THURBER ENGINEERING LTD.

PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
POTTER CREEK CULVERT REPLACEMENT
HIGWAY 401 WIDENING EA
SITE No. 11-316/C
BELLEVILLE, ONTARIO
G.W.P. 4193-15-00

GEOCRES NO. 31C-309

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Report

to

WSP

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THURBER

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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 Potter Creek culvert replacement, located approximately 1 km west of the Highway 401 at Wallbridge-Loyalist Road interchange in the geographic township of Thurlow, Municipality of Belleville, Ontario.

The purpose of this investigation was to explore the subsurface conditions at the culvert location and, based on the data obtained, to provide a borehole location plan, stratigraphic profile, records of boreholes, laboratory test results, and a written description of the subsurface conditions. A model of the subsurface conditions was developed to describe the geotechnical conditions influencing the preliminary design of the culvert replacement.

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.

2.0 SITE DESCRIPTION

The existing culvert is located approximately 1 km west of the Highway 401 at Wallbridge-Loyalist Road interchange in the City of Belleville, Ontario.

The Highway 401 corridor addressed in this project generally runs in an east to west orientation along relatively flat terrain. There are commercial and institutional developments in the vicinity of the interchange. At Potter Creek culvert, the lands to the east and west are predominantly of agricultural usage.

Available information indicates that the existing culvert is a 3.6 m x 1.5 m concrete open footing culvert with a length of about 52 m. The Highway 401 grade at the existing culvert is at approximate Elevation 104.8 m. The embankment fill at the site is approximately 3.8 m in height. Potter Creek flows southerly through the culvert at the site. The width of the creek in the vicinity of the culvert was estimated to be about 4 m based on site observations.

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 for this site were carried out on November 9 and 10, 2020, and consisted of drilling and sampling two (2) boreholes, designated as Boreholes PC20-01 and PC20-02. Boreholes PC20-01 and PC20-02 were drilled from the Highway 401 WBL and EBL shoulders, near the culvert inlet and outlet areas, and terminated at 7.3 m and 7.6 m depths (Elevations 96.7 and 96.3), respectively. Both boreholes were advanced into limestone bedrock by coring 3.0 m in each borehole. The approximate locations of both 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.

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 track-mounted drill rig with hollow stem augers having an outside diameter of 108 mm (4-1/4-inch). 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 (47.6 mm inside diameter) 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 and upon completion. Boreholes were backfilled upon completion of drilling in general conformance with O.Reg. 903 as amended by O.Reg.128/03.

4.0 LABORATORY TESTING

The recovered soil samples were subjected to visual identification (VI) and to natural moisture content determination. Selected samples were subjected to grain size distribution analyses (sieve and/or hydrometer), and Atterberg Limits 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 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 and rock conditions may vary between and beyond the borehole locations.

In general, the subsurface stratigraphy encountered in the boreholes consists of typically compact granular embankment fill overlying native firm to very stiff clayey silt till and very dense silty sand till. Limestone bedrock was encountered below the till layers at 4.3 m to 4.6 m depths. The short term groundwater level was measured at approximately 4.3 m to 4.6 m depths below the existing ground.

More detailed descriptions of the individual stratum are presented below.

5.1 Fill

Sand and gravel road base was encountered surficially in both boreholes, which were located on the gravel portion of the Highway 401 shoulders. The thickness of of the sand and gravel ranged between 700 mm and 900 mm.

Embankment fill was encountered below the road base and consisted of brown gravelly sand to sand containing trace to some silt and clay. The thickness of the gravelly sand to sand fill ranged between 1.7 m and 2.1 m. The depth to the base of the embankment fill was at 2.4 m and 3.0 m (Elevations 101.6 and 100.9) in Boreholes PC20-01 and PC20-02, respectively.

The SPT 'N' values recorded in the fill ranged from 10 to 30 blows per 0.3 m of penetration, indicating a compact condition. The natural moisture contents measured on samples of the cohesionless fill generally varied from 2 percent to 16 percent.

The results of grain size analyses conducted on samples of the gravelly 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 Gravelly Sand Fill (Percent)	Embankment Sand Fill (Percent)
Gravel	24	8
Sand	64	83
Silt and Clay	12	9

5.2 Clayey Silt Till

A layer of brown to grey clayey silt till with sand containing some gravel was contacted below the fill at 2.4 m depth in Borehole PC20-01 located close to the inlet area. The thickness of the clayey silt till was 1.9 m. The base of this till was at 4.3 m depth (Elevation 99.7).

SPT 'N' values for the clayey silt till were 6 and 20 blows per 0.3 m of penetration indicating a firm to very stiff consistency. An SPT 'N' value of 50 blows with no penetration was measured at the base of the till, where the spoon was bouncing on the bedrock surface. Moisture contents measured in the clayey silt till were 19 percent and 23 percent.

The results of grain size distribution analyses carried out on a sample of the clayey silt till are presented on the Record of Borehole sheets included in Appendix A. A grain size distribution curve of the tested sample is presented on Figure B3 Appendix B. The results of the grain size distribution analyses are summarized below:

Soil Particle	Clayey Silt Till (Percent)
Gravel	11
Sand	44
Silt	30
Clay	15

The results of Atterberg Limits tests conducted on a sample of clayey silt till are presented on the Record of Borehole sheets in Appendix A, and illustrated in Figure B5 of Appendix B. The results are summarized as follows:

Index Property	Percentage (%)
Liquid Limit	35
Plasticity Index	19

The results of the Atterberg Limits testing indicate that the clayey silt has a low plasticity with a group symbol of CL.

Glacial tills inherently contain cobbles and boulders.

5.3 Silty Sand Till

A deposit of brown silty sand till containing some gravel and trace clay was contacted below the fill at 3.0 m in Borehole PC20-02. Limestone fragments were noted near the base of the till. The thickness of the silty sand till was 1.6 m. The base of this till was at 4.6 m depth (Elevation 99.3).

The SPT 'N' values recorded in the silty sand till were 50 to 60 blows for less than 0.3 m of penetration, indicating a very dense state. The spoon was observed to be bouncing. The natural moisture content measured on a sample of the silty sand till was 5 percent.

The results of grain size distribution analyses carried out on selected samples of the silty sand till are presented on the Record of Borehole sheets included in Appendix A. Grain size distribution curves of samples tested are presented on Figure B4 Appendix B. The results of the grain size distribution analyses are summarized below:

Soil Particle	Silty Sand Till (Percent)
Gravel	14
Sand	51
Silt	26
Clay	9

Glacial tills inherently contain cobbles and boulders.

5.4 Limestone Bedrock

The soils described above were found to be underlain by limestone bedrock 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 moderately weathered. These cores contain frequent shale interbeds typically ranging between 10 mm and 80 mm in thickness with occasional interbeds up to 100 and 110 mm. Rock core photos are presented in Appendix C.

Depths and elevations of the top of bedrock encountered in this 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)
Close to North side (Inlet)	PC20-01	4.3 *	99.7
Close to South side (Outlet)	PC20-02	4.6 *	99.3

* Proved by coring below augered depth.

Bedrock cores were collected using NQ sized coring equipment. TCR in the bedrock was between 95 percent and 100 percent.

RQD values typically ranged from 75 to 90 percent indicating a good to excellent rock quality. An RQD value of 45 percent was measured in Run 1 of Borehole PC20-01 which indicated a poor rock quality and the presence of a highly fractured zone. FI of the rock, expressed as fractures per 0.3 m of core, ranged from 0 to 6 except for a value of greater than 10 corresponding to the highly fractured zone.

Unconfined compressive strengths interpreted from point load tests conducted on selected rock cores typically varied from 52 MPa to 167 MPa indicating a strong to very strong rock. Point load tests conducted on four rock core samples from depths ranging from 5.2 m to 6.6 m correlated to unconfined compressive strength values ranging from 6.3 MPa to 21 MPa indicating weak zones. Results of point load tests conducted on the rock core samples are included in Appendix B.

