PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
INTERCHANGE IMPROVEMENTS AT KINGSTON ROAD 38
HIGHWAY 401
CITY OF KINGSTON, ONTARIO
G.W.P. 4049-11-00, SITE NO. 7-57

Geocres Number: 31C-240

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 Kingston Road 38 Interchange on Highway 401 in the City of Kingston, Ontario. The proposed improvements include replacement of the existing Highway 401 underpass structure, and realignment of 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 Kingston Road 38 (Gardiners Road) over Highway 401 at the existing interchange in the City of Kingston. At the project site, Highway 401 runs approximately in the northwest-southeast direction, while Kingston Road 38 runs generally north-south. For the purpose of this report, Highway 401 is assumed to run west-east. Kingston Road 38 consists of two lanes of traffic in each direction, and Highway 401 is a six-lane (three lanes in each direction) freeway. The existing interchange is in a “Parclo B” configuration, with on- and off-ramps located in the northwest and southeast quadrants of the interchange.

The existing underpass was constructed in 1960 and consists of a four-span, concrete T-beam deck, supported on a combination of spread footings and piles. The bridge spans are 14.6, 21.4, 21.4 and 14.6 m long, for a total length of 72 m. The underpass was previously rehabilitated in 2003, when it received an overlay, new parapet walls, and was converted to semi-integral abutments.

The surrounding lands are generally wooded with some clear areas. Commercial properties exist approximately 400 m south of Highway 401 along Centennial Drive, and some residential houses are located approximately 200 m north of Highway 401 along Jackson Mills Road. Collins Creek crosses under Highway 401 approximately 0.7 km west of the site.

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 near the north and south abutments and piers.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out on May 20 and 21, 2015 and consisted of drilling and sampling three boreholes, designated as Boreholes 38-01 to 38-03, and five bedrock probes, identified as R-01 to R-05. Boreholes 38-01 and 38-03 were drilled on exposed bedrock outcrops on the west side of the existing abutments. Borehole 38-02 was drilled through the existing Highway 401 median west of the existing alignment. Boreholes 38-01 to 38-03 were advanced to depths from 3.9 to 5.4 m below the existing ground surface. The bedrock probes were conducted at selected locations along the approximate alignments of the proposed ramps, and were advanced to depths from exposed bedrock at the ground surface to 1.7 m below the ground surface.

The approximate locations of the boreholes and bedrock probes are shown on the attached Borehole Location Plan on Figure D1 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 information provided to Thurber by MMM Group Limited.

A track-mounted CME 850 drill rig was used to advance the boreholes using NQ rock coring techniques to collect core samples of the bedrock. At Borehole 38-02, hollow stem augers were also utilized to penetrate the pavement structure and granular fill. Soil samples were obtained within the granular fill using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). The bedrock probes were advanced using hollow-stem augers, continuous SPTs, or hand-excavating.

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 38-01 to monitor the groundwater level after drilling. 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 boreholes are summarized in Table 3.1.

Table 3.1 – Borehole Completion Details

Borehole Location	Borehole Number	Borehole Depth/Base Elevation (m)	Piezometer Tip Depth/ Elevation (m)	Completion Details
North Abutment	38-01	3.9 / 107.9	3.9/ 107.9	#2 Well gravel from 3.9 m to 0.9 m and bentonite holeplug from 0.9 m to surface.
Central Pier/Highway 401 Median	38-02	5.4 / 105.2	None installed	Bentonite holeplug and cuttings to 0.15 m and asphalt patch from 0.15 m to surface.
South Abutment	38-03	3.9 / 112.4	None installed	Bentonite holeplug 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 on 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 and rock 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 typically comprises a thin layer of root mat or roadway asphalt and granular fill, occasionally overlying silty sand, which in turn are underlain by limestone bedrock. More detailed description of the individual strata are presented below.

5.1 Asphalt / Root mat

A 150 mm thick layer of asphalt was encountered at the surface of the Highway 401 median at Borehole 38-02.

At Borehole 38-01 and at bedrock probes R-01 to R-03 and R-05, a 50 mm to 100 mm thick root mat layer was encountered at the ground surface.

5.2 Granular Fill

Underlying the asphalt at Borehole 38-02 was a 1.0 m thick layer of granular road base extending to a depth of 1.2 m (Elev. 109.4 m). The granular fill consisted of sand with some gravel. SPT ‘N’ values recorded in the fill were 60 blows per 0.3 m penetration and 100 blows per 0.25 m penetration, indicating a very dense relative density. The measured moisture content was 1% to 2%.

