

**PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 401 INTERCHANGE IMPROVEMENTS AT HIGHWAY 15
CITY OF KINGSTON, ONTARIO
G.W.P. 4059-11-00, SITE NO. 7-71
Geocres Number: 31C-241**

Report to:

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a preliminary foundation investigation conducted at the location of proposed improvements to the Highway 15 Interchange on Highway 401 in the City of Kingston, Ontario. The proposed improvements include replacement of the existing Highway 401 underpass structure, as well as realignment of Highway 15 and the interchange ramps.

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

Thurber carried out the investigation as a sub-consultant to MMM Group Limited, under the Ministry of Transportation Ontario (MTO) Agreement Number 4012-E-0020.

2 SITE DESCRIPTION

The existing underpass structure carries Highway 15 over Highway 401 at the existing interchange in the City of Kingston, Ontario. At the project site, Highway 401 runs approximately in the southwest-northeast direction, while Highway 15 runs generally north-south. For the purpose of this report, Highway 401 is assumed to run west-east. Highway 15 consists of two lanes of traffic in each direction, and Highway 401 is a four-lane (two lanes in each direction) divided freeway. The interchange also includes on- and off-ramps in the northwest, northeast and southeast quadrants, as well as ramps that meet with Middle Road, south of the bridge.

The existing underpass was constructed in 1953 and consists of a 36.2 m long single-span, concrete rigid frame, supported on spread footings founded on bedrock. The underpass was previously rehabilitated in 1996, when it received concrete patches on the deck, soffit and abutments, as well as new sidewalks and barriers.

The surrounding lands are generally wooded with some clear areas. Residential and agricultural properties exist at the northwest and southeast quadrants of the interchange.

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

The site lies within the physiographic region known as the Napanee Plain, which is generally characterized by limestone plains, covered by a discontinuous thin layer of drift. Locally, the limestone bedrock is exposed on both sides of Highway 401, including at the north and south abutments.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out in two phases. The first phase was conducted on May 19 and 20, 2015 and consisted of drilling and sampling two boreholes, designated as Boreholes 15-01 and 15-02. Borehole 15-01 was drilled off-road in the vicinity of the south abutment. Borehole 15-02 was drilled through the existing Highway 401 median in the vicinity of the proposed pier. The second phase was conducted on October 13 and 14, 2015 and consisted of drilling and sampling one borehole, designated as Borehole 15-03. Borehole 15-03 was drilled off-road in the vicinity of the north abutment. All boreholes were drilled on the west side of the existing underpass structure. The boreholes were advanced to depths of 5.3 to 6.7 m below the existing ground surface.

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

The borehole locations were marked in the field and utility clearances were obtained prior to drilling operations. The coordinates and ground surface elevations for the boreholes were derived from topographic mapping provided to Thurber by MMM Group Limited.

A track-mounted CME 850 drill rig was used to advance Boreholes 15-01 and 15-02 using hollow stem augers and NQ rock coring techniques to collect core samples of the bedrock. Borehole 15-03 was advanced using a tripod drill rig and NQ rock coring. Soil samples were obtained within the overburden soils using a split spoon sampler in conjunction with Standard Penetration Testing (SPT).

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

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Groundwater conditions observed after completion of drilling were not representative of site conditions as water was used during coring operations. A standpipe piezometer was installed in Borehole 15-01 to monitor the groundwater level after drilling and to obtain representative water level at this site. The piezometer was decommissioned following the final water level reading. The boreholes were backfilled in general accordance with MOE Regulation 903. Completion details of the piezometer and borehole are summarized in Table 3.1.

Table 3.1 – Borehole Completion Details

Borehole Location	Boreholes Number	Borehole Depth/Base Elevation (m)	Piezometer Tip Depth/Elevation (m)	Completion Details
West of South Abutment	15-01	5.3 / 107.2	5.3/ 107.2	#2 Well gravel to 3.2 m and bentonite holeplug to surface.
Highway 401 Median/Central Pier	15-02	5.4 / 100.7	None installed	Bentonite holeplug and cuttings to 0.15 m and asphalt patch from 0.15 m to surface.
West of North Abutment	15-03	6.7 / 104.3	None installed	Grout to 2.9 m and bentonite holeplug from 2.9 m to surface.

4 LABORATORY TESTING

All recovered soil samples were subjected to visual identification and natural moisture content determination. Point load tests were conducted on selected samples of the bedrock core. The results of this testing program are summarized on the Record of Borehole sheets included in Appendix A and in the Point Load Test sheets in Appendix B.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

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

The stratigraphy comprises limestone bedrock, overlain by a thin root mat and silty sand, pavement structure, or silty clay fill. More detailed description of the individual strata are presented below.

5.1 Root Mat and Asphalt

At Borehole 15-01, a 75 mm thick root mat layer was encountered at the ground surface.

A 150 mm thick layer of asphalt was encountered at the surface of the Highway 401 median at Borehole 15-02.

5.2 Granular Fill

Underlying the asphalt at Borehole 15-02 was a 2.0 m thick layer of granular fill forming a road base. The fill consisted of sand with some gravel and trace to some silt, and extended to a depth of 2.2 m (Elev. 103.9 m).

SPT 'N' values recorded in the fill ranged from 25 to 34 blows per 0.3 m penetration, indicating a compact to dense relative density. The measured moisture ranged from 2% to 10%.

5.3 Silty Clay Fill

In Borehole 15-03, the cohesive fill was encountered extending from the ground surface to a depth of 2.9 m (Elev. 108.1 m). The fill consisted of silty clay with some sand and trace gravel. Trace roots and rootlets were present throughout the soil mass, as well as organic odour was noticeable. The silty clay was mottled brown and grey in colour in the upper 0.5 m, then it became dark grey to the bottom of the layer.

SPT 'N' values recorded in the fill ranged from 5 to 32 blows per 0.3 m penetration, indicating a firm to hard relative density. The measured moisture ranged from 25% to 31%.

5.4 Silty Sand

Underlying the rootmat at Borehole 15-01, a 1.7 m thick silty sand deposit was encountered extending to a depth of 1.8 m (Elev. 110.7 m). SPT 'N' values recorded in the silty sand ranged from 6 to 18 blows per 0.3 m penetration, indicating a loose to compact relative density. The measured moisture was 21% to 23%.

5.5 Bedrock

Limestone bedrock was encountered in all boreholes either below the fill or silty sand at depths ranging from 1.8 m to 2.9 m. Bedrock was proved by coring in all three boreholes. Table 5.1 summarizes the depths and elevations to the top of bedrock at the borehole locations.