5.5 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)	
PC20-01	November 10, 2020	4.3 *	99.7	Open borehole
PC20-02	November 9, 2020	4.6 *	99.3	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) drawing provided by WSP dated October 2020 indicates that the high water level (HWL 50 years) at Potter Creek was measured at Elevation 102.48.

The values shown in Table 5.2 are very 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 the replacement of the existing Potter Creek culvert, located approximately 1 km west of the Highway 401 at Wallbridge-Loyalist Road interchange in the City of Belleville, Ontario.

The existing Potter Creek Culvert is a cast-in-place open footing concrete culvert of 3.1 m in width and 1.5 m in height. The length of the culvert is about 52 m. The inlet and outlet invert levels of the culvert opening are at approximate Elevations 101.5 and 101.0, respectively. The base of the existing footings are at approximate Elevation 100.4. Based on the current boreholes, the existing footings appear to be founded on the glacial tills. The culvert was built in 1956 and there is no record of previous major rehabilitation. The most recent biennial inspection (August 2015) noted spalling/delamination on the barrel soffit as well as disintegration of the barrel at the inlet and outlet.

Based on a draft General Arrangement (GA) drawing dated October 2020 provided by WSP, the project requirements involve replacement of the existing culvert with a precast concrete rigid frame supported on cast-in-place concrete footings to be located along the existing culvert alignment. The opening of the new culvert will have dimensions of 2.0 m high and 7.0 m wide. The length of the new culvert will be about 63.3 m to accommodate the highway widening. The GA drawing indicates that each footing is proposed to be 1 m thick and 1.8 m wide founded at approximate Elevation 100. The base of the flow channel is lined with a 400 mm thick rock protection. The GA drawing also shows that the existing culvert will be removed.

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 CULVERT FOUNDATIONS

In general, the subsurface stratigraphy encountered in the boreholes consists of typically compact granular embankment fill overlying native firm to very stiff clayey silt till and very dense silty sand till. Limestone bedrock was encountered below the till layers at 4.3 m to 4.6 m depths (Elevations 99.3 to 99.7). The short term groundwater level was measured at approximately 4.3m to 4.6 m depths below the existing ground (Elevations 99.3 to 99.7).

The limestone bedrock surface may step up or down along the footing alignments. Given the shallow bedrock, it is recommended that the culvert footings be founded on bedrock provided all other relevant requirements are satisfied. Minor rock excavation may be required during preparation of the rock subgrade.

8.1 Culvert Replacement Options

Based on the above, several common culvert types that may be considered for the culvert replacement at this site are listed below:

- Concrete box (closed) culvert
- Concrete open frame culvert on strip footings
- Circular pipe culvert

Discussions on feasible culvert alternatives are presented in the following paragraphs. A preferred culvert type from a foundations perspective is also recommended.

Concrete box (closed) culvert

Provided environmental and other requirements can be satisfied, a closed box culvert consisting of precast concrete segments may be considered under typical circumstances. These segments can be installed rapidly with less potential for disturbance of the founding subgrade during installation. A segmental box structure can accommodate some potential differential settlement along the culvert axis. At this site, however, the presence of shallow and potentially undulating bedrock pose the risk of rock protrusions above the founding elevation that may require excavation.

Concrete open frame culvert on strip footings

Concrete, open frame, culvert is considered feasible at this site. It will likely satisfy environmental and other requirements. The preliminary GA indicates that an open frame culvert supported by strip footings is being proposed. Based on the two current boreholes, these footings may be designed to be founded on bedrock. The proposed cast-in-place strip footings can better accommodate potential rock protrusions that require excavation. Segmental precast rigid frame, if used, is anticpated to have similar advantages as those for the close box segments outlined above.

Circular Pipes

From a foundation engineering standpoint, concrete, steel and HDPE pipes are technically feasible alternatives provided that other design issues including flow capacity, hydraulic properties and durability can be satisfied. Multiple pipes may be required to provide adequate hydraulic capacity. It is understood that this option is not considered at this site and therefore foundation recommendations for pipe culverts are not developed.

Recommended Foundations

From a foundation technical, constructability, cost-effectiveness perspective, and considering potential environmental and other requirements, a concrete open frame culvert on strip footings is feasible. Based on currently available subsurface information, we concur that the proposed combination of cast-in-place strip footings and precast (possibly segmental) rigid frame is appropriate for this site.

At the time of preparation of this report, there is no subsurface information for the middle section and the inlet/outlet areas of the new culvert. Additional borehole data will be required at those locations for detail design.

8.2 Concrete Open Frame (Strip Footing) Culvert

For preliminary design, the culvert footings may be founded at or below Elevations 99.3 to 99.7 on the limestone bedrock. The footings may be stepped along their alignments to accommodate changes of bedrock levels.

The following geotechnical resistance is recommended for preliminary design of strip footings founded on bedrock.

- Factored Geotechnical Resistance of 1,500 kPa at Ultimate Limit States (ULS)

The SLS condition does not apply to footings founded on bedrock.

The value of 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 sliding resistance of cast-in-place concrete placed on undisturbed limestone bedrock may be computed based on an ultimate coefficient of friction, tan δ, of 0.7.

8.2.1 Footing Construction

After the footing excavation reaches the design founding elevation, any remaining fill, topsoil, alluvium and organic deposits, loose/soft or disturbed soils and any deleterious materials within the footing footprint must be sub-excavated down to bedrock and the exposed surface must be inspected to confirm that the subgrade is suitable and uniformly competent.

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.

Should there be rock protrusions above the founding elevation, rock sub-excavation would be required to lower the rock level to below the founding elevation. Mass concrete fill similar to the working slab should be used to raise the subgrade to the design founding level.

Likewise, any area of sub-excavation required for removal of the existing culvert or any disturbed locations during the excavation should be restored with the same mass concrete fill.

This work must be carried out in accordance with OPSS 902 and construction must be carried out in the dry.

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.

9.0 CULVERT BACKFILL AND LATERAL EARTH PRESSURES

It is recommended that backfill to the culvert consists of free-draining, non-frost susceptible granular materials such as Granular A or B Type II conforming to the requirements of OPSS.PROV 1010 and SP110S06. Reference should be made to the backfill arrangements stipulated in OPSD 803.01 as appropriate.

All fills must be placed in regular lifts and be compacted in accordance with OPSS.PROV 501. The backfill must be placed and compacted in simultaneous lifts on both sides of the culvert, and the difference of the top of backfill elevation on both sides of the culvert should be kept within 500 mm of each other at all times. Heavy compaction equipment must not be used adjacent to the culvert.

For a rigid structure such as concrete box culvert, it is recommended that at-rest horizontal earth pressures be used for design.

Earth pressures acting on the culvert walls may be assumed to impose a triangular distribution. For a fully drained backfill, the pressures should be computed in accordance with the CHBDC 2019 but are generally given by the expression:

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

where

p_h = horizontal pressure on the wall at depth h (kPa)
K = earth pressure coefficient (see table below)
γ = bulk 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)

Earth pressure coefficients for backfill are dependent on the material used as backfill. Recommended unfactored values are shown in the following Table 9.1.

Table 9.1 - Earth Pressure Coefficients (K)

Wall Condition	**Earth Pressure Coefficient (K)**		
	OPSS Granular A or OPSS Granular B Type II $\phi = 35°; \gamma = 22.8\ kN/m^3$	**OPSS Granular B Type I (modified) $\phi = 32°; \gamma = 21.2\ kN/m^3$**	**Embankment Fill $\phi = 30°; \gamma = 20.0\ kN/m^3$**
	Horizontal Surface Behind Wall	**Horizontal Surface Behind Wall**	**Horizontal Surface Behind Wall**
Active (Unrestrained Wall)	0.27	0.31	0.33
At rest (Restrained Wall)	0.43	0.47	0.50
Passive (Movement Towards Soil Mass)	3.7	3.3	3.0

For sloping backfill, the earth pressure parameters should be adjusted upwards.

