



**THURBER** ENGINEERING LTD.

**PRELIMINARY  
FOUNDATION INVESTIGATION AND DESIGN REPORT  
HIGHWAY 37 UNDERPASS REPLACEMENT  
SITE NO. 11-163  
HIGHWAY 401 WIDENING EA  
BELLVILLE, ONTARIO  
G.W.P. 4193-15-00**

**GEOCRES NO. 31C-310**

**Latitude: 44.194885°  
Longitude: -77.387874°**

**Report**

to

**WSP**

Date: July 5, 2021  
File: 11566



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**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 replacement of the Highway 401 underpass bridge at Highway 37 in the geographic township of Thurlow – Municipality of Belleville, Ontario. It is noted that the new bridge will be located some 200 m to the east of the existing bridge as part of an interchange reconstruction.

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, borehole logs, 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 abutment 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 drawing and a desktop study for the existing bridge. The titles of these documents are as follows:

- 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 1).
- Draft Desktop Study Report, Preliminary Foundation Investigation and Design, Replacement of Highway 401 Underpass at Highway 37 (Site No. 11-163), Highway 401 Widening, Belleville, Ontario prepared by Thurber Engineering, File: 11566, dated May 24, 2018 (Reference 2).
- Archive drawing, Thurlow Township Bridge No. 5, Highway No. 37 over Highway 401, Dated May 1955 (Reference 3).

## **2.0 SITE DESCRIPTION**

The existing Highway 401 underpass at Highway 37 is located some 700 m east of the Highway 401 and Highway 62 interchange in Belleville, Ontario.

The existing Highway 37 underpass consists of a single span, rigid frame reinforced concrete structure with a span length of about 35 m and a width of about 17 m. The structure was originally constructed in 1956. The structure accommodates 4 traffic lanes (2 northbound and 2 southbound) and carries Highway 37 over Highway 401. The last rehabilitation of the existing underpass was completed in 2011.

The proposed underpass will be located some 200 m east of the existing underpass. Reconstruction of the interchange to accommodate the new bridge will include realignment of a section of Highway 37 just north of Highway 401 and realignment of several ramps. The new structure will carry two through lanes in each direction and also turning/auxiliary lanes.

The natural terrain in the vicinity of the bridge is generally flat. Immediate lands around the underpass are vegetated with grass and trees. The Moira River is located approximately 400 m west of the new bridge site. The river floodplain is mainly vegetated with grass, shrubs and some trees.



Select photographs of the site are included 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 borehole investigation and field testing program for this site were carried out between November 11 and 12, 2020, and consisted of drilling and sampling two (2) boreholes, designated as Boreholes H37 20-01 and H37 20-02. Borehole H37 20-1 was drilled near the northeast corner of the proposed north abutment, and Borehole H37 20-2 near the southwest corner of the proposed south abutment.

Boreholes H37 20-01 and H37 20-02 were terminated at 4.6 m depth (Elevations 91.0 and 90.2). Both boreholes were advanced into limestone bedrock by coring 3.1 m in each borehole. The approximate locations of the boreholes are shown on the Borehole Locations Plan and Soil Strata Drawing in Appendix E. The records of borehole sheets are provided in Appendix A.

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.

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 laboratory for further examination and testing.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. A standpipe piezometer (19 mm diameter) was installed and the slotted screen enclosed in filter sand in each borehole to permit groundwater level monitoring. The details of the piezometers are shown in Table 3.1.

**Table 3.1 – Borehole Completion Details**

Foundation Unit	Borehole	Borehole Depth / Base Elevation (m)	Piezometer Tip Depth/ Elevation (m)	Completion Details
North Abutment	H37 20-01	4.6 / 91.0	4.6 / 91.0	Piezometer with 1.5 m slotted screen installed within sand filter from 4.6 m to 2.7 m, bentonite holeplug from 2.7 m to 0.3 m, then sand from 0.3 m to ground surface.
South Abutment	H37 20-02	4.6 / 90.2	1.2 / 93.6	Piezometer with 0.7 m slotted screen installed within sand filter from 1.2 m to 0.3 m, then bentonite holeplug to ground surface.

Upon request by the MTO, the two piezometers are left in place to assist in the detail design. Once the detail design is completed, these two piezometers, along with other piezometers to be installed during detail investigation, should then be decommissioned by the detail design foundation consultant in general accordance with O.Reg. 903.

#### **4.0 LABORATORY TESTING**

The recovered soil samples were subjected to visual identification (VI) and to natural moisture content determination. A fill sample was subjected to grain size distribution analyses (sieve)



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 the results are summarized 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 stratigraphy are presented on the Record of Borehole sheets included in Appendix A, and on the Borehole Locations and Soil Strata drawing 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.