Table 5.1 – Depths and Elevations of Top of Bedrock

Foundation Element	Borehole	Top of Bedrock	
		Depth (m)	Elevation (m)
South Abutment	15-01	1.8	110.7
Central Pier	15-02	2.2	103.9
North Abutment	15-03	2.9	108.1

The top of bedrock has been established at only one point for each foundation element and may vary across the footprint of the footing.

The limestone bedrock was described as moderately to slightly weathered and thinly bedded.

Total Core Recovery (TCR) in the bedrock ranged from 44 to 100%, typically ranging from 85% to 100%. One TCR value of 44% was obtained for the first core retrieved from Borehole 15-03.

The RQD values ranged from 0% to 100%, being typically 32% to 100%, which indicates a poor to excellent rock quality.

The Fracture Index (FI) of the rock, expressed as number of fractures per 0.3 m of core run, ranged typically from 1 to 6. The Fracture Index was greater than 10, where occasional highly fractured zones were encountered in Boreholes 15-02 and 15-03. Occasional vertical fractures and shale seams were also observed.

The average unconfined compressive strength of the rock interpreted from point load tests conducted on core samples recovered from the boreholes ranged from 71 to 112 MPa, indicating a strong to very strong rock.

The unconfined compressive strengths interpreted from point load tests are presented on the Record of Borehole sheets in Appendix A. The Point Load Test Sheets are provided in Appendix B.

5.6 Water Levels

Where possible, water levels were monitored in the open boreholes during drilling operations. Wash boring techniques were used to advance the boreholes in overburden and water was used during rock coring; therefore, water levels recorded during or upon completion of drilling were not considered to reflect natural groundwater levels. A standpipe piezometer was installed in Borehole 15-01 to monitor the groundwater level at this site. The water levels measured in the piezometer are shown in Table 5.2.

Table 5.2 – Water Level Measurements

Borehole	Date	Water Level		Comment
		Depth (m)	Elev. (m)	
15-01	May 21, 2015	4.4	108.1	In piezometer
	October 6, 2015	3.9	108.6	

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

6 MISCELLANEOUS

Borehole locations were selected and established in the field by Thurber Engineering Ltd. The coordinates and the ground surface elevations for the boreholes were established based on topographic mapping information provided by MMM Group Limited.

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

Eastern Ontario Diamond Drilling of Hawkesbury, Ontario supplied a track-mounted CME-850 drill rig and conducted the drilling, sampling and in-situ testing operations for Boreholes 15-01 and 15-02. OGS Inc. of Almonte, Ontario supplied a tripod drill rig and conducted the drilling, sampling and in-situ testing operations for Borehole 15-03. The drilling operations were supervised by Mr. Chris Murray and Mr. Amir Fereidouni of Thurber.

Overall supervision of the field program, interpretation of the data, and preparation of the report were carried out by Mr. Stephane Loranger, CET and Mr. Mark Farrant P.Eng.

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

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 GENERAL

This report presents interpretation of the geotechnical data in the factual report and provides preliminary geotechnical recommendations for the Highway 401/Highway 15 Interchange improvements in the City of Kingston, Ontario.

The existing underpass structure carries Highway 15 over Highway 401 on a 36.2 m long single-span, concrete, rigid frame structure, supported on spread footings placed on bedrock. The interchange also includes on- and off-ramps in the northwest, northeast and southeast quadrants, as well as ramps that meet with Middle Road, south of the bridge.

The proposed improvements include replacement of the existing Highway 401 underpass structure, as well as realignment of the interchange ramps.

Based on the preliminary General Arrangement drawing dated September 2015, the proposed structure will be a two span bridge with a total length of 84 m (between abutments) and with each span 42 m in length. The new underpass is located immediately to the west of the existing bridge with a horizontal clearance of 3.2 m between the proposed and existing underpasses.

In light of shallow or exposed limestone bedrock surface, spread footings are envisioned for the support of the new structure. The approach fill heights above the surrounding ground are indicated to be approximately 4 m at the south abutment and 3 m at the north abutment.

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

8 STRUCTURE FOUNDATIONS

Limestone bedrock was encountered in all boreholes either below the fill or native silty sand at depths ranging from 1.8 m in Borehole 15-01 to 2.9 m in Borehole 15-03. The depths and elevations to the top of bedrock at the borehole locations are summarized in Table 5.1.

The groundwater level in the standpipe piezometer installed in Borehole 15-01 and sealed in the limestone bedrock was measured at 3.9 m below the ground surface or at Elev. 108.6. The groundwater level at the bridge site will fluctuate and, in general, will be governed by the seasonal weather patterns.

8.1 Foundation Alternatives

Consideration was given to the following foundation types for the new abutments and piers:

- Spread footings placed on limestone bedrock
- Spread footings placed on granular pad
- Steel H-piles socketed into limestone bedrock, and
- Caissons socketed into bedrock.

Limestone bedrock underlies the bridge site at relatively shallow depths. Spread footings constructed directly on the bedrock surface or on granular pads placed on bedrock are expected to offer cost effectiveness and relatively easy construction.

A foundation consisting of H-piles socketed into bedrock is feasible and would permit design of integral abutments. Due to relatively shallow depth to bedrock, socketing piles in the bedrock would be required to meet the pile flexibility requirements.

Caissons socketed into bedrock could be also considered at this site; although installation of caissons would require coring of large diameter holes in the hard limestone, which will be significantly less cost effective than spread footings.

Advantages and disadvantages of feasible foundation alternatives are presented in the table in Appendix F.

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

8.2 Spread Footings on Bedrock

Based on the subsurface conditions encountered at this site, the use of spread footings founded on bedrock to support the abutments and central pier is considered feasible from a geotechnical perspective.

The depths to bedrock and the bedrock surface elevations encountered in the boreholes advanced during the present investigation are indicated in Table 5.1.

Table 8.1, below, provides anticipated founding elevations for spread footings utilizing the available bedrock information and the Preliminary General Arrangement drawing.

Table 8.1 - Anticipated Founding Elevations for Spread Footings on Bedrock

Foundation Element	Borehole Number	Depth to Bedrock / Elevation of Bedrock Surface (m)	Thickness of Fractured Zone (m)	Anticipated Founding Elevation (m)
South Abutment	15-01	1.8 / 110.7	0.0	109.4
Central Pier	15-02	2.2 / 103.9	0.0	103.9
North Abutment	15-03	2.9 / 108.1	0.2	107.9

The actual founding elevation at specific foundation locations will have to be confirmed during detailed design stage.