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular A or Granular B Type II, or at a depth of 1.7 m for Granular B Type I. Compaction equipment to be used adjacent to the culvert walls should be restricted in accordance with OPSS.PROV 501.

10.0 EMBANKMENT DESIGN AND CONSTRUCTION

Based on currently available information, the existing embankment fills are up to about 3.8 m in height above the culvert invert with design slope inclinations of 2H : 1V. It is understood that widening of Highway 401 is planned for both the EBL and WBL lanes. These widenings will require placement of new fill.

Embankment widening construction should be carried out in accordance with OPSS.PROV 206. The new fill should consist of Granular A or B Type II material, or Select Subgrade Material (SSM). The new slopes should be designed to match the existing slope configuration with an inclination of 2H : 1V or flatter. Where applicable, benching of the existing earth slope surface should be carried out as per OPSD 208.010 in order to enhance the keying in of the new fill.

No global stability issues are anticipated for the new embankment fills at this site given the compact embankment fill and the native very dense/firm to very stiff native tills 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.

New fill will be placed around the new culvert at both ends for the highway widening. Based on available information, the new fill above the new culvert will be less than 1.3 m in height, and approximately 3.8 m in height beyond the culvert zone. Therefore, it is anticipated that the subgrade soils in the vicinity of the culvert within the highway widening footprint will be subjected to additional loading.

It is estimated that the total settlement will be less than 25 mm which is largely immediate (elastic) settlement of the underlying subgrade soils as the fill is placed, and should be essentially completed by the end of construction.

11.0 EXCAVATION AND GROUNDWATER CONTROL

All excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the embankment fill and native glacial tills are classified as Type 3 soils. Surficial alluvial deposits that are anticipated in the inlet and outlet areas are classified as Type 4 soils. Soils below the groundwater level are classified as Type 4 soils.

Excavation and backfilling for culvert construction must be carried out in accordance with OPSS 902. Rock excavation for culvert construction should be minimized. Some 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 culvert installation will extend below the creek level, and must be carried out in conjunction with temporary creek diversion, temporary protection and cofferdams. Dewatering will be required to facilitate footing construction in the dry.

Design of the dewatering system that will be required is the responsibility of the Contractor. This design must take into account the maximum creek level likely to occur during construction. Suitable systems that might be considered to maintain an unwatered condition at this site include sandbagging and/or steel cofferdam enclosures. Pumping from filtered sumps should be required in all cases. Dewatering must remain operational and effective until the new culvert is installed and backfilled.

Dewatering of all excavations should be carried out in accordance with OPSS.PROV 517, SP 517F01 Amendment to OPSS 517, November 2016 (issued July 2017).

12.0 EROSION CONTROL

Erosion control should be provided at the culvert inlet and outlet areas as applicable. Design of erosion protection measures must consider hydrologic and hydraulic issues and should be carried out by specialists experienced in this field.

Typically, rock protection should be provided over all surfaces with which creek water is likely to be in contact. Treatment at the outlets should be in accordance with OPSD 810.010. A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion in general accordance with OPSS.PROV 804.

It is recommended that a clay seal at the inlet end or a concrete cut-off wall be used to minimize the potential for piping around the culvert. The clay seal must extend to the order of 0.3 m above the high water level and laterally for the width of the granular material, and have a minimum thickness of 0.5 m. The material requirements should be in accordance with OPSS.PROV 1205. A geo-synthetic clay liner may be used as a clay seal.

13.0 TEMPORARY PROTECTION SYSTEMS

Temporary protection (shoring) systems will be required for construction of the new culvert 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 may not 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 GEOCRES 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 culvert.

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

- 1 borehole at each end of the culvert (i.e. inlet and outlet).
- Minimum of 1 borehole at the embankment crest.
- Boreholes shall be placed at a maximum of 25 m spacing.
- Boreholes shall be advanced to a minimum of 10 m below culvert invert or to refusal, whichever is less.
- If bedrock is encountered, bedrock shall be cored for a minimum depth of 3 m.

The two boreholes advanced near the crest of the existing highway embankment during this preliminary investigation can be incorporated into the detail investigation program. In order to satisfy the minimum requirements above, the borehole configuration for detail design should be as follows:

- 2 boreholes at each end of the culvert near the inlet and outlet for a total of 2 boreholes.
- 1 borehole near the mid-point of the culvert.
- 2 boreholes near the crest of slope on both sides (current preliminary investigation).

There should be a total of 3 new boreholes.

15.0 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.

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Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

NOTE: Hierarchy of Soil Strength Prediction
1) Laboratory Triaxial Testing
2) Field Insitu Vane Testing
3) Laboratory Vane Testing
4) SPT value
5) Pocket Penetrometer

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE

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

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

▼ Water Level

C_{pen} Shear Strength Determination by Pocket Penetrometer

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

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

UNIFIED SOILS CLASSIFICATION

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

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

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

SYMBOLS

CLAYSTONE

SILTSTONE

SANDSTONE

COAL

Bedrock (general)

DISCONTINUITY SPACING

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

TERMS

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

STRENGTH CLASSIFICATION

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

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RECORD OF BOREHOLE No PC 20-01

METRIC

W.P. 4193-15-00 LOCATION Potter Creek Culvert, MTM NAD83-9 N 4 893 455.3 E 228 092.2 ORIGINATED BY GA

DIST Eastern HWY 401 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN

DATUM Geodetic DATE 2020.11.10 - 2020.11.10 LATITUDE 44.177068 LONGITUDE -77.459334 CHECKED BY RPR

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
104.0	GROUND SURFACE						
0.0 103.3	**SAND** and **GRAVEL**, trace silt Compact Brown Moist (FILL)	1	SS	30			
0.7	Gravelly **SAND**, some silt and clay Compact Brown Moist (FILL)	2	SS	17			24 64 12 (SI+CL)
101.6		3	SS	12			
2.4	Clayey **SILT**, with sand, some gravel Firm to Very Stiff Brown to Grey Moist (TILL)	4	SS	6			11 44 30 15
		5	SS	20			
99.7	No recovery Spoon bouncing Coring started at 4.3m	6	SS	50/ 0.0	∇	FI	
4.3	**LIMESTONE**, moderately weathered, grey to dark grey, with shale interbeds, laminated, horizontally bedded: (Simcoe Group) Highly fractured zone from 4.6m to 4.8m Horizontal fractures at 4.37m, 4.49m, 4.58m, 4.78m, 4.83m, 4.90m, 5.02m, 5.06m, 5.10m, 5.11m, 5.15m, 5.21m, 5.28m, 5.38m, 5.47m, 5.49m, 5.59m, 5.64m, 5.65m, 5.67m, 5.69m and 5.77m	1	RUN			4 >10 6 6 5	RUN #1 TCR=100% SCR=83% RQD=45% UCS=99MPa (Average) UCS=15MPa at 5.7m UCS=6.8MPa at 5.9m
96.7 7.3	Clay seam: 4.90m - 4.91m (10mm) Shale interbeds: 4.56m - 4.58m (20mm) 4.94m - 4.99m (50mm) 5.48m - 5.51m (30mm) 5.63m - 5.64m (10mm) 5.73m - 5.74m (10mm) 5.80m - 5.83m (30mm) 5.90m - 5.93m (30mm) 5.97m - 5.98m (10mm) 6.08m - 6.12m (40mm) 6.60m - 6.61m (10mm) 6.66m - 6.68m (20mm) 6.69m - 6.79m (100mm) 6.89m - 6.99m (100mm) Vertical fracture from 5.28m to 5.38m Moderately to slightly weathered Horizontal fractures at 5.84m, 5.90m, 5.91m, 6.04m, 6.13m, 5.19m, 6.27m, 6.30m, 6.33m, 6.40m and 6.82m	2	RUN			4 5 2 1 1	RUN #2 TCR=100% SCR=98% RQD=76% UCS=86MPa (Average)
	END OF BOREHOLE AT 7.3m. BOREHOLE OPEN TO 7.3m AND WATER LEVEL AT 4.3m UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH						