In general, the subsurface stratigraphy encountered at the two boreholes consists of granular embankment fill overlying limestone bedrock at 1.5 m depth below ground surface. A short term groundwater level was measured at 2.4 m depth below the existing ground.

More detailed descriptions of the individual stratum are presented below.

### **5.1 Topsoil**

A layer of topsoil was encountered surficially in both boreholes with measured thicknesses of 100 mm to 150 mm.

The topsoil thickness may vary between and beyond the borehole locations, and the data is not intended for the purpose of estimating quantities.

### **5.2 Embankment Fill**

Embankment fill was encountered below the topsoil in both boreholes, and consisted of layers of brown and grey sand and gravel, sandy gravel to sand containing trace to some silt and clay. Occasional limestone fragments were encountered within the fill in Borehole H37 20-01. The



thickness of the embankment fill was 1.3 m to 1.4 m. The depth to the base of the embankment fill was at 1.5 m (Elevations 94.1 to 93.3) in both boreholes.

The SPT 'N' values recorded in the embankment fill ranged from 11 to 39 blows per 0.3 m of penetration indicating a compact to dense condition. An SPT 'N' value of 50 blows per 0.025 m of penetration, possibly due to the limestone fragments, was measured in Borehole H37 20-01 just above the bedrock surface. The natural moisture contents measured on samples of the cohesionless fill ranged from 4 percent to 16 percent.

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

Soil Particle	Embankment Gravel Fill (Percent)
Gravel	68
Sand	24
Silt and Clay	8

### 5.3 Limestone Bedrock

The soils described above were found to be underlain by limestone bedrock of the Simcoe Group. The limestone is typically fossiliferous, argillaceous and laminated, and varies from medium to thickly bedded. The limestone is grey to dark grey in colour and described as moderately weathered. These cores contain frequent shale interbeds typically ranging between 10 mm and 60 mm in thickness with occasional interbeds ranging between 70 mm and 120 mm. Rock core photos are presented in Appendix C.

Depth and elevation 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)
North Abutment	H37 20-01	1.5 *	94.1
South Abutment	H37 20-02	1.5 *	93.3



\* Proved by coring below augered depth.

Bedrock cores were recovered using NQ sized coring equipment. TCR measured for the rock cores was 100 percent throughout.

RQD values ranged from 60 to 87 percent indicating a fair to good rock quality. FI of the rock, expressed as fractures per 0.3 m of core, ranged from 0 to 5, and greater than 10 where the value corresponded to a rubble/broken zone in Run 1 of Borehole H37 20-01.

Unconfined compressive strengths interpreted from point load tests conducted on selected rock cores typically varied from 62 MPa to 137 MPa indicating a strong to very strong rock. There are zones within the bedrock where the unconfined compressive strengths ranged from 36 to 50MPa, indicating a medium strong rock. Results of point load tests conducted on the rock core samples 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 measured 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)	
H37 20-01	November 12, 2020	1.5 *	94.1	Open borehole Piezometer
	November 18, 2020	2.4	93.2	
H37 20-02	November 11, 2020	1.3 *	93.5	Open borehole Piezometer
	November 18, 2020	Dry	-	

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

The values shown in Table 5.2 are short term readings, and 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.



## **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.



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**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 a replacement Highway 401 underpass at Highway 37 in Belleville, Ontario.

The proposed bridge will be located some 200 m to the east of the existing bridge as part of the proposed reconstruction of the interchange at Highway 401 and Highway 37. The new bridge will carry a realigned Highway 37 over Highway 401. Several interchange ramps will also be realigned.

Based on a preliminary/draft General Arrangement (GA) drawing, dated October 2020, provided by WSP, the new bridge will be a two-span structure supported on two integral abutments and one pier at the median. Subsequent information from WSP indicates that the length of the new bridge is proposed to be 73 m, with a south span of 37.5 m and a north span of 35.5 m. There will be a 6.0 m long approach slab at each of the abutments. The width of the new bridge deck will be about 30.45 m.

It is proposed that each of the north and south abutments be supported on a single row of H-piles and the pier be supported on spread footings. The Highway 37 grade within the structure limits will vary from approximate Elevations 104.2 to 103.7, north to south. The Highway 401 grade is



at approximate Elevation 96.2, resulting in approach embankments in the order of 7.5 m to 8.0 m in height. The GA drawing indicates that the top of the footing at the pier will be at approximate Elevation 94.5.

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 at the site consists of cohesionless embankment fill overlying limestone bedrock at 1.5 m depth below ground surface. A short term groundwater level was measured at 2.4 m depth below the existing ground.