5.3 Silty Sand

West of Kingston Road 38, Probes R-01 to R-03 and R-05 encountered a thin layer of silty sand with trace organic matter (rootlets) underlying the root mat. Some clay was noted in Probe R-03. The silty sand deposit ranged in thickness from 0.1 to 1.7 m. SPT ‘N’ values recorded in the silty sand ranged from 7 to 22 blows per 0.3 m penetration, indicating a loose to compact relative density. Higher ‘N’ values of 100 blows per 0.2 to 0.225 m penetration were obtained when the SPT encountered the bedrock surface. The measured moisture content of the silty sand ranged from 18% to 30%.

5.4 Bedrock

Limestone bedrock was encountered in all test holes. Bedrock was exposed at the ground surface at Boreholes 38-03 and covered by a thin root mat in Borehole 38-01. In Borehole 38-02, bedrock was encountered at 1.2 m depth beneath the fill. Bedrock surface was covered by a thin veneer of root mat in the Probes R-01 and R-02, and was outcropping in Probe R-04. In Probes R-03 and R-05, bedrock was inferred beneath the granular fill at depths of 0.8 m and 1.7 m, respectively. Bedrock was proved by coring in Boreholes 38-01 to 38-03. Table 5.1 summarizes the depths and elevations to the top of bedrock at the locations of boreholes and probes.

Table 5.1 – Depths and Elevations of Top of Bedrock

Borehole/Probe Location	Borehole/Probe Number	Top of Bedrock	
		Depth (m)	Elevation (m)
North Abutment	38-01	0.1	111.7
Highway 401 Median	38-02	1.2	109.4
South Abutment	38-03	0.0	116.3
W-N/S Ramp / N/S-E Ramp	R-01	0.1	124.4
W-N/S Ramp / N/S-E Ramp	R-02	0.1	123.6
N/S-E Ramp	R-03	0.8	113.8
W-N/S Ramp	R-04	0.0	112.3
E-N/S Ramp	R-05	1.7	103.3

The limestone bedrock was described as moderately to slightly weathered, thinly bedded with occasional seams of shale. Total Core Recovery (TCR) in the bedrock ranged from 78 % to 100%. The RQD values ranged from 8% to 88%, being generally between 29% and 72%. The RQD values indicate a very poor to fair rock quality rock, typically being a poor to fair quality rock.

The Fracture Index (FI) of the rock, expressed as number of fractures per 0.3 m of core run, ranged from 0 to greater than 10 with typical values between 2 and 8. Occasional highly fractured zones were generally encountered within the upper 1.5 m below the bedrock surface, and in those zones FI values greater than 10 were obtained.

The average unconfined compressive strength of the rock interpreted from point load tests conducted on core samples recovered from the boreholes ranged from 60 to 100 MPa, indicating a strong rock.

The unconfined compressive strength results 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.

5.5 Water Levels

Where possible, water levels were monitored in the open boreholes during drilling operations. Given that wash boring and coring methods were used to advance the boreholes, water levels recorded during or upon completion of drilling may not reflect natural groundwater levels. A standpipe piezometer was installed in one borehole to monitor the groundwater level after completion of drilling. The water levels measured in the piezometer and during drilling are presented in Table 5.2.

Table 5.2 – Water Level Measurements

Borehole	**Date**	**Water Level**		**Comment**
		Depth (m)	Elev. (m)	
38-01	May 21, 2015 October 6, 2015	3.2 3.8	108.6 108.0	In piezometer
38-02	May 20, 2015	N/A	-	-
38-03	May 21, 2015	N/A	-	-

The above 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 the boreholes. The drilling operations were supervised by Mr. Chris Murray 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 Alastair Gorman, 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 GENERAL

This report presents interpretation of the geotechnical data in the factual report and provides preliminary geotechnical recommendations for the proposed improvements to the Kingston Road 38 Interchange on Highway 401 in the City of Kingston, Ontario.

The existing interchange is in a 'Parclo B' configuration, with on- and off-ramps located in the northwest and southeast quadrants of the interchange. The existing underpass was constructed in 1960 and consists of four spans with a total length of 72 m. The underpass is a concrete T-beam bridge, supported on a combination of spread footings at the south abutment and pier, and piles at the north abutment.