Elevation scale: 103, 102, 101, 100, 99, 98, 97

Dynamic Cone Penetration Resistance Plot: 20 40 60 80 100
Shear Strength kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100
Plastic Limit w_P, Natural Moisture Content w, Liquid Limit w_L; WATER CONTENT (%) 20 40 60

Continued Next Page

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

RECORD OF BOREHOLE No PC 20-01

METRIC

W.P. 4193-15-00 LOCATION Potter Creek Culvert, MTM NAD83-9 N 4 893 455.3 E 228 092.2 ORIGINATED BY GA

DIST Eastern HWY 401 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN

DATUM Geodetic DATE 2020.11.10 - 2020.11.10 LATITUDE 44.177068 LONGITUDE -77.459334 CHECKED BY RPR

SOIL PROFILE			SAMPLES									
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
	Continued From Previous Page											
	BENTONITE HOLEPLUG TO 0.3m THEN SAND AND GRAVEL TO SURFACE.											

+ 3, × 3: Numbers refer to Sensitivity

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

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RECORD OF BOREHOLE No PC 20-02 — 1 OF 1 — METRIC

W.P. 4193-15-00 | LOCATION Potter Creek Culvert, MTM NAD83-9 N 4 893 419.4 E 228 111.8 | ORIGINATED BY GA

DIST Eastern | HWY 401 | BOREHOLE TYPE Hollow Stem Augers/NQ Coring | COMPILED BY AN

DATUM Geodetic | DATE 2020.11.09 - 2020.11.09 | LATITUDE 44.176747 | LONGITUDE -77.459084 | CHECKED BY RPR

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
103.9	GROUND SURFACE						
0.0	**SAND** and **GRAVEL**, trace silt Compact Brown Moist (FILL)	1	SS	22			
103.0 / 0.9	**SAND**, trace gravel, trace silt and clay Compact Brown Moist (FILL)	2	SS	17			
		3	SS	10			8 83 9 (SI+CL)
		4	SS	13			
100.9 / 3.0	Silty **SAND**, some gravel, trace clay Very Dense Brown Moist (TILL) Spoon bouncing Limestone fragments Coring started at 4.6m	5	SS	60/ 0.150			14 51 26 9
		~~6~~	~~SS~~	~~50/~~ 0.025	∇ (4.6m)	FI	
99.3 / 4.6	**LIMESTONE,** moderately weathered, grey to dark grey, with shale interbeds, laminated, horizontally bedded: (Simcoe Group) Horizontal fractures at 4.60m, 4.63m, 4.80m, 4.85m, 4.90m, 5.01m, 5.02m, 5.10m and 5.37m Moderately to slightly weathered	1	RUN			4, 4, 1, 0, 0	UCS=6.3MPa at 5.2m RUN #1 TCR=95% SCR=90% RQD=75% UCS=105MPa (Average)
	Shale interbeds: 4.70m - 4.73m (30mm) 5.33m - 5.37m (40mm) 5.44m - 5.55m (110mm) 6.07m - 6.09m (20mm) 6.10m - 6.18m (80mm) 6.28m - 6.34m (60mm) 6.48m - 6.53m (50mm) 6.72m - 6.77m (50mm) 7.16m - 7.17m (10mm) 7.34m - 7.36m (20mm) Horizontal fractures at 6.10m, 6.11m, 6.27m, 6.49m and 7.23m	2	RUN			3, 1, 0, 1, 0	UCS=21MPa at 6.6m RUN #2 TCR=100% SCR=97% RQD=90% UCS=92MPa (Average)
96.3 / 7.6	END OF BOREHOLE AT 7.6m. BOREHOLE OPEN TO 7.6m AND WATER LEVEL AT 4.6m UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.3m THEN SAND AND GRAVEL TO SURFACE.						

Elevation scale: 103, 102, 101, 100, 99, 98, 97

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE: 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

+ ³, × ³: Numbers refer to Sensitivity

15 ⊕ 5 (20, 10) (%) STRAIN AT FAILURE

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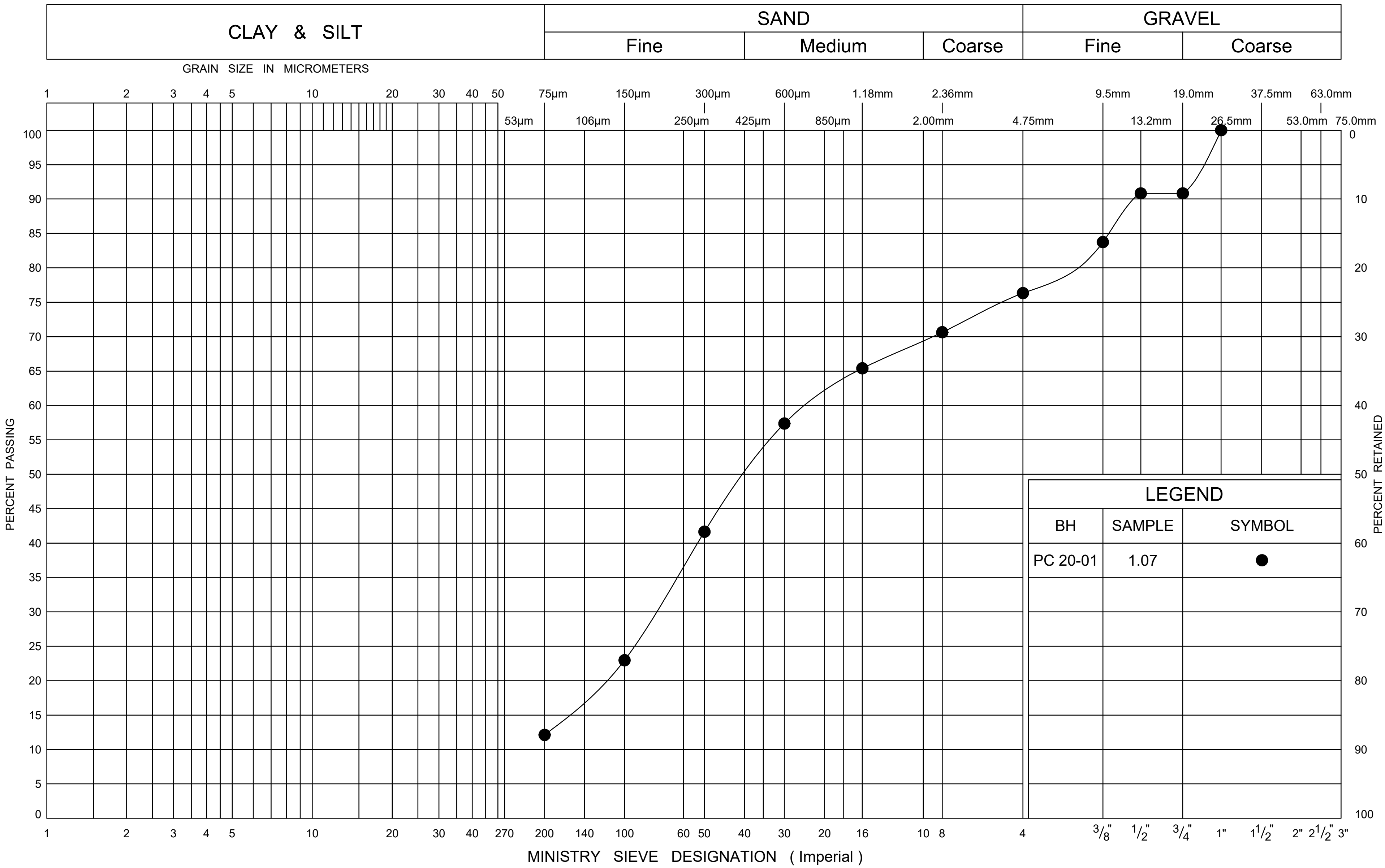
Appendix B