### **8.1 Foundation Alternatives**

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

- Spread footings on bedrock
- Steel H-piles socketted into bedrock
- 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 pier. The footings would likely be founded at or close to the groundwater level. However, groundwater control systems will be required to handle seepage from seams and fractures in the limestone bedrock, perched water from the fills, and accumulation of surface runoff and precipitation in order to facilitate footing construction in the dry. Temporary protection will also be required, especially at the pier location where adjacent live traffic is anticipated during construction. This foundation option precludes the use of integral abutments.

#### Drilled Steel H-Piles Socketted into Bedrock

For supporting an integral abutment, drilled steel H-piles arranged in a single row socketted into



bedrock may be used. The required depth of rock sockets depends on structural requirements for base fixity and lateral geotechnical resistance. Consideration may be given to a sufficiently large pre-drilled (cored) hole in rock for installing each pile. Alternatively, a rock trench can be excavated by mechanical means for installing an entire row of piles. The cost effectiveness of these alternatives should be assessed by the designers.

Driven piles are not considered to be a feasible foundation option for this bridge due to the presence of shallow bedrock.

#### 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 pier which can be designed to connect with the superstructure columns without a pier cap. Given the shallow bedrock, however, this is not considered to be a cost effective foundation option, which will also preclude the use of integral abutments. As such, foundation recommendations for caissons are not provided at this time.

#### Recommended Foundations

From a foundation technical and cost effectiveness perspective, spread footings founded on bedrock is a feasible option for all the foundation elements. Since integral abutments are considered, the recommended foundation alternatives would be steel H-piles socketted into bedrock at both abutments, and spread footings on bedrock at the pier.

### **8.2 Spread Footings on Bedrock**

For preliminary design, spread footings may be founded on bedrock at depths and elevations shown in Table 5.1. Boreholes were not drilled at the pier location and therefore bedrock elevation is unknown.

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 factored geotechnical resistance at ULS was 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.

#### **8.2.1 Footing Lateral Resistance on Bedrock**

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 must be included in the design.

#### **8.3 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.

#### **8.4 Steel H-Piles Socketted into Bedrock**

For integral abutment design, consideration should be given to supporting the abutments on steel H-piles set in pre-drilled holes with sockets formed within the limestone bedrock. The portion of a pile above rock should accommodate a 3 m long CSP just below the abutment stem. The rock

sockets should be cored using appropriate rotary coring equipment. The socket base should be cleaned of loose and shattered rock. The pile should then be lowered into the socket, and the remaining space between the pile and the surrounding rock be grouted with minimum 30 MPa concrete. Alternatively, rock trenches can be excavated by mechanical means. Excavated vertical faces and base inside a rock trench can be rugged which must be adequately cleaned. Similar concrete placement is also required in a rock trench after all the piles for one abutment is installed. The required rock socket depth should be governed by base fixity, lateral load resistance and any other structural requirements.

Table 5.1 presents the top of bedrock elevations for preliminary design of the H-piles and the rock socket depths. It is recommended that a minimum rock socket depth of 2 m be used.

#### 8.4.1 Axial Pile Resistance

For a steel H-pile grouted within a 600 mm nominal diameter socket of 2 m in bedrock, or grouted within a 2 m deep trench in bedrock, Table 8.1 below presents geotechnical resistances that may be used for preliminary design.

**Table 8.1 Design H-Pile Axial Resistances**

Foundation Unit	Steel H-Pile Section			
	HP 310 x 110		HP 360 x 132	
	Factored Axial Pile Resistance at ULS (kN)	Axial Pile Resistance at SLS (kN)	Factored Axial Pile Resistance at ULS (kN)	Axial Pile Resistance at SLS (kN)
Both Abutments	2,000	N/A	2,400	N/A

N/A: Not applicable since SLS condition does not govern design of piles in rock

The values of the factored axial pile resistance at ULS were assessed based on static analysis assuming a Consequence Factor equal to 1 (Typical), and a geotechnical resistance factor equal to 0.4 (Typical degree of understanding of the subsurface conditions), as per CHBDC 2019. The SLS condition does not govern design of piles socketted within bedrock.



#### 8.4.2 Lateral Pile Resistance

Given the shallow bedrock and the overlying variable fill, it is recommended that the lateral geotechnical resistance for the piles be derived from the rock sockets only.