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 39 m and 46 m span lengths, and a total length of 85.0 m between abutments. The new underpass is located immediately to the west of the existing bridge with a horizontal clearance of 3.0 m between the new and old underpasses.

In light of shallow and/or exposed limestone bedrock surface, spread footings are envisioned for the support of the new structure.

The existing approach fill height above the surrounding ground is approximately 6.0 m at the north abutment and 3.0 m at the south 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

In general, all boreholes drilled at this site encountered bedrock surface at shallow depths. The grey limestone bedrock was encountered at shallow depth at each foundation unit location. In Borehole 38-01 at the north abutment, the bedrock was covered by a thin veneer of root mat and was encountered at Elev. 111.7. In Borehole 38-02, advanced at the Highway 401 median, bedrock was overlain by a 1.2 m layer of fill, and was encountered at Elev. 109.4. The bedrock surface at the south abutment was encountered outcropping at much higher elevation, namely at Elev. 116.3.

The groundwater level in the standpipe piezometer installed in Borehole 38-01 and sealed in the limestone bedrock was measured at 3.2 m depth immediately after piezometer installation and at 3.8 m depth or Elev. 108.0 on October 6, 2015. The groundwater level at the bridge site will fluctuate and, in general, it 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 the granular pad
- Steel H-piles socketed into limestone bedrock, and
- Caissons socketed into bedrock.

Limestone bedrock underlies the bridge site at 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 shallow depth to bedrock, socketing piles in the bedrock would probably be required to develop sufficient lateral resistance.

Caissons socketed into bedrock could be also considered at this site, however it will require coring of large diameter holes in the relatively 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)
North Abutment	38-01	0.1 / 111.7	0.3	111.4
Central Pier	38-02	1.2 / 109.4	0.3	109.1
South Abutment	38-03	0.0 / 116.3	0.3	114.6

The actual founding elevation at specific foundation locations must be confirmed during detailed design stage.

The footings founded on weathered limestone bedrock could 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). 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 outer 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.

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 elevation at the north abutment, if this is beneficial for the design, and central pier, if required. It is neither recommended nor beneficial at the south abutment.

The engineered fill pad should consist of OPSS Granular “A” or Granular B Type II placed in 150 mm lifts and compacted to 100% of its SPMDD at ±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 sketch illustrating a granular pad foundation is enclosed in Appendix E.

A 2 m wide footing bearing on an engineered fill placed on bedrock may be designed for the following capacities:

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

A foundation consisting of H-piles socketed into bedrock is feasible at the north abutment and would permit design of an integral abutment, if this is considered beneficial to the overall design.

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. At the north abutment, the pile would be installed through the compacted fill materials.

Abutment piles to be installed through the compacted fill will have to be surrounded in the upper 3 m of the piles 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, to facilitate the required flexibility. 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 will have to be founded in bedrock, as the resistance developed in the relatively shallow overburden soils (fill and sand) will be 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 platform.

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

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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. 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 30 MPa.

The structural capacity of the pile material will govern the design. The structural designer should check socket design to ensure sufficient base fixity.

8.4.3 Augered Pile Installation

Pile installation should be in general accordance with OPSS 903. The pre-drilled 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 may be used to support the socket sidewalls of the predrilled holes in the overburden soils and to minimize groundwater inflow.

The pile socket excavation should be dewatered to allow cleaning of the base 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 should be of sufficient diameter to permit placement of the CSPs required in the integral abutment design.

8.5 Augered Caissons (Drilled Shafts)

Augered caisson foundations socketed into limestone bedrock may be considered to support the bridge at this site. The bedrock surface indicated in Table 8.1 could be used for the preliminary design of the socketed caissons, if this option is selected.

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 Foundation

The spread footings founded on limestone bedrock is the preferred foundation option for this bridge, based on geotechnical considerations, cost effectiveness and constructability.

8.7 Erosion Protection

Erosion protection should be provided over all earth or granular surfaces. A vegetation cover should be established on all exposed surfaces to protect against surficial erosion, 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 footings or pile caps in fill or overburden.

9 LATERAL EARTH PRESSURE

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 wall are dependent on the material used as backfill. Typical values are given in Table 9.1.

