown<br>Moist<br>(FILL)   |            | 1       | SS   | 21         |                            |                 |  |    |    |    |   |  |  |  |   |    |    |    |   |
|               |  |            | 2       | SS   | 11         |                            |                 |  |    |    |    |   |  |  |  |   |    |    |    |   |
| 92.9          |  |            |         |      |            |                            |                 |  |    |    |    |   |  |  |  |   |    |    |    |   |
| 1.5           | Silty SAND, some gravel, trace to<br>some clay<br>Very Loose<br>Brown to Grey<br>Wet<br>(FILL)   |            | 3       | SS   | 3          |                            |                 |  |    |    |    |   |  |  |  |   |    |    |    | 15 47 28 10   |
| 92.0          |  |            |         |      |            |                            |                 |  |    |    |    |   |  |  |  |   |    |    |    |   |
| 2.4           | SAND, some silt, some clay,<br>occasional clay pockets<br>Compact<br>Brown<br>Wet<br>(FILL)<br>Coring started at 2.9m  |            | 4       | SS   | 12         |                            |                 |  |    |    |    |   |  |  |  |   |    |    |    | 0 72 15 13  |
| 91.5          |  |            |         |      |            |                            |                 |  |    |    |    |   |  |  |  |   |    |    |    |   |
| 2.9           | LIMESTONE, highly to moderately<br>weathered, grey to dark grey, with<br>shale interbeds, laminated, horizontally<br>bedded: (Simcoe Group)  |            | 1       | RUN  |            |                            |                 |  |    |    |    |   |  |  |  |   |    |    |    | RUN #1<br>TCR=100%<br>SCR=93%<br>RQD=52%<br>UCS=94MPa<br>(Average)  |
|               | Rubble zone from 3.27m to 3.37m  |            |         |      |            |                            |                 |  |    |    |    |   |  |  |  |   |    |    |    |   |
|               | Horizontal fractures at 2.94m, 2.96m,<br>3.15m, 3.21m, 3.22m, 3.40m, 3.68m,<br>3.72m, 3.85m, 3.89m and 3.97m   |            |         |      |            |                            |                 |  |    |    |    |   |  |  |  |   |    |    |    |   |
|               | Moderately weathered<br>Vertical fracture at 4.4m  |            | 2       | RUN  |            |                            |                 |  |    |    |    |   |  |  |  |   |    |    |    | UCS=30MPa at<br>3.5m  |
|               | Shale interbeds:<br>3.43m - 3.45m (20mm)<br>3.62m - 3.65m (30mm)<br>3.76m - 3.80m (40mm)<br>3.85m - 3.89m (40mm)<br>4.26m - 4.29m (30mm)<br>4.32m - 4.33m (10mm)<br>4.45m - 4.46m (10mm)<br>4.59m - 4.60m (10mm)<br>4.70m - 4.76m (60mm)<br>4.91m - 4.93m (20mm)<br>5.09m - 5.18m (90mm)<br>5.40m - 5.41m (10mm)<br>5.56m - 5.57m (10mm)<br>5.86m - 5.87m (10mm) |            |         |      |            |                            |                 |  |    |    |    |   |  |  |  |   |    |    |    | RUN #2<br>TCR=100%<br>SCR=100%<br>RQD=96%<br>UCS=67MPa<br>(Average) |
| 88.4          |  |            |         |      |            |                            |                 |  |    |    |    |   |  |  |  |   |    |    |    |   |
| 6.0           | Horizontal fractures at 4.40m, 4.61m,<br>4.62m, 4.72m, 4.92m, 4.96m, 5.05m,<br>5.16m, 5.27m, 5.73m and 5.88m   |            |         |      |            |                            |                 |  |    |    |    |   |  |  |  |   |    |    |    |   |
|               | END OF BOREHOLE AT 6.0m.<br>BOREHOLE OPEN TO 6.0m AND<br>WATER LEVEL 2.7m UPON<br>COMPLETION OF DRILLING.<br>BOREHOLE BACKFILLED WITH<br>BENTONITE HOLEPLUG TO 0.6m,<br>CONCRETE TO 0.3m, THEN<br>ASPHALT COLD PATCH TO<br>SURFACE.  |            |         |      |            |                            |                 |  |    |    |    |   |  |  |  |   |    |    |    |   |