For rock sockets formed within undisturbed limestone bedrock, the ultimate passive force that can be mobilized by the embedded portion of a socket is as follows:

$$P_{P(ult)} = (1 + 1.4 z / D) \sigma_{rm} \cdot D \cdot L \quad (\text{kN}) \quad \text{for } z \leq 3D$$

$$P_{P(ult)} = 5 \sigma_{rm} \cdot D \cdot L \quad (\text{kN}) \quad \text{for } z > 3D$$

where

$z$	=	depth of socket below rock surface	(m)
$D$	=	socket diameter	(m)
$\sigma_{rm}$	=	1.5 MPa (equivalent rock mass strength within rock socket).	

### 9.0 RETAINED SOIL SYSTEMS (RSS) WALL

The use of RSS walls in conjunction with false integral abutments is a feasible option. The proposed wall retaining heights may be up to about 6.5 m under the bridge deck and gradually decreasing to zero at the ends of the wingwalls. The length of the wingwalls will be 17m except for the northwest corner where it will be 12 m.

The performance of a RSS mass is dependent on, amongst other factors, the characteristics of its foundation. Failure to provide an adequate foundation may lead to settlement and distortion of the RSS and, in severe cases, to possible failure of the system. The foundation of the entire RSS mass must be considered, i.e. from the face of the wall to the furthest extent of the reinforcement. Based on available information, it is recommended that the RSS mass be founded on bedrock contacted at approximate Elevations 94.1 and 93.3 at the north and south abutments, respectively.

The RSS mass should be founded on a minimum 0.3 m thick of engineered fill consisting of OPSS.PROV 1010 Granular A compacted to 100 percent of its Standard Proctor Maximum Dry Density (SPMDD) at a moisture content within 2 percent of optimum. The engineered pad must laterally extend at least 500 mm beyond the footprint of the RSS mass and levelling strips.



The RSS block and wall footings founded on such compacted granular pad, which is itself bearing on undisturbed bedrock, may be designed for the following geotechnical resistances:

- Factored geotechnical resistance of 600 kPa at Ultimate Limit States (ULS).
- Geotechnical Resistance of 400 kPa at Servicability Limit State (SLS)

The entire block of reinforced earth must be designed against various modes of failure including sliding and overturning. Sliding resistance along the base of the wall may be estimated using an ultimate friction coefficient of 0.55 for an engineered granular fill subgrade.

The proprietary RSS system must meet the MTO's specifications for performance and appearance. The RSS supplier/designer may specify more stringent criteria or other requirements related to the particular design. The internal stability of the RSS wall should be analyzed by the supplier/designer of the proprietary product selected for this site.

RSS walls for the bridge abutments should be designed with reference to the latest version of the MTO RSS Design Guidelines published by the Engineering Standards Branch.

Global stability issues are not anticipated for the RSS walls founded on bedrock. However, this must be analysed and confirmed after the locations and detail configurations of the walls are finalized.

## **10.0 APPROACH EMBANKMENTS**

Based on the preliminary GA drawing dated October 2020, the finished grade level of the Highway 37 is at approximate Elevations 104.2 to 103.7, from north to south abutment. Highway 401 grade is at approximate Elevation 96.2, resulting in approach embankments of 7.5 m to 8.0 m high.

The north and south approach embankments will be constructed in open areas and straddling existing ramp fills to accommodate the new structure alignment. Placement of granular or earth fill will be required. The approach and side slopes should be designed for an inclination of 2H : 1V or flatter.

No global stability issues are anticipated for the approach fills at this site provided the approved new fill is placed and compacted in accordance with OPSS.PROV 206 and OPSS.PROV 501,



and provided that all surficial vegetation, organics and topsoil, soft/loosened or wet soils and debris are removed from the proposed embankment areas prior to fill placement.

It is recommended that all exposed slope surfaces be vegetated and seeded in accordance with current MTO practice with reference to OPSS.PROV 804. Erosion protection measures must be provided for the slopes.

Based on available information, the new fill will be up to 7.5 m to 8.0 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 not anticipated to exceed 25 mm if the new fill is placed and compacted as outlined above.

## 11.0 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 11.1. 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 11.1 – Lateral Earth Pressure Coefficients**

Loading Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Backfill	Sloping Backfill (2H : 1V)	Horizontal Backfill	Sloping Backfill (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	-

## 12.0 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.

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.

Given the anticipated shallow excavations and the general layout of the site, it is anticipated that any excavation required to be carried out for construction of the new bridge will not extend below the groundwater level. However, seepage or perched water from the approach fills as well as surface runoff and precipitation are to be expected. In addition, concentrated seepage may be experienced from seams or fractures in the limestone bedrock. All surface runoff should be diverted away from excavations.



The Contractor should be prepared to pump from properly filtered sumps to remove any seepage water or surface water collecting in an excavation. Unwatering must remain operational and effective until the excavation is backfilled.