The footings founded on weathered limestone bedrock can be designed using a Factored Geotechnical Resistance at ULS of 1500 kPa, which is assessed assuming a Consequence Factor equal to 1 (Typical), and a Resistance Factor equal to 0.6 (High degree of understanding of the subsurface conditions), as per CHBDC 2014. The Geotechnical Reaction at SLS will not govern the design.

The geotechnical resistance quoted above is for concentric, vertical loads only. In the case of eccentric or inclined loading, the geotechnical resistance should be calculated as illustrated in the CHBDC 2014 Clause 6.10.3 and Clause 6.10.4.

Resistance to lateral forces / sliding resistance between the footing concrete and the bedrock surface should be evaluated in accordance with the CHBDC, 2014 assuming an ultimate (unfactored) coefficient of friction of 0.7.

If the frictional component is insufficient to resist lateral forces, 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.

The forward face of spread footings should be designed to be set back from the edge of the bedrock face a minimum distance defined by a line drawn at 1H:4V from the toe of the rock face or at least 2 m away from the rock face.

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

8.3 Spread Footings on Engineered Fill

The foundations consisting of spread footing founded on engineered fill pads bearing on bedrock can be considered at this site. The founding levels on bedrock indicated in Table 5.1 can be used in design. This foundation option would allow for a higher founding elevations at the abutments and pier, if required or if this is beneficial for the design.

The engineered fill should consist of OPSS Granular “A” or Granular B Type II placed in 150 mm lifts and compacted to 100% of its SPMDD at $\pm 2\%$ of optimum moisture content. The top of the founding pad should be at least 1 m wider than the footprint of the spread footing. The side slopes of the engineered fill pad should be inclined not steeper than 1H:1V. A figure illustrating a granular pad configuration, is included in Appendix E.

Factored Geotechnical Resistance at ULS - 900 kPa

Geotechnical Resistance at SLS - 350 kPa

The geotechnical resistance at SLS quoted above corresponds to 25 mm of settlement of an individual footing.

The geotechnical resistances are for vertical, concentric loads. Where eccentric or inclined loads are applied, the resistance used in design must be reduced in accordance with the CHBDC 2014, Clause 6.10.3 and Clause 6.10.4.

The lateral resistance of the footings founded on engineered fill may be computed using an unfactored friction coefficient of 0.6. This is an “ultimate” value and requires a degree of sliding movement to occur to fully mobilize the resistance.

8.4 Augered H-Piles/Integral Abutment Considerations

Foundations consisting of H-piles socketed into bedrock are feasible and would permit design of integral abutments, if required.

The bedrock surface elevations to be used in the preliminary design of pile foundations are indicated in Table 5.1, above. The integral abutment design requires that the piles possess flexibility in the upper 3 m of the pile length, as well as a minimum total length to accommodate structural requirements. Assuming the proposed grade of Highway 15 at Elev. 114.2, the depth to limestone bedrock surface would be 6.1 m at the north abutment and 3.5 m at the south abutment. In order to facilitate the minimum pile length for flexibility requirements, piles would have to be installed in pre-augered/cored holes to ensure a total pile length (in fill and in rock) of 5 m, at least.

Abutment piles to be installed through the compacted fill and in pre-augered/cored holes will have to be surrounded in the upper 3 m by a 600 mm diameter CSP, as specified by the integral

abutment design procedures. After the pile is installed, the space between the pile and the CSP should be filled with loose uniformly graded sand. An NSSP should be included in the contract documents specifying the grain size distribution of the sand according to Table 8.2.

Table 8.2 - Integral Abutment Sand Backfill Grading

MTO Sieve Designation		Percentage Passing
2 mm	#10	100%
600 µm	#30	80%-100%
425 µm	#40	40%-80%
250 µm	#60	5%-25%
150 µm	#100	0%-6%

8.4.1 Axial Resistance

Steel H-piles should be considered as founded entirely in bedrock, as the resistance developed in the fill and relatively shallow native soils will be relatively low. For an HP 310 x 110 pile grouted within a 600 mm diameter socket in the limestone bedrock, a factored axial structural resistance at ULS of 2,000 kN per pile may be used for design.

The base of the socket should extend to the lower of the following elevations:

- 2 m below the elevation indicated in Table 5.1,
- 2 m below the elevation of the bedrock below the Highway 401 platform.

The SLS condition does not govern the design of piles founded in bedrock.

8.4.2 Lateral Resistance

The lateral resistance that can be mobilized in front of a pile socket in limestone, assuming a clear spacing of at least one socket diameter between the sockets, will exceed the structural capacity of the pile, provided socket bases are installed at elevations indicated in Sec. 8.4.1. For the evaluation, the ultimate lateral resistance, p_{ult} , of the limestone could be assumed to be half of the unconfined compressive strength presented on the Record of Borehole sheets at the design depths, i.e., approximately 36 MPa.

The structural capacity of the pile material will govern the design. The structural designer should check requirements for pile base fixity.

8.4.3 Augered Pile Installation

Pile installation should be in general accordance with OPSS 903. The pre-drilled/cored holes for forming the H-pile socket should have a diameter of 600 mm minimum.

The pile installation equipment should be capable of dislodging and removing any obstructions such as cobbles, boulders, slabs of rock and other obstructions in the fill and native soils. The use of coring equipment will be required to advance sockets in the limestone bedrock. Temporary steel liners will be required to support the socket sidewalls of the predrilled holes in the overlying soils and to minimize groundwater inflow.

The pile socket excavation should be dewatered to allow cleaning of the bases and walls prior to placing concrete. Concrete should be placed with minimum delay after the socket is drilled, cleaned, inspected and approved.

Subsequent to the seating of a pile in the socket, the socket should be grouted with 30 MPa concrete.

If an integral abutment design is selected, the holes augered through the overburden and rock should be of sufficient diameter to permit placement of the CSPs required in the integral abutment design.

8.5 Augered Caissons (Drilled Shafts)

Considering relatively shallow bedrock surface and available high bearing capacities for the spread footings, the augered caissons advanced into the limestone bedrock do not seem to offer benefits in light of cost effectiveness or constructability. This option was not developed further in this preliminary foundation report.

8.6 Recommended Foundations

The spread footings founded on limestone bedrock are preferred foundations for this bridge based on geotechnical considerations, cost effectiveness and constructability.

8.7 Erosion Protection

Erosion protection should be provided over all earth and granular surfaces, in general accordance with OPSS 804.