Geotechnical Laboratory Test Results



BH	SAMPLE	SYMBOL
PC 20-01	1.07	●

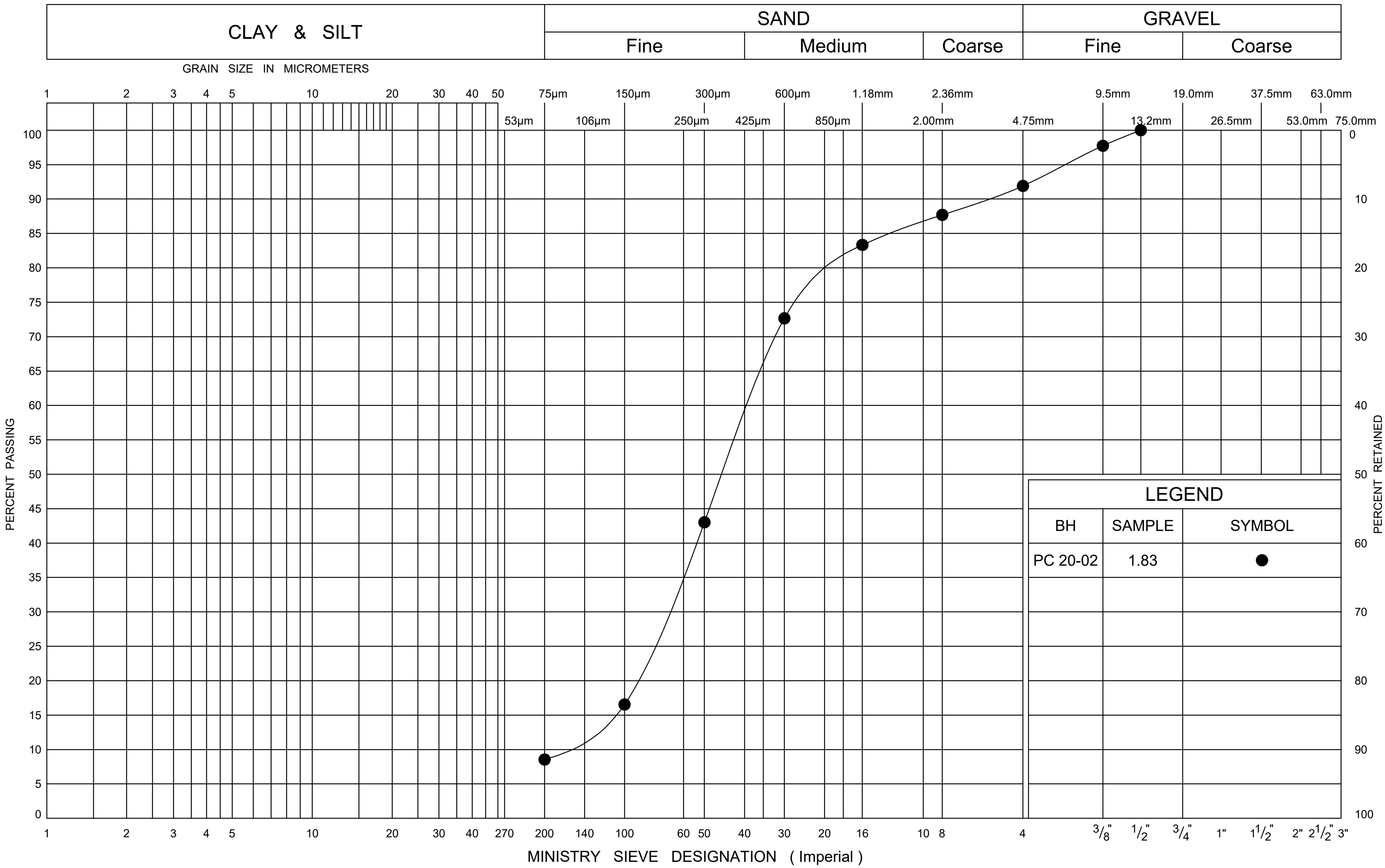
GRAIN SIZE DISTRIBUTION

Gravelly SAND FILL

FIG No B1

W P 4193-15-00

78 12 M



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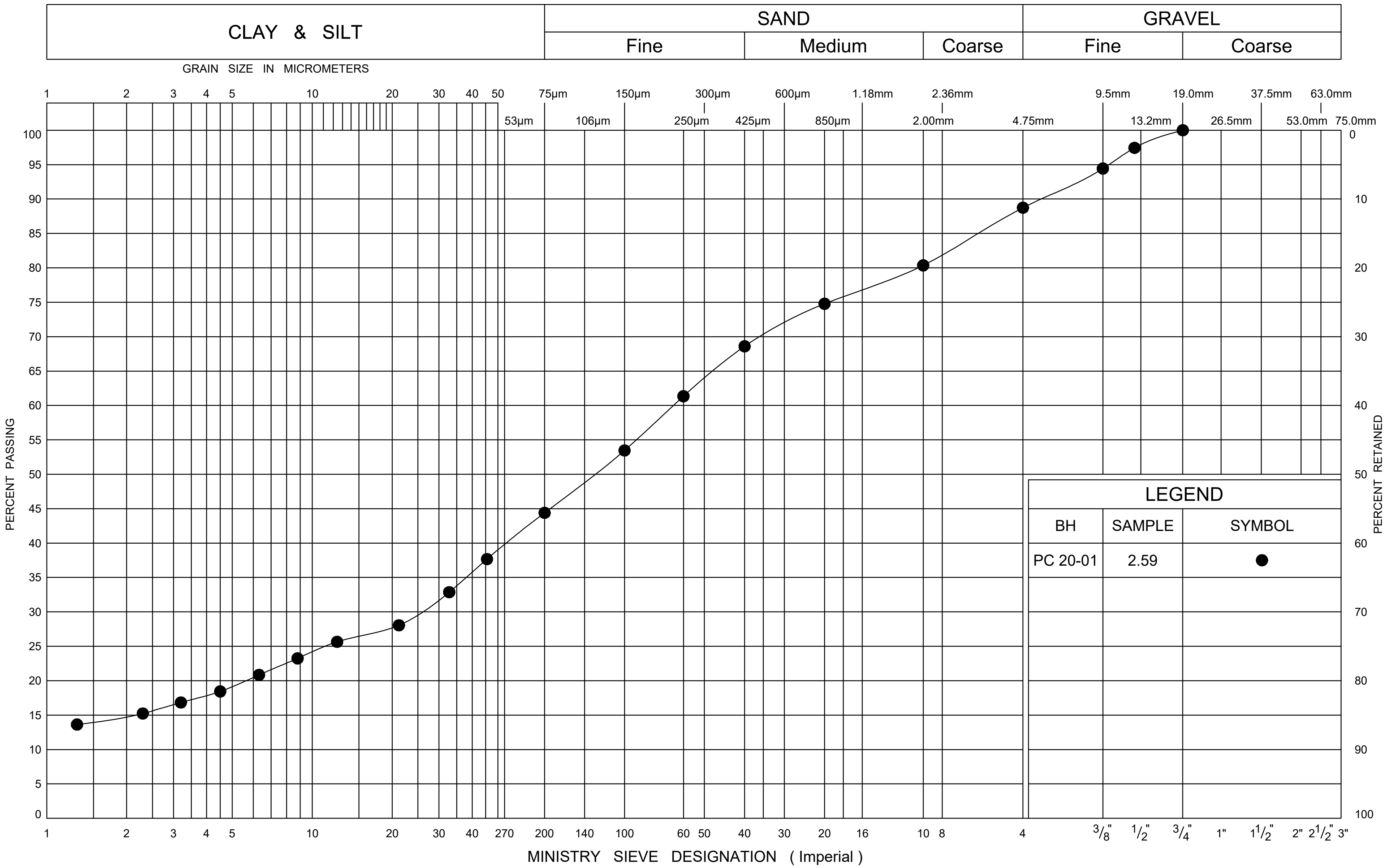
GRAIN SIZE DISTRIBUTION