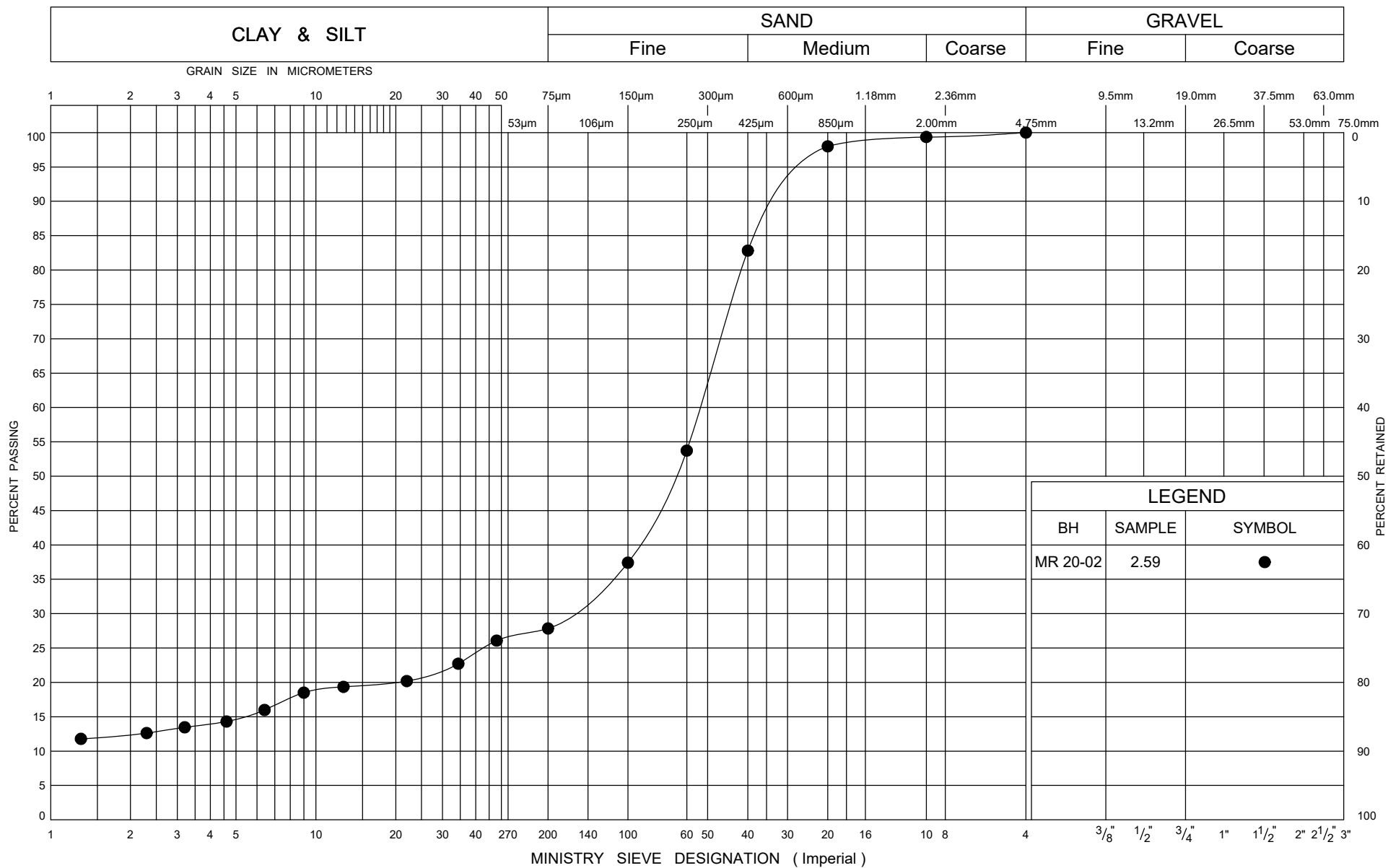
ONTMT4S2 MTO-11566.GPJ 2017TEMPLATE(MTO).GDT 1/5/21



## **Appendix B**

### **Geotechnical Laboratory Test Results**







**THURBER ENGINEERING LTD.****POINT LOAD TEST SHEET****ASTM D5731-08**

**Job No:** 11566  
**Client:** WSP Canada Group Ltd.  
**Project Name:** Hwy 401 Belleville  
**Core Size:** NQ **BH No :** MR20-01