8.8 Frost Cover

The depth of frost penetration at this site is approximately 1.5 m. Frost protection is not required for footings founded on bedrock. However, 1.5 m of earth cover is recommended for pile caps in fill or overburden.

9 LATERAL EARTH PRESSURES

Backfill to the abutment walls should be in accordance with OPSS 902 and should consist of Granular A or Granular B Type II material. All granular material should meet the specifications of OPSS.PROV 1010. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS 902.

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

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

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

K = coefficient of lateral earth pressure (see table below)

γ = unit weight of retained soil (see table below)

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

q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the abutment walls are dependent on the material used as backfill. Typical values are given in Table 9.1, below.

The coefficients provided in Table 9.1 are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.16 in the Commentary to the Canadian Highway Bridge Design Code (CHBDC).

In accordance with Clause 6.12.3 of the CHBDC, 2014, 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 1.7 m for Granular B Type I, or at a depth of 2.0 m for Granular A or Granular B Type II.

Table 9.1 – 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	0.39*	0.31	0.47*
At-rest (Restrained Wall)	0.43	-	0.47	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-

* For wing walls.

10 SEISMIC CONSIDERATIONS

The following seismic parameters should be used for design in accordance with the CHBDC for a design earthquake with 475-year return period (an event with a 10% of probability of exceedance in 50 years):

- Velocity Related Seismic Zone 1.0
- Zonal Velocity Ratio 0.05
- Acceleration Related Seismic Zone 2
- Zonal Acceleration Ratio 0.10
- Peak Ground Acceleration 0.11 g

The soil profile type at this site has been classified as Type I. Therefore, according to Clause 4.4. 3.2 of the CHBDC, a Site Class “A” should be used in seismic design. In accordance with Clause 4.6. 5 of the CHBDC, retaining structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading. For the design of retaining walls, the coefficients of lateral earth pressure in Table 10.1 may be utilized.

Table 10.1 – Coefficient of Lateral Earth Pressure for Seismic Loading

Loading Condition	Earth Pressure Coefficient (K_E) for Seismic Loading			
	OPSS Granular A or Granular B Type II $\phi = 35^\circ, \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I or Existing Embankment Fill $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Backfill	Sloping Backfill (2H:1V)	Horizontal Backfill	Sloping Backfill (2H:1V)
Active (K_{AE})*	0.29	0.42	0.32	0.51
At-rest (K_{OE})**	0.46	-	0.51	-
Passive (K_{PE})*	3.5	-	3.1	-

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

** After Woods (1973).

Based on review on the SPT data, seismically-induced liquefaction of foundation soils is not anticipated under the design earthquake.

11 APPROACH AND RAMP EMBANKMENTS AND PERMANENT CUTS

The approach embankment will be up to 4.0 m in height at the south abutment and 6 m in height at the north abutment. At the intersection location, the existing Highway 401 corridor has been constructed in cut, therefore, permanent cuts will also be required for the new ramps, in combination with embankments.

As indicated on the preliminary drawings showing cross sections of E-N/S Ramp and S-W Ramp, the depth of the required cuts will vary and could be as much as 11 m. The embankments are not expected to be higher than 6 m.

Rock cut faces could be constructed vertically to 10 m height and above that height the cuts should be flattened to an inclination of 4V:1H.

The cuts through the overburden soils should be constructed at 2H:1V or flatter.

Discussion on constructing the permanent cuts in rock is presented in the following section.

Given the presence of shallow bedrock surface, the earth fill embankments not exceeding 6 m in height and inclined at 2H:1V or flatter are considered to be stable.

Settlement induced by the fill placement is anticipated to be essentially completed at the end of construction.

Prior to embankment construction, all organic soils/topsoil/vegetation and fill placed in an uncontrolled manner should be removed from below the footprint of the proposed embankments. The existing embankment fill in the transition zone/footprint area of the new embankment may remain in place.

For new embankments, the earth fill should be placed in lifts with loose thickness not exceeding 300 mm and compacted in accordance with OPSS.PROV 501. Side slopes of the embankment fill should be no steeper than 2H:1V. Inspection and testing should be carried out by qualified personnel during placement operation to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

Embankments constructed in this fashion are expected to be stable.

In order to improve interaction between the existing and newly placed embankment fill, when placing new fill against the existing embankment, the new fill should be benched into the existing embankment side slope in accordance with the requirements of OPSD 208.010.

If the embankment height is planned to exceed 8 m, a mid-height berm 2 m wide should be incorporated into the embankment design.

Erosion protection of the new embankment slopes in form of topsoil placement and seeding should be implemented as soon as possible. Run-off should not be allowed to discharge on the embankment m slopes in an uncontrolled manner.

In some locations, particularly at the N – W Ramp, construction of an earth fill embankment using 2H:1V side slopes may result in an undesirable property impact. This impact may be mitigated by using rock fill embankments constructed with side slopes at 1.25H:1V. In any case, where the rock fill embankment is higher than 10 m, a 2 m wide mid-height bench should be provided.

12 EXCAVATION AND GROUNDWATER CONTROL

All excavations and backfilling should be carried out in accordance with OPSS 902 and the requirements of the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the existing fill and native soils, if encountered within the depth of excavation may be classified as Type 3 soils. Flatter slopes may be required at locations where water seepage affects stability of an excavation.

Excavations for abutment footings construction are expected to extend through the embankment fill through shallow native deposits overlying the bedrock and into the bedrock. The groundwater level as measured in the piezometer on the north side of the structure was approximately at Elev. 108.6 or 3.9 m below the ground surface. However, perched water may be present in the fill material. Some water control, such as pumping from sumps, may be required.

The selection of the method of excavation is the responsibility of the Contractor and should be based on his equipment availability, experience and interpretation of the site conditions. It is anticipated that a hydraulic excavator will be suitable. Provision should be made for the handling of pavement materials and potential obstructions in the fill, and cobbles, boulders and rock slabs above the bedrock. Some fractured bedrock excavation will also be required.

To achieve the design founding level at the abutments, excavation of the limestone bedrock will be required. The upper 1 m to 1.5 m of the bedrock is typically weathered and excavation should be possible using heavy excavation equipment and rippers, supplemented by pneumatic rock breakers where thick layers of less weathered materials are encountered. The limestone below that depth is harder and less weathered, and intensive use of pneumatic/hydraulic breakers or other methods of loosening the bedrock will likely be required. Near vertical temporary sidewalls may be employed in limestone bedrock. Blasting should not be permitted in the abutment excavations.