SAND FILL

FIG No B2

W P 4193-15-00



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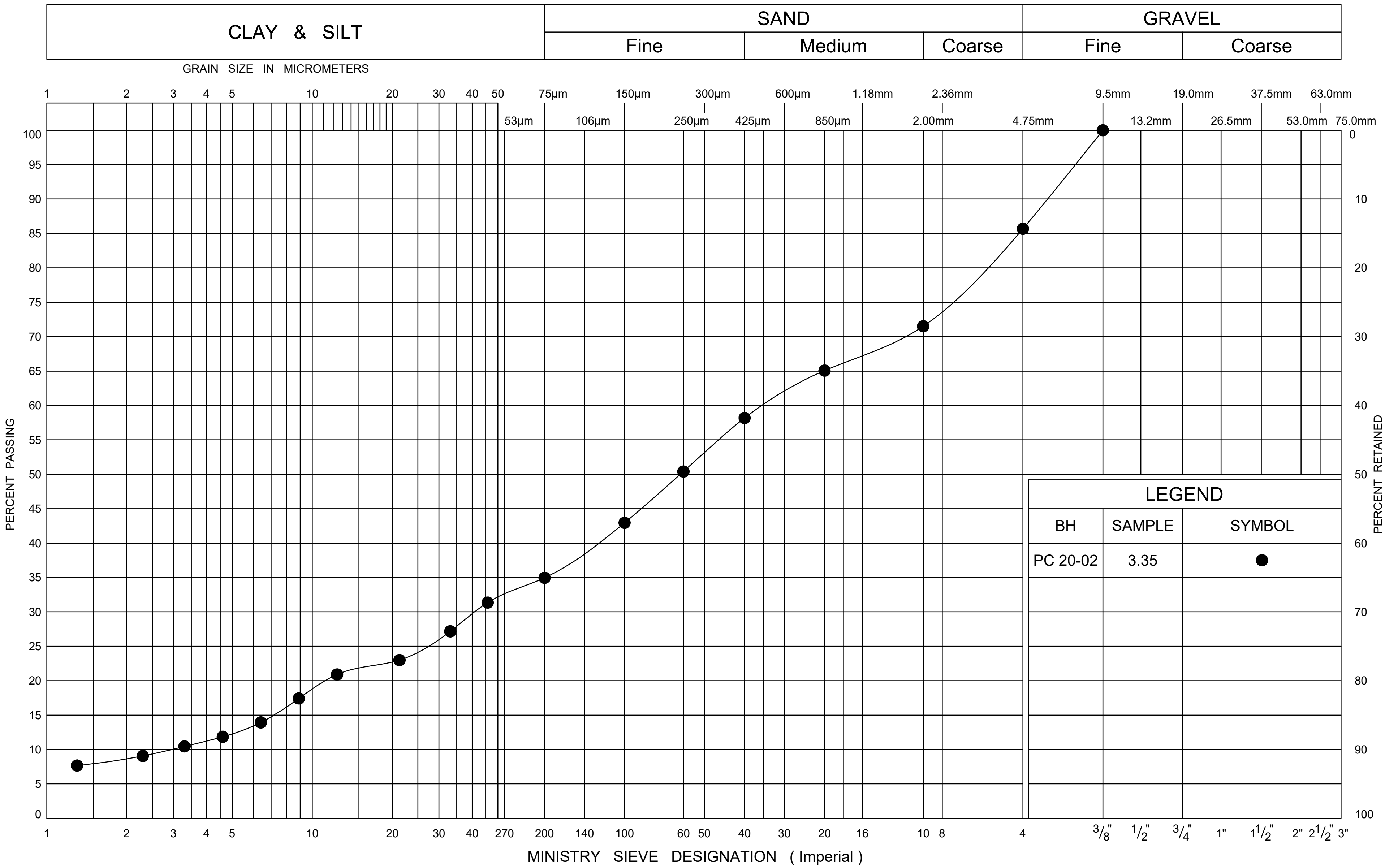
GRAIN SIZE DISTRIBUTION

Clayey SILT TILL

FIG No B3

W P 4193-15-00

78 12 M



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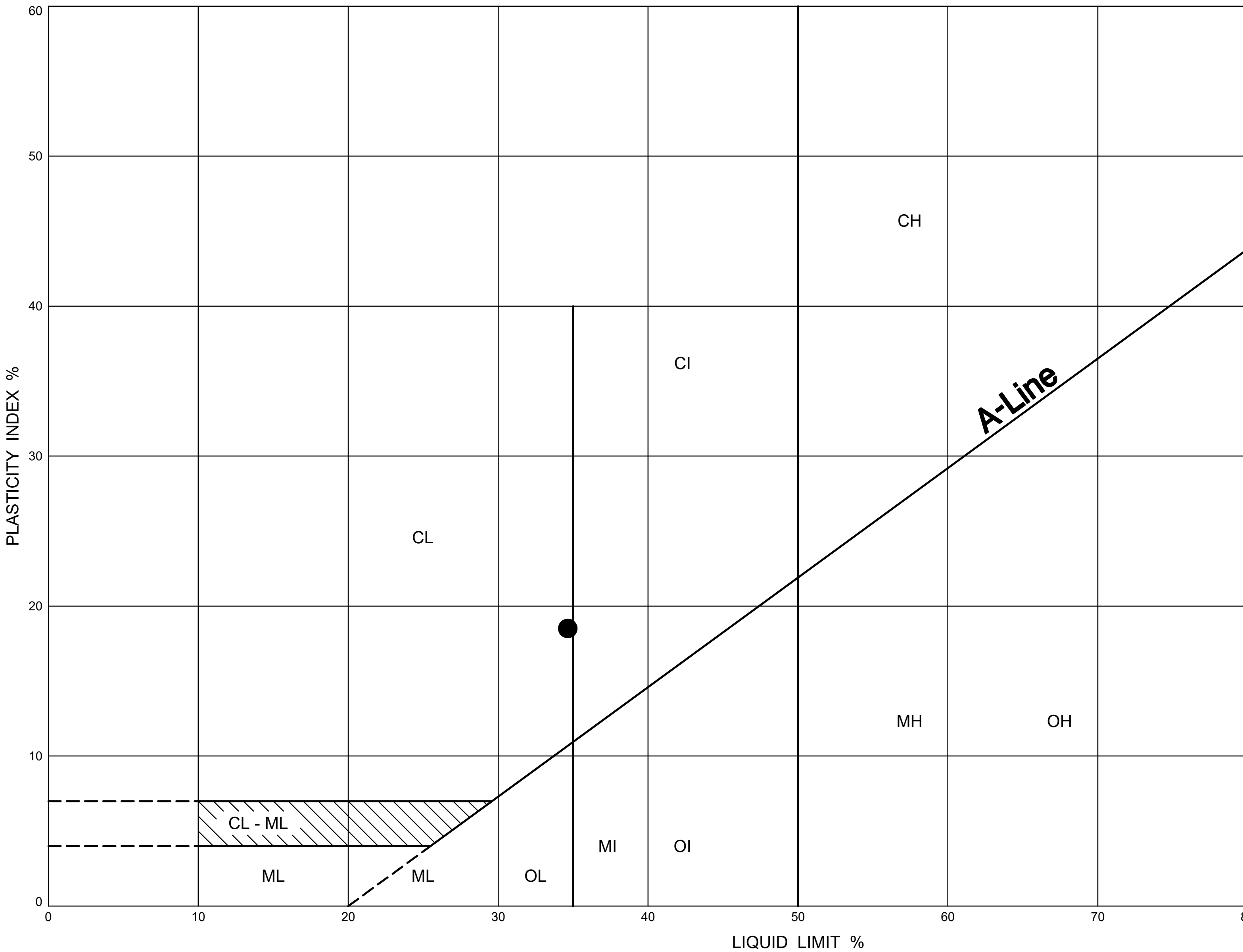
GRAIN SIZE DISTRIBUTION