**Date Drilled:** 12-Nov-20  
**Date Tested:** 18-Nov-20  
**Tester:** MP  
**Reviewed by:** WM

| Test No. | Run No. | Depth (m) | Axial or Diametral | Gauge (MPa) | Diameter (mm) | Length (mm) | $I_{s(50)}$ (MPa) | UCS (MPa) | Rock Type | Rock Strength<br>(after Hoek & Brown, 1997) |
|----------|---------|-----------|--------------------|-------------|---------------|-------------|-------------------|-----------|-----------|---|
| 1        | 1       | 2.7       | A                  | 17.6        | 47.2          | 50.5        | 5.7               | 138.0     | Limestone | Very Strong                                 |
| 2        | 1       | 3.2       | D                  | 3.1         | 47.3          | 81.5        | 1.3               | 30.5      | Limestone | Medium Strong                               |
| 3        | 1       | 3.2       | A                  | 13.3        | 47.3          | 50.1        | 4.4               | 104.9     | Limestone | Very Strong                                 |
| 4        | 1       | 3.6       | A                  | 17.9        | 47.3          | 52.0        | 5.7               | 136.8     | Limestone | Very Strong                                 |
| 5        | 2       | 4.0       | A                  | 9.9         | 47.3          | 51.0        | 3.2               | 77.0      | Limestone | Strong                                      |
| 6        | 2       | 4.1       | D                  | 9.8         | 47.3          | 69.3        | 4.0               | 96.9      | Limestone | Strong                                      |
| 7        | 2       | 4.8       | A                  | 14.0        | 47.3          | 53.0        | 4.4               | 105.6     | Limestone | Very Strong                                 |
| 8        | 2       | 5.1       | D                  | 4.3         | 47.6          | 71.5        | 1.8               | 42.6      | Limestone | Medium Strong                               |
| 9        | 2       | 5.2       | A                  | 10.9        | 47.3          | 52.0        | 3.5               | 83.3      | Limestone | Strong                                      |
| 10       |         |           |                    |             |               |             |                   |           |           |   |
| 11       |         |           |                    |             |               |             |                   |           |           |   |
| 12       |         |           |                    |             |               |             |                   |           |           |   |
| 13       |         |           |                    |             |               |             |                   |           |           |   |
| 14       |         |           |                    |             |               |             |                   |           |           |   |
| 15       |         |           |                    |             |               |             |                   |           |           |   |
| 16       |         |           |                    |             |               |             |                   |           |           |   |
| 17       |         |           |                    |             |               |             |                   |           |           |   |
| 18       |         |           |                    |             |               |             |                   |           |           |   |
| 19       |         |           |                    |             |               |             |                   |           |           |   |
| 20       |         |           |                    |             |               |             |                   |           |           |   |
| 21       |         |           |                    |             |               |             |                   |           |           |   |
| 22       |         |           |                    |             |               |             |                   |           |           |   |
| 23       |         |           |                    |             |               |             |                   |           |           |   |
| 24       |         |           |                    |             |               |             |                   |           |           |   |
| 25       |         |           |                    |             |               |             |                   |           |           |   |
| 26       |         |           |                    |             |               |             |                   |           |           |   |
| 27       |         |           |                    |             |               |             |                   |           |           |   |
| 28       |         |           |                    |             |               |             |                   |           |           |   |
| 29       |         |           |                    |             |               |             |                   |           |           |   |
| 30       |         |           |                    |             |               |             |                   |           |           |   |
| 31       |         |           |                    |             |               |             |                   |           |           |   |
| 32       |         |           |                    |             |               |             |                   |           |           |   |
| 33       |         |           |                    |             |               |             |                   |           |           |   |
| 34       |         |           |                    |             |               |             |                   |           |           |   |

\* It is ideal to perform axial test on core specimens with D/L ratio of  $1.1 \pm 0.1$

\* Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

\* Diametral Test should have  $0.7 \times D$  on either side of test point.

\* Correlation factor to obtain UCS values is 24.



# ASTM D5731-08

|               |           |
|---------------|-----------|
| Date Drilled: | 11-Nov-20 |
| Date Tested:  | 18-Nov-20 |
| Tester:       | MP        |
| Reviewed by   | WM        |

[illegible]



## **Appendix C**

### **Rock Core Photographs**

PHOTOGRAPHS OF ROCK CORES

BOREHOLE MR20-01  
RUNS 1 AND 2

TOP

Run 1

Run 2



BOTTOM

| Run # | Depth (m) |
|-------|-----------|
| 1     | 2.3 – 3.8 |
| 2     | 3.8 – 5.3 |

PHOTOGRAPHS OF ROCK CORES

BOREHOLE MR20-02  
RUNS 1 AND 2

TOP

Run 1

Run 2



BOTTOM

| Run # | Depth (m) |
|-------|-----------|
| 1     | 2.9 – 4.4 |
| 2     | 4.4 – 5.9 |