The design of any dewatering system that may be required is the responsibility of the Contractor.

Where required, construction will need to be carried out in conjunction with temporary protection.

### **13.0 TEMPORARY PROTECTION SYSTEMS**

Temporary protection (shoring) systems will be required for construction of the new abutments and pier 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 into 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 ADJACENT STRUCTURES AND BURIED UTILITIES**

It is recommended that the exact locations of any existing utilities that are present in the vicinity of the work areas be established by the designer, and compared with the extent of the potential work zones related to the proposed construction.

The utilities should not be undermined or damaged during construction of the new bridge. Relocation of, and/or special protective measures for, some or all of these affected utilities may be required.

### **15.0 INVESTIGATION FOR DETAIL DESIGN**

There is no GEOCRE 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 new bridge and its approach fills.

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.
- BHs shall be advanced at each end of the RSS wall and at a maximum longitudinal spacing of 50 m. BHs shall be advanced to refusal or 3 m into a competent stratum that will provide sufficient geotechnical bearing resistance for the RSS foundation.

The two boreholes advanced for this preliminary investigation can be incorporated into the detail investigation program. The two piezometers are left in place for the detail investigation, after which all piezometers will need to be decommissioned by the detail design foundation consultant. The minimum borehole configuration for detail design should be as follows:

- 2 BHs at each abutment area within existing bridge for a total of 4 BHs.
- 2 BHs at the pier location for a total of 2 BHs.
- 1 BH at each approach area for a total of 2 BHs.
- 1 BH at the end of each RSS wingwall for a total 4 BHs.

There should be a total of 12 boreholes, or 10 new boreholes taking into consideration the 2 existing boreholes.



## **16.0 CLOSURE**

Engineering analysis and preparation of the 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


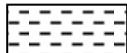



$C_{pen}$  Shear Strength Determination by Pocket Penetrometer

- (1) SPT 'N' Value      Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.
- (2) DCPT      Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

# UNIFIED SOILS CLASSIFICATION

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

## EXPLANATION OF ROCK LOGGING TERMS

<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 H37 20-01

1 OF 1

METRIC

W.P. 4193-15-00 LOCATION HWY 37 Underpass, MTM NAD83-9 N 4 895 412.1 E 233 829.6 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 44.195258 LONGITUDE -77.387854 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
							20	40	60	80	100						
95.6	GROUND SURFACE																
0.0	TOPSOIL: (150mm)																
0.2	GRAVEL, some sand to sandy, trace silt and clay Compact to Very Dense Brown to Grey Moist (FILL)		1	SS	22												
			2	SS	39												
94.1	Occasional limestone fragments Coring started at 1.5m		3	SS	50/												
1.5	LIMESTONE, moderately weathered, grey to dark grey, with shale interbeds, laminated, horizontally bedded: (Simcoe Group) Highly broken zones from 1.50m to 1.70m and 1.77 to 1.82m  Horizontal fractures at 1.71m, 1.74m, 1.86m, 1.88m, 1.94m, 2.02m, 2.10m, 2.12m, 2.15m, 2.28m, 2.35m, 2.41m, 2.46m, 2.58m, 2.82m and 2.94m Shale interbeds: 1.83m - 1.87m (40mm) 2.05m - 2.10m (50mm) 2.14m - 2.15m (10mm) 2.38m - 2.46m (80mm) 3.24m - 3.27m (30mm) 3.39m - 3.42m (30mm) 3.68m - 3.70m (20mm) 3.79m - 3.84m (50mm) 4.12m - 4.15m (30mm) 4.48m - 4.52m (40mm)		1	RUN	0.025												
			2	RUN													
91.0	Horizontal fractures at 3.15m, 3.29m, 3.35m, 3.45m, 3.80m, 3.84m, 3.89m, 3.98m, 4.12m, 4.25m, 4.40m, 4.49m and 4.56m																
4.6	END OF BOREHOLE AT 4.6m. BOREHOLE OPEN TO 4.6m AND WATER LEVEL AT 1.5m UPON COMPLETION OF DRILLING.  Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.  WATER LEVEL READINGS DATE DEPTH(m) ELEV.(m) 2020.11.18 2.4 93.2																

ONTMT4S2 MTO-11566.GPJ 2017TEMPLATE(MTO).GDT 1/14/21

## METRIC

[illegible]