Ramps at this interchange will require significant lengths of cuts, predominantly in the limestone bedrock, which will be as deep as 11 m.

As noted above, the upper weathered zone could be excavated using heavy excavation equipment and rippers, supplemented by pneumatic rock breakers. The deeper layers of rock will be harder and more difficult to excavate using hydraulic excavators and breakers. Given the larger volumes to be excavated for the ramp cut, consideration could be given to blasting to loosen the rock. If blasting is not permitted at this site, the contract documents should alert bidders to the fact that extensive use of pneumatic breakers may be required.

It is understood that the new structure will be constructed immediately to the west of the existing bridge, and that the traffic on Highway 15 will be maintained at all times, which will require roadway protection in the existing approach fills. Roadway protection should be provided in accordance with OPSS.PROV 539 and designed for Performance Level 2.

The design of any roadway protection or dewatering system that may be required is the responsibility of the Contractor. All shoring systems should be designed by a professional engineer experienced in such design.

13 CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to the excavation and dewatering for the spread footings, namely:

- Cobbles, fragments/slabs of rock or other buried obstruction may be encountered and interfere during excavation in the existing embankment fill.
- If deep foundations are selected for this structure, the pile installation equipment should be capable of coring the limestone and dislodging/removing any obstructions such as cobbles, boulders, etc., in the fill and native soils, if encountered. Contractor should be prepared to advance holes to specified elevations.
- If the existing foundations are to be removed, the removal procedure should not disturb the ground within the zone of influence of the new foundations.

14 INVESTIGATION FOR DETAIL DESIGN

During the detailed design phase of this project, additional site investigations and field testing will be required. Depending on the final configuration/location of the foundation units, the existing subsurface information should be reviewed and supplemented, as required, to comply with the MTO Foundation Engineering Terms of Reference.

It is recommended that as a minimum four boreholes will be required at each foundation element to refine the bedrock surface profile and the thickness of the weathered/fractured zone within the bedrock. Piezometer should be installed in the selected boreholes to establish the groundwater levels at the site.

15 CLOSURE

Engineering analysis and preparation of the foundation design report were carried out by Ms. Anna Piascik, P.Eng. The report was reviewed by Mr. Alastair E. Gorman, P.Eng. and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

THURBER ENGINEERING LTD.

Anna Piascik, P.Eng.
Senior Geotechnical Engineer



Alastair Gorman, P.Eng.
Senior Geotechnical Engineer/Senior Associate



Dr. P.K. Chatterji, Ph.D., P.Eng.
Designated Principal for MTO Foundation Projects



Appendix A
Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

NOTE: Hierarchy of Soil Strength Prediction

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



4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

5. LEGEND FOR RECORDS OF BOREHOLES

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

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

 Water Level
 Shear Strength Determination by Pocket Penetrometer

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

EXPLANATION OF ROCK LOGGING TERMS


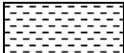



ROCK WEATHERING CLASSIFICATION

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

DISCONTINUITY SPACING

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

SYMBOLS

	CLAYSTONE
	SILTSTONE
	SANDSTONE
	COAL
	BEDROCK

STRENGTH CLASSIFICATION

Rock Strength	Approximate Uniaxial Compressive Strength (MPa)	Approximate Uniaxial Compressive Strength (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

TERMS

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

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			

RECORD OF BOREHOLE No 15-01

1 OF 1

METRIC

GWP# 4059-11-00 LOCATION Highway 15 Interchange N 4 905 406.7 E 310 380.9 ORIGINATED BY CAM
HWY 401 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
DATUM Geodetic DATE 2015.05.20 - 2015.05.20 CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
112.5	GROUND SURFACE							20	40	60	80	100		
0.0	ROOTMAT: (75mm)													
0.1	Silty SAND , some clay Loose to Compact Brown Moist		1	SS	6		112							
			2	SS	18									
			3	SS	17		111							
110.7														
1.8	BEDROCK: limestone, moderately to slightly weathered, thinly bedded, strong, grey Vertical fracture from 2.3m to 3.0m Highly fractured zone from 3.5m to 3.6m		1	RUN			110							

+³, ×³: Numbers refer to
Sensitivity

20
15
10



(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 15-02

1 OF 1

METRIC

GWP# 4059-11-00 LOCATION Highway 15 Interchange N 4 905 451.0 E 310 390.9 ORIGINATED BY CAM
HWY 401 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN
DATUM Geodetic DATE 2015.05.19 - 2015.05.19 CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			WATER CONTENT (%)				
								20 40 60 80 100			w _P w w _L				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE							
106.1	GROUND SURFACE														
0.0	ASPHALT: (150mm)														
0.2	SAND, some gravel, trace to some silt Compact to Dense Grey Moist (FILL)		1	SS	34										
			2	SS	26										
			3	SS	25										
103.9															
2.2	BEDROCK: limestone, moderately to slightly weathered, thinly bedded, strong, grey Occasional shale seams Highly fractured zone from 2.4m to 2.5m Vertical fracture from 3.6m to 3.8m		1	RUN											
			2	RUN											
100.7															
5.4	END OF BOREHOLE AT 5.4m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO 0.15m AND COLD PATCH ASPHALT TO SURFACE.														

+³, ×³: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 15-03

1 OF 1

METRIC

GWP# 4059-11-00 LOCATION Highway 15 Interchange N 4 905 477.9 E 310 390.8 ORIGINATED BY AHF
 HWY 401 BOREHOLE TYPE Tripod/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2015.10.13 - 2015.10.14 CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								<div><div><div>20406080100</div><div>○ UNCONFINED + FIELD VANE</div><div>● QUICK TRIAXIAL × LAB VANE</div></div></div>							
111.0	GROUND SURFACE														
0.0	Silty CLAY , some sand, trace gravel, organic odour through out, trace roots and rootlets Firm to Hard Mottled Brown and Grey to 0.5m, becoming Dark Grey (FILL)		1	SS	5										
			2	SS	30		110								
			3	SS	32										
			4	SS	17		109								
			5	SS	18										
108.1															
2.9	BEDROCK: limestone, moderately to slightly weathered, thinly bedded, strong to very strong, grey Sub-vertical fracture (100mm) at 3.0m Highly fractured zone from 4.3m to 4.5m Sub-horizontal fracture (25mm) at 5.5m Sub-vertical fracture (100mm) at 6.3m		1	RUN			108								
			2	RUN											
			3	RUN			107								
			4	RUN			106								
			5	RUN			105								
104.3															
6.7	END OF BOREHOLE AT 6.7m. NO FREE STANDING WATER IN BOREHOLE. COREHOLE BACKFILLED WITH GROUT TO 2.9m, THEN BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG FROM 2.9m TO SURFACE.														