Silty SAND TILL

FIG No B4

W P 4193-15-00

Oct 75, FF - S - 21



LEGEND		
BH	SAMPLE	SYMBOL
PC 20-01	2.59	●

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PLASTICITY CHART

Clayey SILT TILL

FIG No B5

W P 4193-15-00

ONTARIO MOT PLASTICITY CHART MTO-11566.GPJ ONTARIO MOT.GDT 11/30/20

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	11566	**Date Drilled:**	11-Nov-20
Client:	WSP Canada Group Ltd.	**Date Tested:**	18-Nov-20
Project Name:	Hwy 401 Belleville	**Tester:**	MP
Core Size:	NQ **BH No :** PC20-01	**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	4.8	A	10.2	47.3	56.5	3.0	73.1	Limestone	Strong
2	1	5.2	A	22.3	47.5	53.0	7.0	167.1	Limestone	Very Strong
3	1	5.5	D	5.7	47.4	73.0	2.4	56.5	Limestone	Strong
4	1	5.7	A	2.0	47.3	49.5	0.6	15.6	Limestone	Weak
5	1	5.9	D	0.7	47.2	67.3	0.3	6.8	Limestone	Weak
6	2	6.1	A	12.6	47.3	54.3	3.9	92.8	Limestone	Strong
7	2	6.6	A	13.3	47.3	50.2	4.4	104.8	Limestone	Very Strong
8	2	6.9	D	5.8	47.6	67.9	2.4	57.3	Limestone	Strong
9	2	7.2	A	11.0	47.2	50.0	3.6	87.1	Limestone	Strong
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* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

* Diametral Test should have 0.7 x D on either side of test point.

* Correlation factor to obtain UCS values is 24.

Last Modified: September 14, 2016

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	11566	**Date Drilled:**	09-Nov-20
Client:	WSP Canada Group Ltd.	**Date Tested:**	17-Nov-20
Project Name:	Hwy 401 Belleville	**Tester:**	MP
Core Size:	NQ **BH No :** PC20-02	**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	4.7	A	20.8	47.3	50.4	6.8	162.8	Limestone	Very Strong
2	1	5.2	D	0.6	47.6	72.2	0.3	6.3	Limestone	Weak
3	1	5.4	A	13.7	47.5	52.9	4.3	102.9	Limestone	Very Strong
4	1	6.0	A	12.5	47.5	52.8	3.9	94.1	Limestone	Strong
5	1	6.0	D	6.1	47.4	73.2	2.5	60.3	Limestone	Strong
6	2	6.2	A	12.5	47.5	51.3	4.0	96.1	Limestone	Strong
7	2	6.6	D	2.1	47.6	72.9	0.9	21.0	Limestone	Weak
8	2	7.1	A	17.3	47.4	49.4	5.7	137.5	Limestone	Very Strong
9	2	7.3	D	8.5	47.7	74.6	3.5	83.2	Limestone	Strong
10	2	7.6	A	6.3	47.4	47.4	2.2	51.8	Limestone	Strong
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* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing
* Diametral Test should have 0.7 x D on either side of test point.
* Correlation factor to obtain UCS values is 24.

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Appendix C

Rock Core Photographs

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Potter Creek Culvert
Pictures taken on November 11, 2021

PHOTOGRAPHS OF ROCK CORES

BOREHOLE PC20-1
RUNS 1 AND 2

TOP

Run 1

Run 2

BOTTOM

Run #	Depth (m)
1	4.3 – 5.8
2	5.8 – 7.3

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Potter Creek Culvert
Pictures taken on November 11, 2021

PHOTOGRAPHS OF ROCK CORES

BOREHOLE PC20-1
RUNS 1 AND 2

TO

Run 1

Run 2

Run #	Depth (m)
1	4.3 – 5.0
2	5.8 – 6.5

Run 1

Run 2

Run #	Depth (m)
1	5.0 – 5.8
2	6.5 – 7.3

BOTTOM

PHOTOGRAPHS OF ROCK CORES

BOREHOLE PC20-2
RUNS 1 AND 2

TOP

Run 1

Run 2

BOTTOM

Run #	Depth (m)
1	4.6 – 6.1
2	6.1 – 7.6

PHOTOGRAPHS OF ROCK CORES

BOREHOLE PC20-2
RUNS 1 AND 2

TOP

Run 1

Run 2

Run #	Depth (m)
1	4.6 – 5.5
2	6.1 – 7.0

Run 1

Run 2

Run #	Depth (m)
1	5.5 – 6.1
2	7.0 – 7.6

BOTTOM

THURBER

Appendix D

Selected Site Photographs

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Photo 1- Potter Creek Culvert , north side
Photo taken on November 18, 2020

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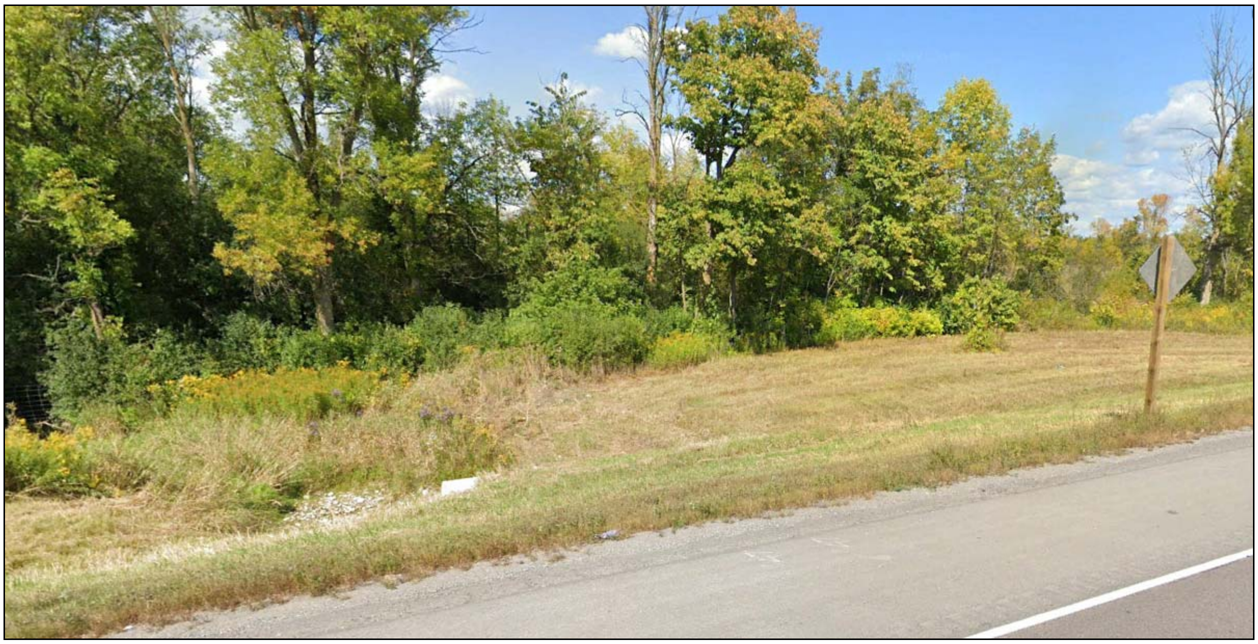