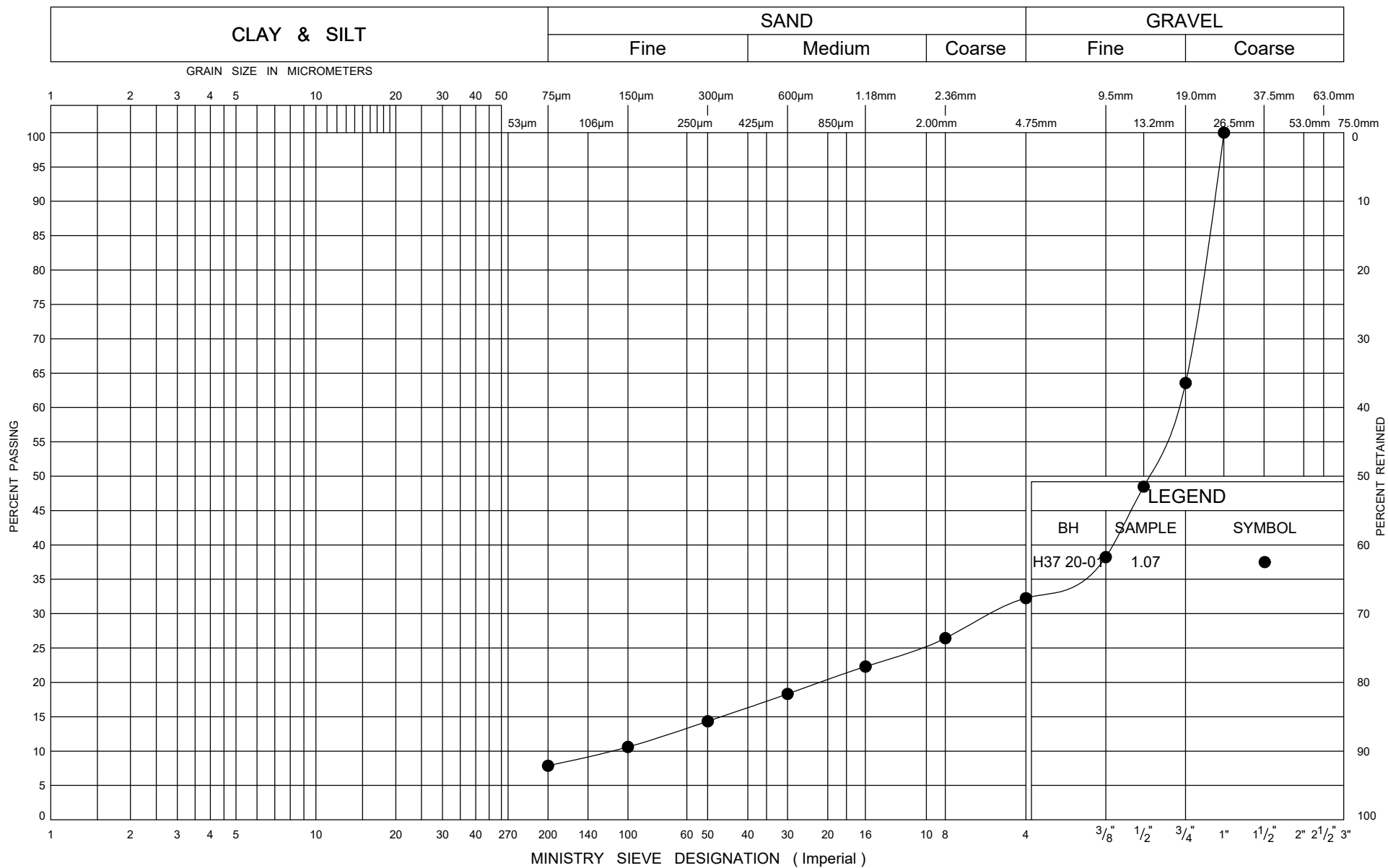
CONTMT4S2 MTO-11566.GPJ 2017TEMPLATE(MTO).GDT 1/14/21

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



## **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 : H3720-01

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

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 (after Hoek & Brown, 1997)
1	1	1.9	A	9.3	47.3	51.4	3.0	71.4	Limestone	Strong
2	1	2.0	D	8.2	47.4	70.4	3.4	80.7	Limestone	Strong
3	1	2.4	A	13.1	47.2	53.8	4.1	97.7	Limestone	Strong
4	1	2.6	D	9.1	47.4	70.9	3.7	89.6	Limestone	Strong
5	1	2.9	A	10.2	47.3	51.4	3.3	78.8	Limestone	Strong
6	2	3.3	A	12.7	47.3	56.1	3.8	91.8	Limestone	Strong
7	2	4.0	A	6.3	47.4	49.9	2.1	49.9	Limestone	Medium Strong
8	2	4.5	A	9.8	47.5	48.5	3.3	78.6	Limestone	Strong
9	2	4.4	D	4.7	47.7	68.8	1.9	46.4	Limestone	Medium Strong
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\* 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.