ONTMT4S 1256.GPJ 2015TEMPLATE(MTO).GDT 10/22/15

Appendix B
Point Load Test Results and Rock Core Photographs



POINT LOAD TEST SHEET

Job No :	19-5161-256	Client :	MMM Group
		Drilled On:	May, 2015
Project Name :	Kingston Interchanges	Tested On :	September, 2015
Core Size :	NQ BH No : 15-1	Tester :	CAM

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	1.9	D	8.14	47.6	70.0	79.9	Limestone	Strong
2	1	1.9	A	9.43	47.6	50.0	73.9	Limestone	Strong
3	1	3.3	D	9.26	47.6	80.0	91.0	Limestone	Strong
4	2	3.9	D	13.67	47.6	70.0	134.3	Limestone	Very Strong
5	2	4.4	D	8.34	47.6	80.0	81.9	Limestone	Strong
6	2	5.0	D	8.03	47.6	120.0	78.9	Limestone	Strong
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8									
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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 $0.7 \times D$ on either side of test point.



POINT LOAD TEST SHEET

Job No :	19-5161-256	Client :	MMM Group
		Drilled On:	May, 2015
Project Name :	Kingston Interchanges	Tested On :	September, 2015
Core Size :	NQ BH No : 15-2	Tester :	CAM

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	2.2	D	15.82	47.6	120.0	155.4	Limestone	Very Strong
2	1	2.7	D	8.53	47.6	80.0	83.8	Limestone	Strong
3	1	2.7	A	12.27	47.6	50.0	96.2	Limestone	Strong
4	1	3.5	D	5.90	47.6	75.0	57.9	Shale Seam	Strong
5	1	3.5	A	13.72	47.6	50.0	107.6	Limestone	Very Strong
6	2	4.4	D	9.51	47.6	70.0	93.4	Limestone	Strong
7	2	4.5	A	10.40	47.6	50.0	81.5	Limestone	Strong
8	2	5.1	D	10.84	47.6	90.0	106.5	Limestone	Very Strong
9									
10									
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30									

* 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 $0.7 \times D$ on either side of test point.



POINT LOAD TEST SHEET

Job No :	19-5161-256	Client :	MMM Group
		Drilled On:	October 14, 2015
Project Name :	Kingston Interchanges	Tested On :	October 15, 2015
Core Size :	NQ BH No : 15-3	Tester :	AHF

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	2	3.2	D	12.04	49.3	105.7	112.0	Limestone	Very Strong
2	3	3.4	D	7.26	47.7	83.0	71.2	Limestone	Strong
3	4	5.3	D	7.90	48.5	123.2	75.4	Limestone	Strong
4	5	6.2	D	7.46	48.3	117.9	71.7	Limestone	Strong
5									
6									
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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

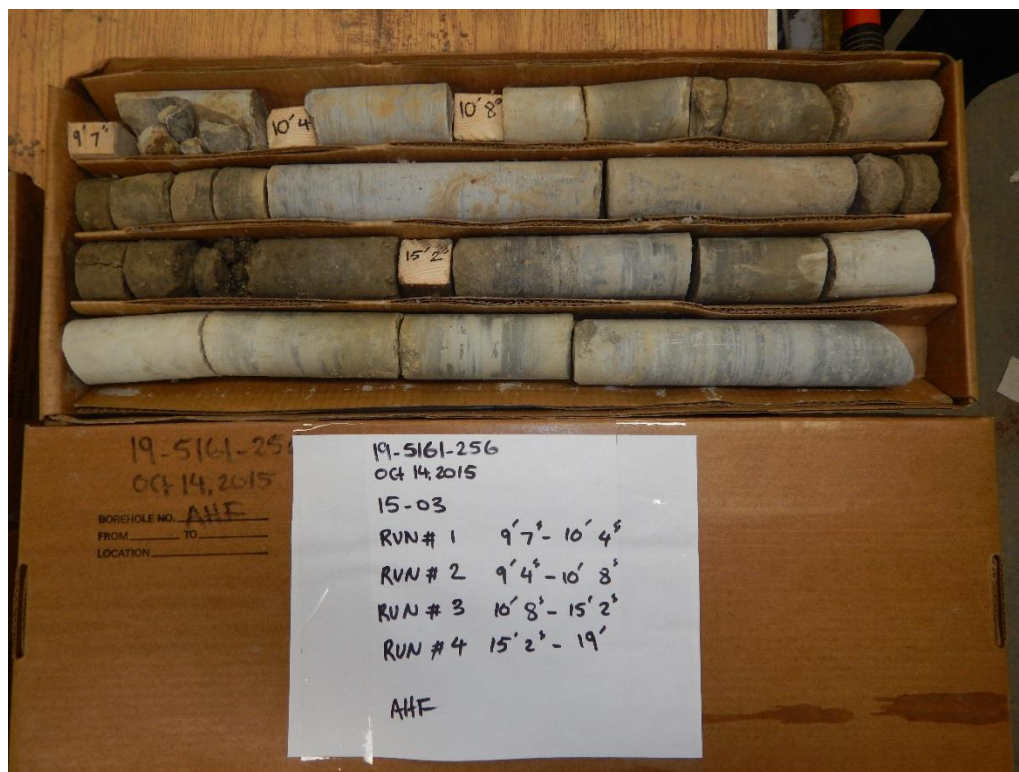
* Diametral Test should have $0.7 \times D$ on either side of test point.