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, 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 – Coefficients of Lateral Earth Pressure

Loading Condition	**Earth Pressure Coefficient (K)**			
	OPSS Granular A or Granular B Type II $\phi = 35°, \gamma = 22.8$ **kN/m**^{3}		**OPSS Granular B Type I** $\phi = 32°, \gamma = 21.2$ **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:

- 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 used.

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 $\phi = 35°, \gamma = 22.8$ kN/m^3		OPSS Granular B Type I or Existing Embankment Fill $\phi = 32°, \gamma = 21.2$ 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 embankments will be as much as 6.0 m in height. The ramps will require combination of embankments and cuts. The embankments carrying the ramps will be relatively low with the heights not exceeding 4 m. Ramps at this interchange will require significant lengths of cuts, predominantly in the limestone bedrock. The full length of the alignment of the N-E Ramp and approximately half of the length of W-NS Ramp will require cuts as deep as 9 m. Shorter lengths of cuts will be required along the NS-W and E-N Ramps.

All rock cut slopes could be constructed vertically to as much as 10 m height (if required), and above that height the cuts should be flattened to an inclination of 1H:4V.

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

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 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.

The new embankment 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.

In order to improve interaction between the existing and newly placed embankment fill, 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 allow to discharge on the embankment slopes in an uncontrolled manner.

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 existing embankment fill and through shallow native deposits overlying the bedrock. The groundwater level as measured in the piezometer on the north side of the structure was approximately at 3.8 m depth below the ground surface or at Elev. 108.0. However, perched water may be present in the embankment fill. 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 south abutment, 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 excavation.

In order to reduce the potential for claims related to difficult rock excavation, test pits should be excavated during the tendering stage to allow the contractor to make his own assessment of the appropriate equipment required to excavate the limestone bedrock.

Ramps at this interchange will require significant lengths of cuts, predominantly in the limestone bedrock and as deep as 9 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 may 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 adjacent to the existing structure and that the traffic on Kingston Road 38 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

During construction, the Contract Administrator (CA) should retain an experienced geotechnical engineer to observe foundation construction activities and to provide advice to the CA regarding any issues that need to be referred to the design team.

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/caisson installation equipment should be capable of coring the limestone and dislodging/removing any obstructions such as cobbles, boulders and other obstructions 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 DETAILED 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 two 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.

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Mar 15, 2016
PROVINCE OF ONTARIO

Dr. P.K. Chatterji, Ph.D., P.Eng.
Review Principal

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.

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)	(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			

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RECORD OF BOREHOLE No 38-01

METRIC

GWP# 4049-11-00
LOCATION Kingston Rd. 38 Interchange N 4 905 093.0 E 299 195.8
ORIGINATED BY CAM
HWY 401/38
BOREHOLE TYPE NQ Coring
COMPILED BY AN
DATUM Geodetic
DATE 2015.05.20 - 2015.05.20
CHECKED BY MEF

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	FI	REMARKS & GRAIN SIZE DISTRIBUTION (%)
111.8	GROUND SURFACE						
0.0	**ROOTMAT:** (100mm)						
0.1	**BEDROCK**: limestone, moderately to slightly weathered, thinly bedded, occasional shale seams, strong, grey Highly fractured zones at depths: 0.30m to 0.45m 0.60m to 0.78m					>10 >10 >10	
					111		
		1	RUN			3 2 0	RUN #1 TCR=90% SCR=73% RQD=49% UCS=90MPa (Average)
					110	1 7 1	
					109	3	
		2	RUN			3 0	RUN #2 TCR=96% SCR=89% RQD=65% UCS=66MPa (Average)
107.9					108		
3.9	END OF BOREHOLE AT 3.9m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) May 21/15 3.2 108.6 Oct 06/15 3.8 108.0						

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; 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; GR SA SI CL

$+^3, \times^3$: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No 38-02

METRIC

GWP# 4049-11-00 LOCATION Kingston Rd. 38 Interchange N 4 905 055.6 E 299 196.1 ORIGINATED BY CAM