Photo 2- Potter Creek Culvert , north side
Photo taken on April 30, 2018

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Photo 3- Potter Creek Culvert , south side
Photo taken on April 30, 2018

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Appendix E

Borehole Locations and Soil Strata Drawing

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

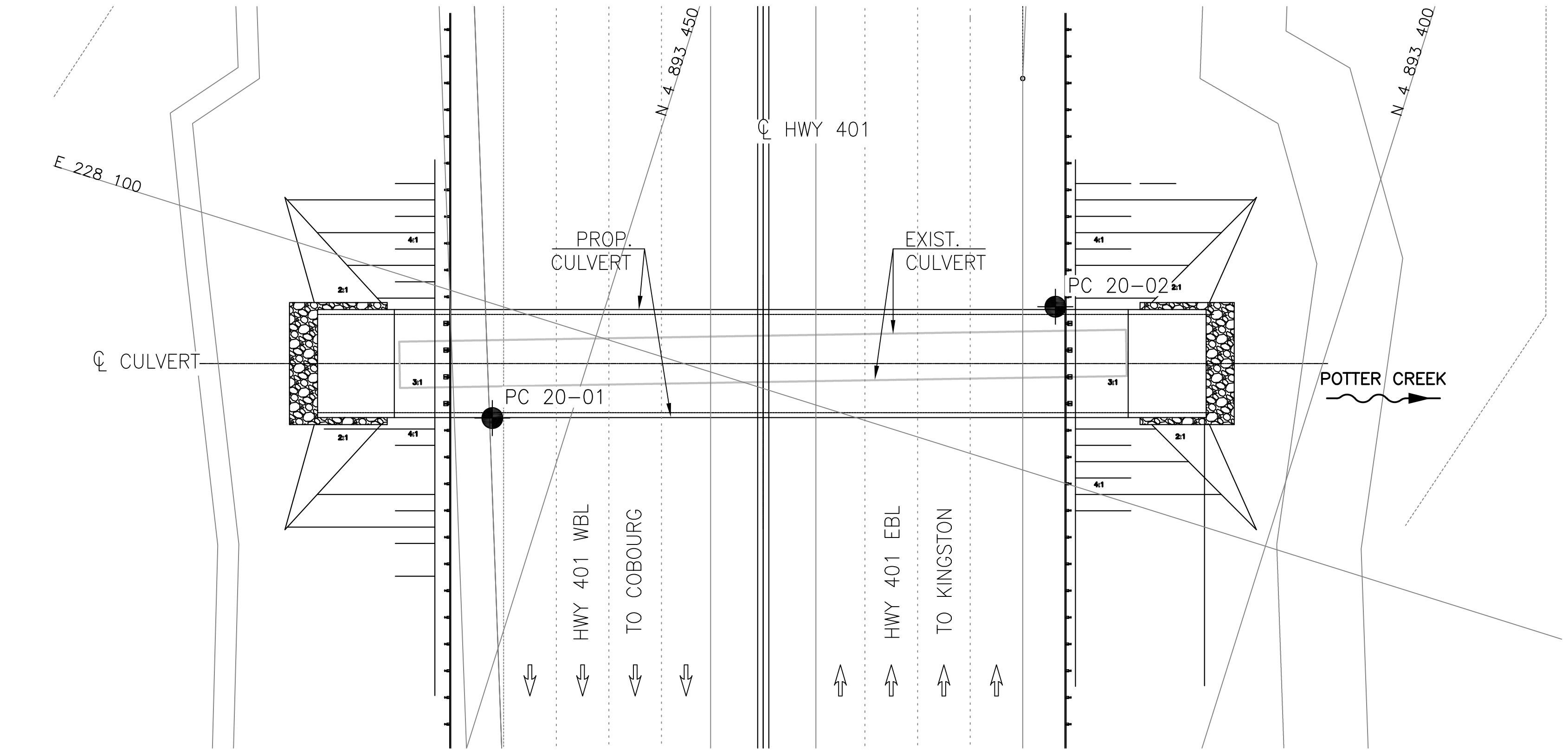
CONT No
WP No 4193–15–00

HIGHWAY 401 WIDENING
POTTER CREEK CULVERT REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

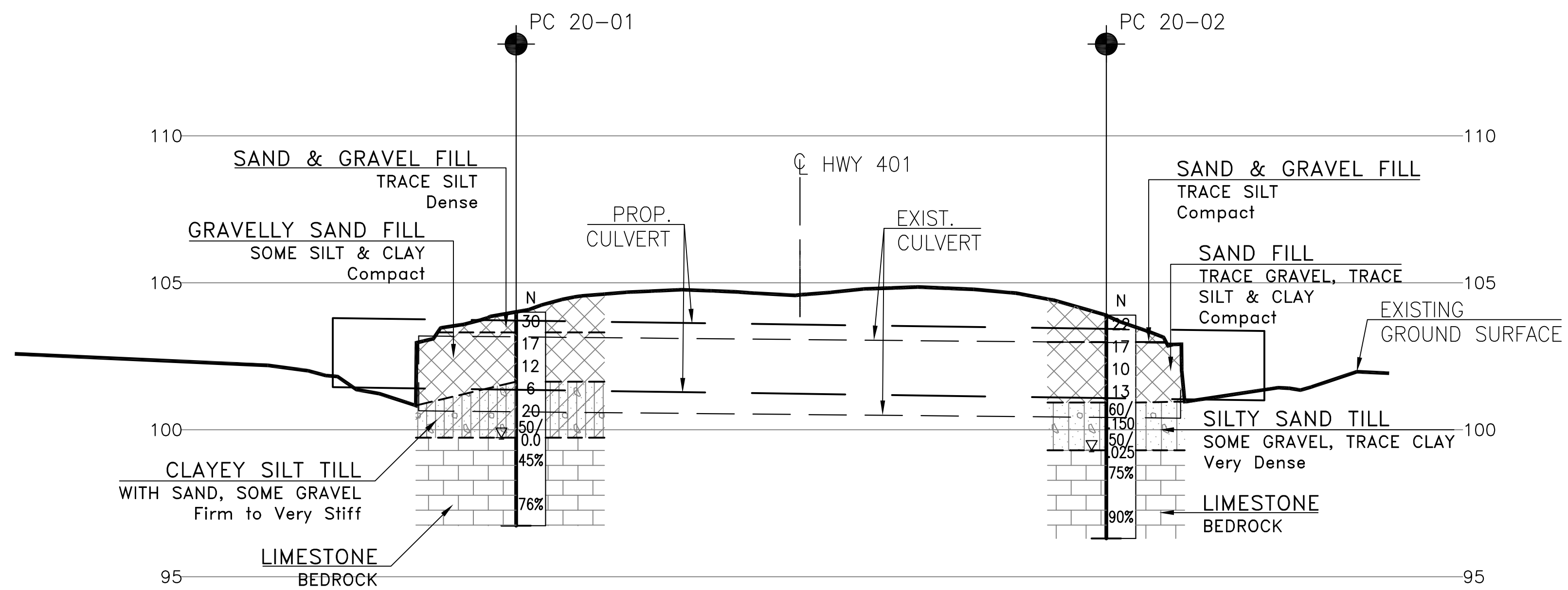
WSP

THURBER ENGINEERING LTD.



PLAN

SCALE 1:500



PROFILE ALONG ℄ CULVERT

H 1:500

V 1:250

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
PC 20–01	104.0	4 893 455.3	228 092.2
PC 20–02	103.9	4 893 419.4	228 111.8

-NOTES-

1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
3) Coordinate system is MTM NAD 83 Zone 9.

GEOCRES No. 31C-309

REVISIONS	DATE	BY	DESCRIPTION

DESIGN RPR | CHK SKP | CODE | LOAD | DATE JUL 2021
DRAWN AN | CHK RPR | SITE 11–316/C | STRUCT | DWG 1

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