**Appendix D**

**Selected Site Photographs**



**Photo 1- Moira River Bridge , southwest side, looking south**



**Photo 2- Moira River Bridge , northeast (Site visit in 2018)**





**Photo 3- Moira River Bridge , north abutment, east side (Site visit in 2018)**

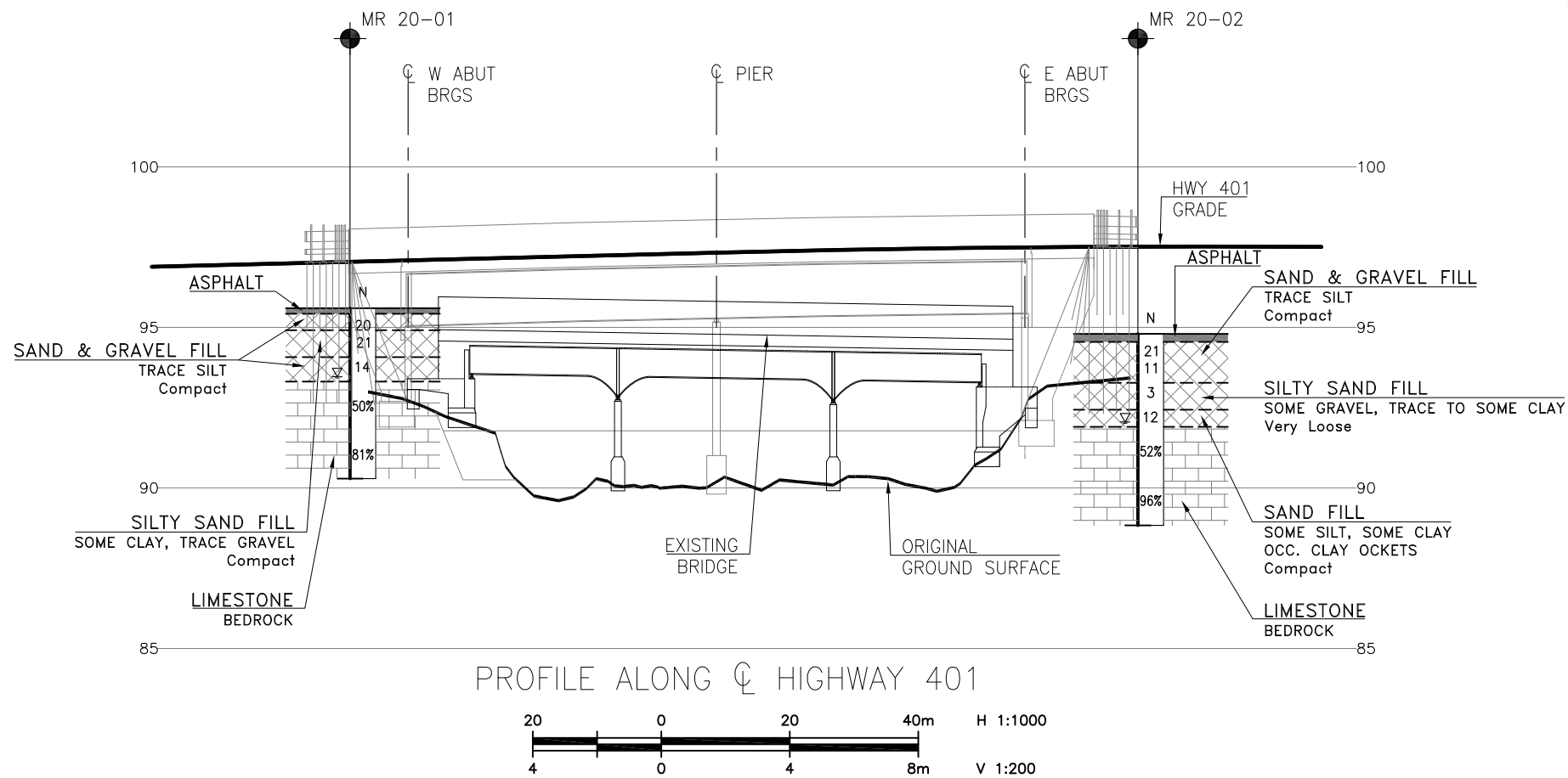
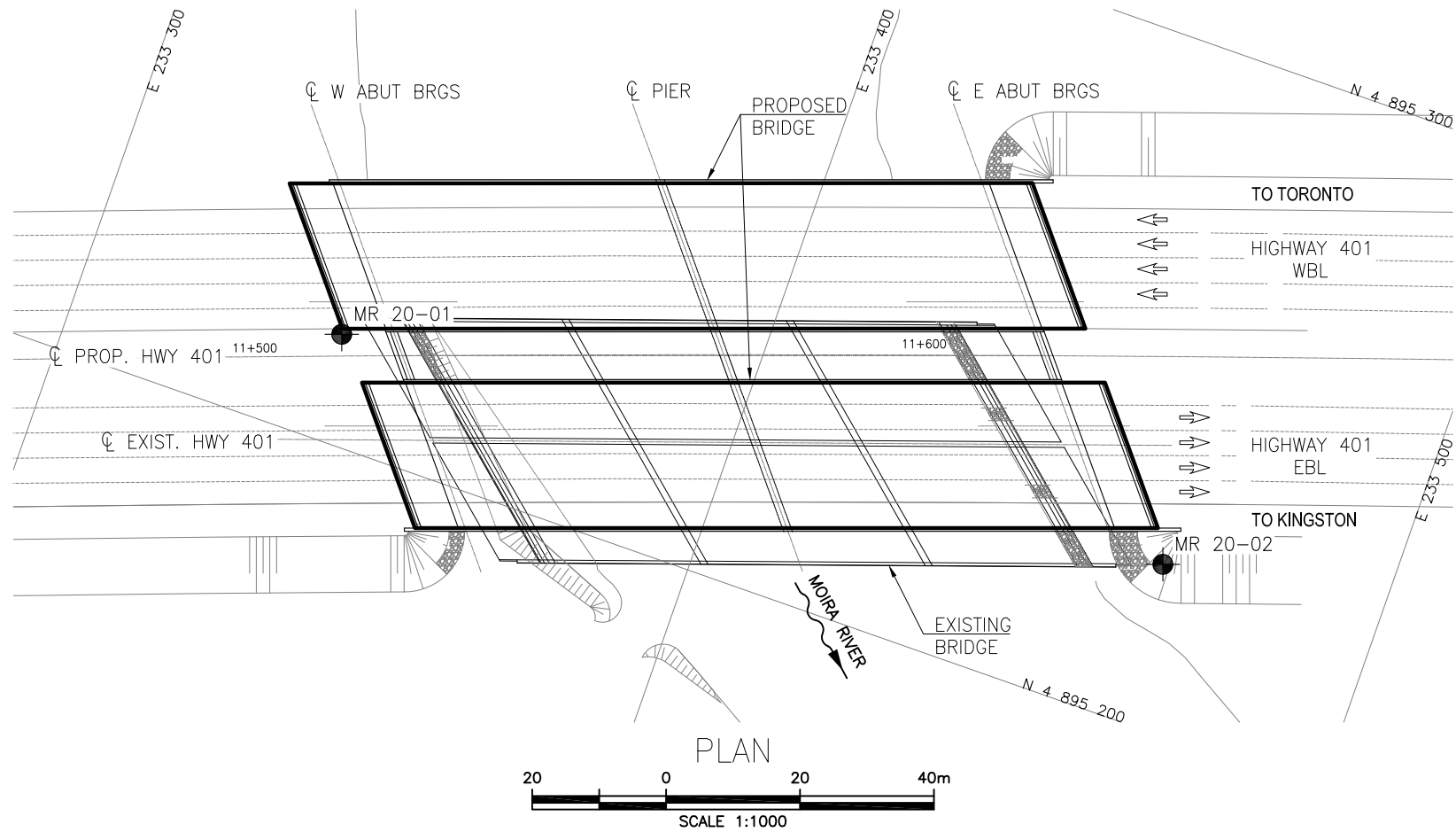


**Photo 4- Moira River Bridge, south abutment, east side (Site visit in 2018)**



## **Appendix E**

### **Borehole Locations and Soil Strata Drawing**



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



CONT No  
WP No 4193-15-00

HIGHWAY 401 WIDENING  
MOIRA RIVER BRIDGE  
REPLACEMENT  
BOREHOLE LOCATIONS AND SOIL STRATA



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

| NO       | ELEVATION | NORTHING    | EASTING   |
|----------|-----------|-------------|-----------|
| MR 20-01 | 94.9      | 4 895 216.1 | 233 339.6 |
| MR 20-02 | 94.4      | 4 895 224.3 | 233 466.7 |

-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.
- Coordinate system is MTM NAD 83 Zone 9.

GEOCRES No. 31C-311

| REVISIONS | DATE   | BY     | DESCRIPTION |
|-----------|--------|--------|-------------|
| DESIGN    | RPR    | CHK    | SKP         |
| DRAWN     | AN     | CHK    | RPR         |
| CODE      | LOAD   | DATE   | JUL 2021    |
| SITE      | 11-162 | STRUCT | DWG 1       |