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 : H3720-02

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

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 (after Hoek & Brown, 1997)
1	1	1.8	A	8.8	47.3	51.1	2.8	67.9	Limestone	Strong
2	1	2.0	D	4.0	47.4	72.5	1.7	39.9	Limestone	Medium Strong
3	1	2.3	A	16.5	47.3	49.6	5.5	131.0	Limestone	Very Strong
4	1	2.8	A	17.2	47.3	49.2	5.7	137.3	Limestone	Very Strong
5	1	2.8	D	9.6	47.6	66.5	3.9	94.5	Limestone	Strong
6	2	3.3	A	7.9	47.3	50.1	2.6	62.3	Limestone	Strong
7	2	4.0	A	13.9	47.3	49.7	4.6	109.9	Limestone	Very Strong
8	2	4.0	D	3.6	47.4	66.5	1.5	36.0	Limestone	Medium Strong
9	2	4.5	A	9.2	47.2	50.2	3.0	72.6	Limestone	Strong
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- \* 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.



## **Appendix C**

### **Rock Core Photographs**

PHOTOGRAPHS OF ROCK CORES

BOREHOLE H37 20-01  
RUNS 1 AND 2

TOP



Run 1

Run 2

BOTTOM

Run #	Depth (m)
1	1.5 – 3.1
2	3.1 – 4.6

PHOTOGRAPHS OF ROCK CORES

BOREHOLE H37 20-02  
RUNS 1 AND 2

TOP

Run 1