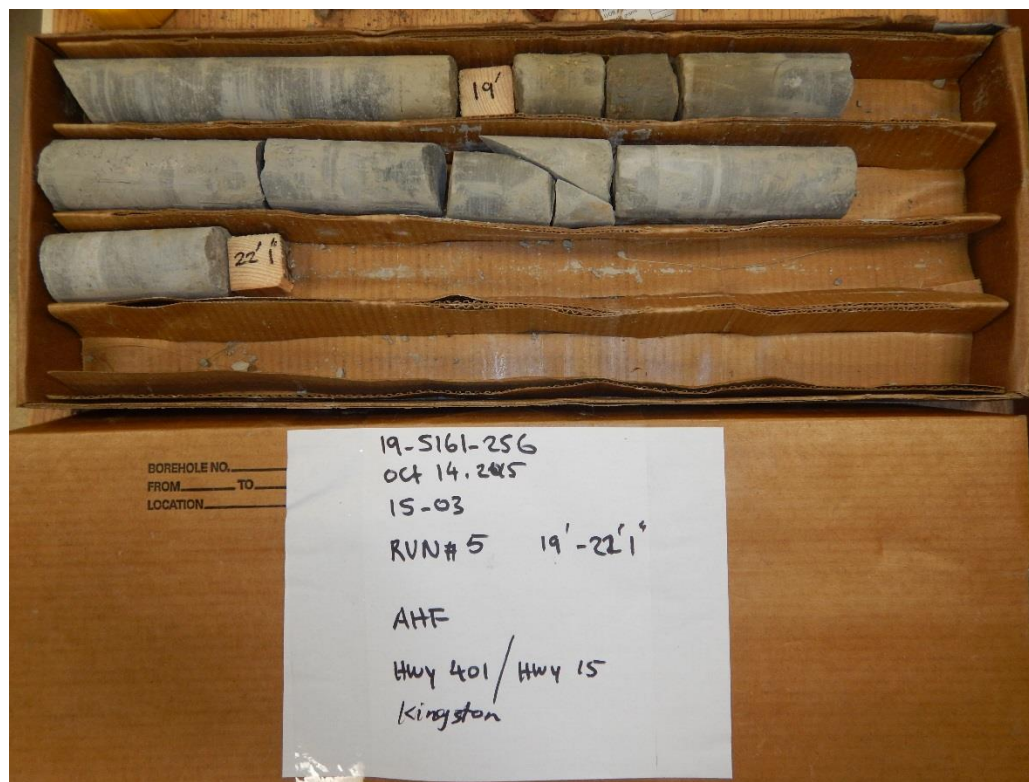
Rock Core - Borehole 15-01



Rock Core - Borehole 15-02



Rock Core - Borehole 15-03 (1 of 2)



Rock Core - Borehole 15-03 (2 of 2)

Appendix C
Site Photographs



Photograph 1 – East side of existing bridge, looking west; exposed bedrock to be noted.



Photograph 2 – West of existing bridge, looking north along approximate alignment of new bridge near new south abutment; rock cut to be noted.



Photograph 3 – South approach to existing bridge, looking north



Photograph 4 – Looking north along approximate alignment of south approach to new bridge



Photograph 5 – West side of existing bridge, looking southeast near new north abutment.



Photograph 6 – West of existing bridge, looking south along approximate alignment of north approach to new bridge; rock cut to be noted.





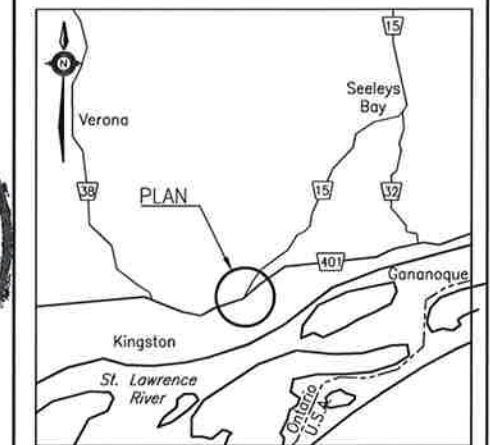
Photograph 7 – North approach to existing bridge, looking south

Appendix D
Borehole Locations and Soil Strata Drawing






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



<p>CONT No WP No 4059-11-00</p>	
<p>HIGHWAY 401 HIGHWAY 15 UNDERPASS PROPOSED BRIDGE</p>	<p>SHEET</p>
<p>BOREHOLE LOCATIONS AND SOIL STRATA</p>	
 <p>MMM GROUP</p>	



KEYPLAN	
	L E G E N D

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

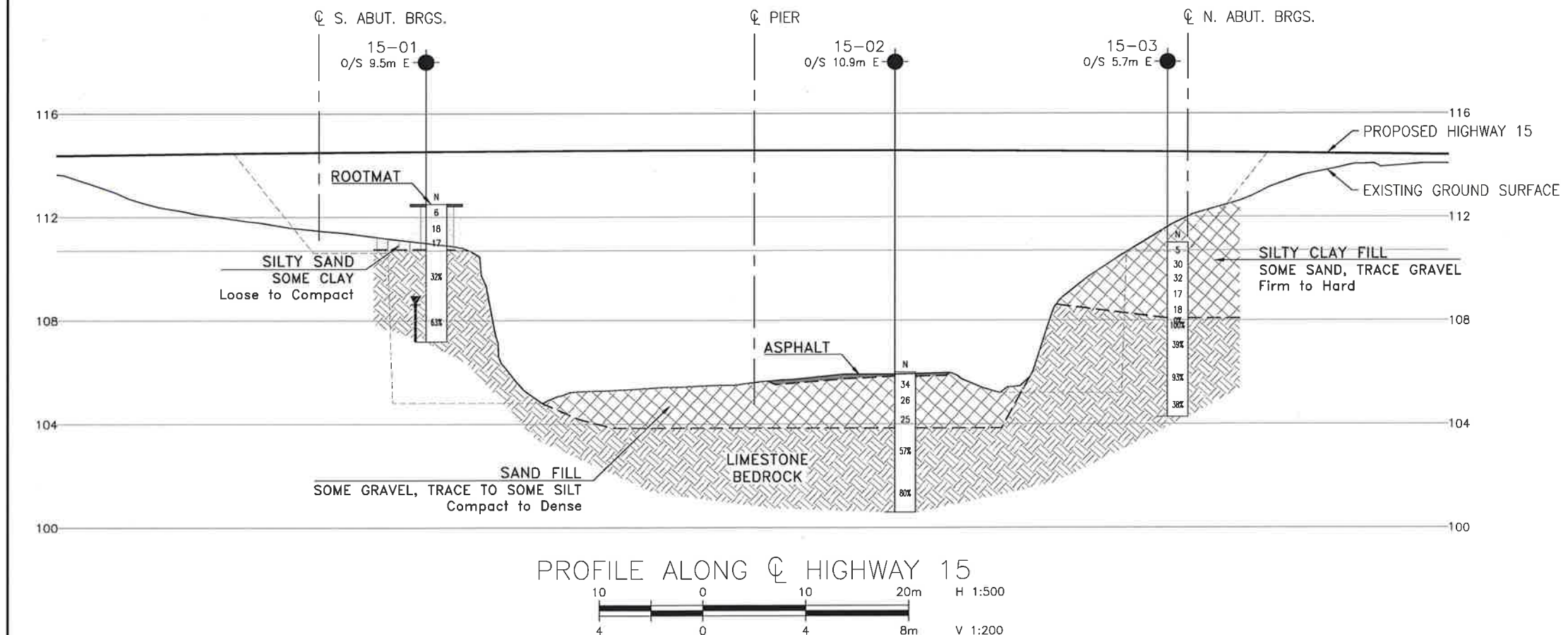
NO	ELEVATION	NORTHING	EASTING
15-01	112.5	4 905 406.7	310 380.9
15-02	106.1	4 905 451.0	310 390.9
15-03	111.0	4 905 477.9	310 390.8