Run 2



BOTTOM

Run #	Depth (m)
1	1.5 – 3.0
2	3.0 – 4.6



**Appendix D**  
**Selected Site Photographs**



**Photo 1- Highway 37 Underpass , southwest side, looking south**



**Photo 2- Highway 37 Underpass, north abutment (Site visit on 2018)**

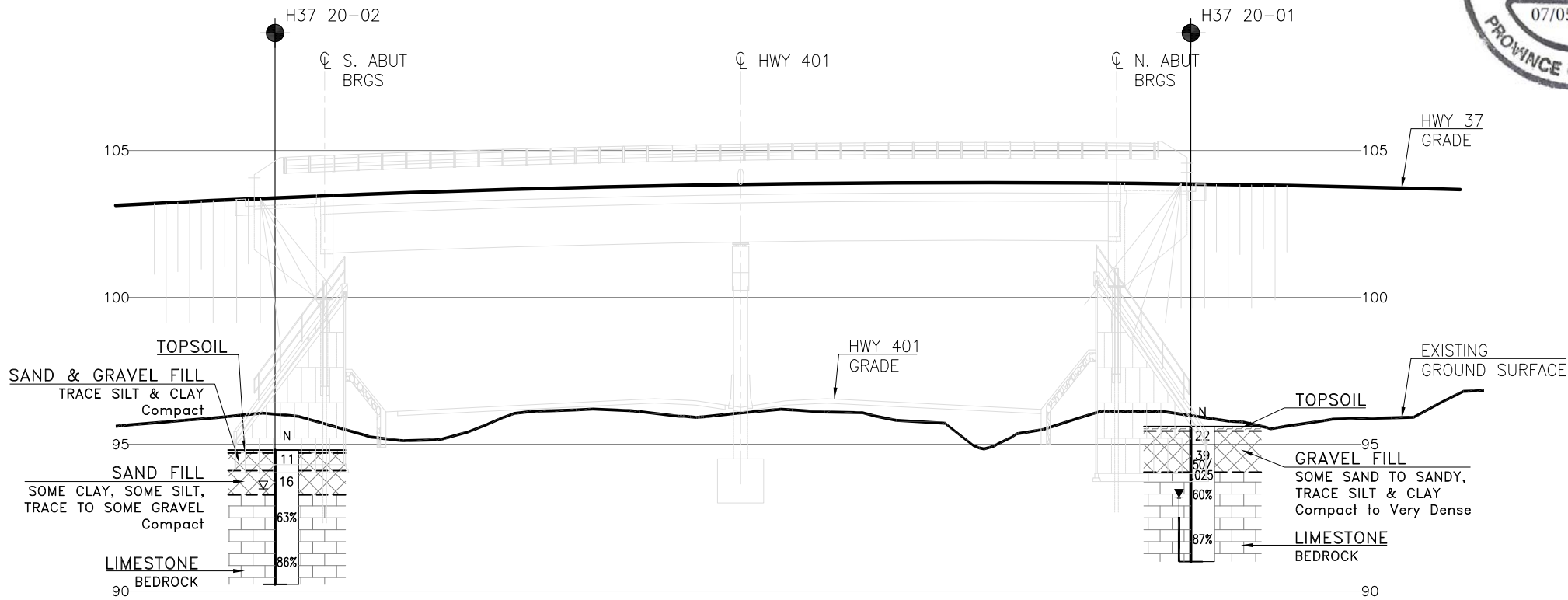
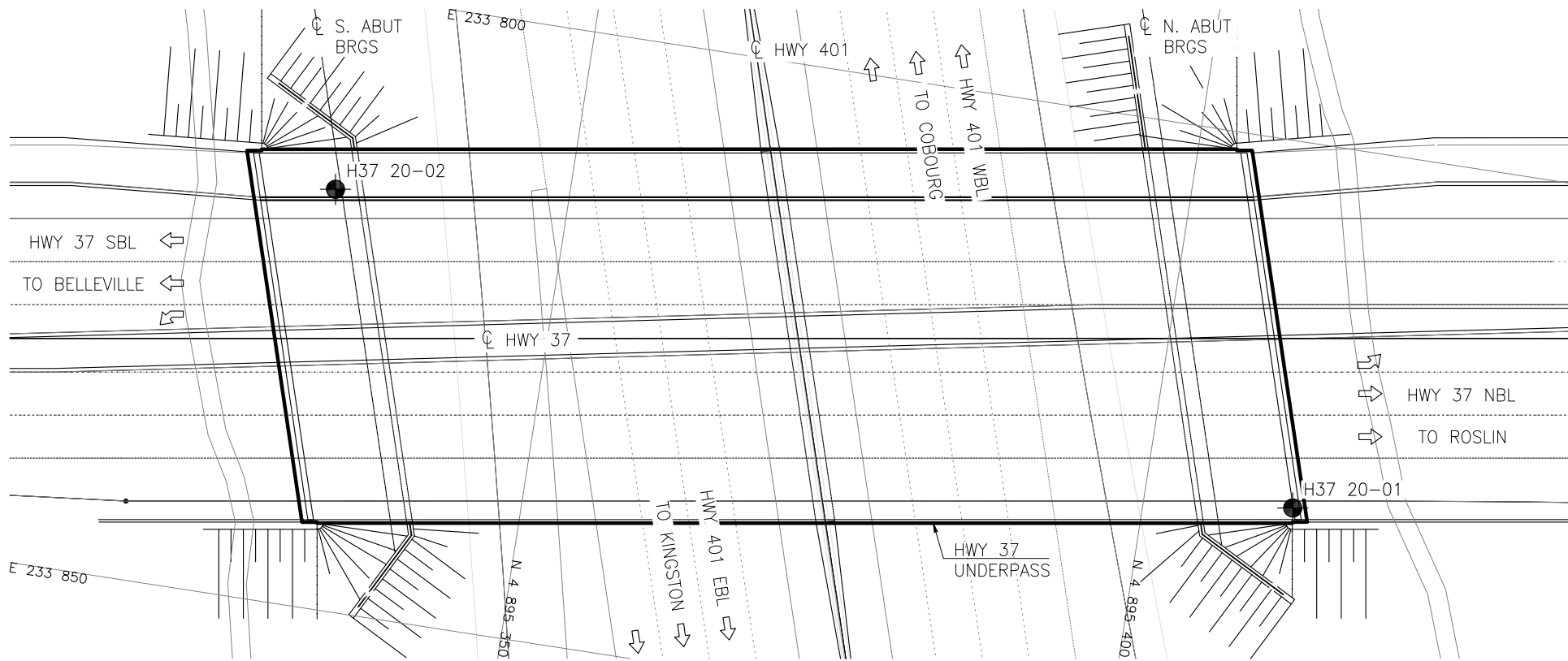


**Photo 3- Highway 37 Underpass, south abutment (Site visit on 2018)**

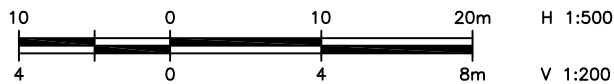


## **Appendix E**

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



PROFILE ALONG  $\phi$  HWY 37



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



CONT No  
WP No 4193-15-00

HIGHWAY 401 WIDENING  
HIGHWAY 37  
REPLACEMENT UNDERPASS  
BOREHOLE LOCATIONS AND SOIL STRATA



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
H37 20-01	95.6	4 895 412.1	233 829.6
H37 20-02	94.8	4 895 331.1	233 815.9

-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-310

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	RPR	CHK	SKP
DRAWN	AN	CHK	RPR
CODE	LOAD	DATE	JUL 2021
SITE	11-163	STRUCT	DWG 1