[illegible]

-NOTES-

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

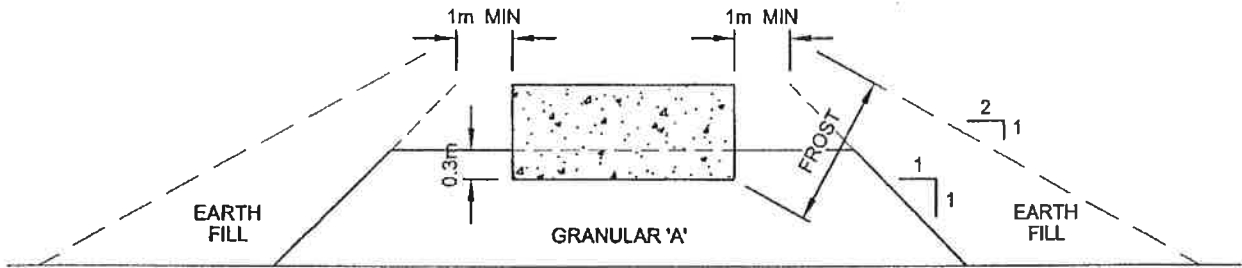
GEOCRES No. 31C-241



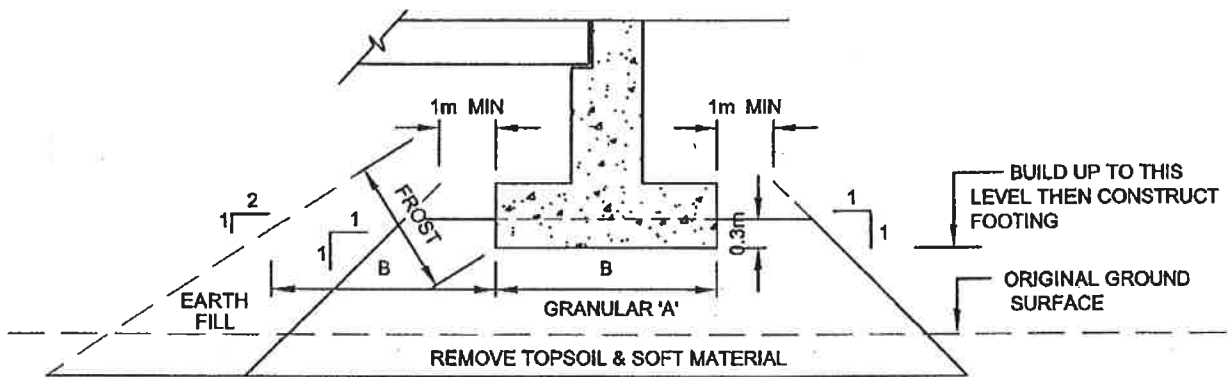
REVISIONS										
	DATE	BY				DESCRIPTION				
DESIGN	AP	CHK	PKC	CODE		LOAD			DATE	MAR 2018
DRAWN	MFA	CHK	AP	SITE	7-71	STRUCT		DWG	1	

Appendix E

Figure 1 – Abutment on Compacted Fill



CROSS-SECTION



LONGITUDINAL SECTION

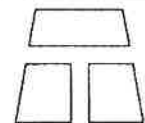
NOT TO SCALE

NOTES:

1. REMOVE TOPSOIL AND OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH FILL.
2. PLACE GRANULAR 'A' AND EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO O.P.S.S. 501.
3. CONSTRUCT CONCRETE FOOTING.
4. PLACE REMAINDER OF GRANULAR 'A' AND EARTH FILL AS REQUIRED.
5. SOURCE M.T.C. 1982.

ENGINEER	AEG
DRAWN	SS
DATE	April , 2004
APPROVED	PKC
SCALE	NTS

ABUTMENT ON COMPACTED FILL SHOWING
GRANULAR A CORE



THURBER

DWG. NO.

FIGURE 1

Appendix F
Comparison of Foundation Alternatives

COMPARISON OF FOUNDATION ALTERNATIVES

Spread Footings on Bedrock	Spread Footings on Granular Pad Constructed on Bedrock	Augered H-piles Socketed in Limestone Bedrock	Caissons / Drilled Shafts
<p>Advantages:</p> <ul style="list-style-type: none"> i. Relative ease of construction. ii. More cost effective than deep foundations. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Relative ease of construction. ii. More cost effective than deep foundations. iii. Allow for higher founding levels 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Higher geotechnical resistance comparing to spread footings ii. Installation less influenced by weather and groundwater compared to spread footing iii. Facilitate the integral abutment design. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Higher geotechnical resistance is available for caissons socketed into bedrock than for socketed piles. ii. Construction of caissons could continue in freezing weather.
<p>Disadvantages:</p> <ul style="list-style-type: none"> i. Potentially deep excavation at abutments ii. May require groundwater control. iii. Not feasible for integral abutment design. 	<p>Disadvantages:</p> <ul style="list-style-type: none"> i. Lower bearing capacity than for footings placed directly on bedrock. ii. Potentially deep excavation at abutments iii. May require groundwater control. iv. Not feasible for integral abutment design. 	<p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit costs than spread footings. ii. Piles will require sockets iii. iii. Potential difficulties penetrating hard limestone bedrock during coring. 	<p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit cost than for spread footings or H-piles. ii. Temporary liners will be required to install caissons through overburden soils. iii. Difficulty in sealing liners at base. iv. Potential difficulties penetrating hard limestone bedrock during coring of relatively large diameter caissons v. Difficulties in cleaning and inspecting bases.
Low risk of encountering problems during construction.	Low risk of encountering problems during construction.	High risk of encountering harder layers within limestone bedrock that would require additional procedures to advance the holes to the desired elevation.	High risk of encountering harder layers within limestone bedrock that would require additional procedures to advance the holes to the desired elevation.
RECOMMENDED	FEASIBLE	FEASIBLE	FEASIBLE