HWY 401/38 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN

DATUM Geodetic DATE 2015.05.20 - 2015.05.20 CHECKED BY MEF

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT (FI)	REMARKS & GRAIN SIZE DISTRIBUTION (%)
110.6	GROUND SURFACE							
0.0	**ASPHALT:** (150mm)							
0.2	**SAND**, some gravel, Very Dense, Grey, Dry (FILL)	1	SS	60	110	o		
		2	SS	100/0.250		o		
109.4 / 1.2	**BEDROCK**: limestone, moderately to slightly weathered, thinly bedded, occasional shale seams, strong, grey. Highly fractured zones at depths: 1.20m to 1.50m	1	RUN		109		>10, >10, 7, 2	RUN #1 TCR=100% SCR=56% RQD=29% UCS=64MPa (Average)
	100mm thick shale seam at 3.1m depth	2	RUN		108, 107		3, 1, 8, 0, 2	RUN #2 TCR=98% SCR=93% RQD=72% UCS=60MPa (Average)
	50mm thick shale seam at 5.1m	3	RUN		106		2, 1, 2, 2, 4	RUN #3 TCR=100% SCR=97% RQD=88% UCS=88MPa (Average)
105.2 / 5.4	END OF BOREHOLE AT 5.4m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND CUTTINGS TO 0.15m, THEN COLD PATCH TO SURFACE.							

Column headings also include: STRAT PLOT; GROUND WATER CONDITIONS; 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; GR SA SI CL.

+ ³, × ³: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No 38-03

METRIC

GWP# 4049-11-00	LOCATION Kingston Rd. 38 Interchange N 4 905 012.8 E 299 182.3	ORIGINATED BY CAM
HWY 401/38	BOREHOLE TYPE NQ Coring	COMPILED BY AN
DATUM Geodetic	DATE 2015.05.21 - 2015.05.21	CHECKED BY MEF

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	SAMPLE TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³ (FI)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
116.3	GROUND SURFACE								
0.0	**BEDROCK**: limestone, moderately to slightly weathered, thinly bedded, occasional shale seams, strong, grey						116	FI >10 6 8	RUN #1 TCR=78% SCR=42% RQD=8% UCS=66MPa (Average)
	Highly fractured zones at depths: 1.05m to 1.20m		1	RUN			115	>10 2 7 8	
			2	RUN			114 113	2 2 2 2	RUN #2 TCR=100% SCR=90% RQD=60% UCS=100MPa (Average)
112.4	12mm thick shale seams at 3.7m and 3.8m depth								
3.9	END OF BOREHOLE AT 3.9m. BOREHOLE BACKFILLED TO SURFACE WITH BENTONITE HOLEPLUG.								

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

$+^3$, $\times^3$: Numbers refer to Sensitivity

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

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RECORD OF BOREHOLE No R-01

METRIC

GWP#	4049-11-00	LOCATION	Kingston Rd. 38 Interchange N 4 904 888.9 E 299 090.3	ORIGINATED BY	CAM
HWY	401	BOREHOLE TYPE	Hand Shovel	COMPILED BY	AN
DATUM	Geodetic	DATE	2015.05.21 - 2015.05.21	CHECKED BY	MEF

SOIL PROFILE			SAMPLES			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
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
124.5	GROUND SURFACE										
0.0 / 0.0	**ROOTMAT:** (50mm)		1	GS					○		
0.1	Silty **SAND**, with organics Brown Moist										
	END OF BOREHOLE AT 0.1m ON LIMESTONE BEDROCK. BOREHOLE BACKFILLED TO SURFACE WITH CUTTINGS.										

$+^3, \times^3$: Numbers refer to Sensitivity

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

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RECORD OF BOREHOLE No R-02

METRIC

GWP#	4049-11-00	LOCATION	Kingston Rd. 38 Interchange N 4 904 945.3 E 299 022.6	ORIGINATED BY	CAM
HWY	401	BOREHOLE TYPE	Hand Shovel	COMPILED BY	AN
DATUM	Geodetic	DATE	2015.05.21 - 2015.05.21	CHECKED BY	MEF

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES 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
123.7	GROUND SURFACE										
0.0 / 0.0	**ROOTMAT:** (50mm)		1	GS					○		
0.1	Silty **SAND**, with organics Brown Moist										
	END OF BOREHOLE AT 0.1m ON LIMESTONE BEDROCK. BOREHOLE BACKFILLED TO SURFACE WITH CUTTINGS.										

+ 3, × 3: Numbers refer to Sensitivity

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

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RECORD OF BOREHOLE No R-03

METRIC

GWP# 4049-11-00 | LOCATION Kingston Rd. 38 Interchange N 4 905 053.3 E 299 086.3 | ORIGINATED BY CAM

HWY 401 | BOREHOLE TYPE Continuous Split Spoon | COMPILED BY AN

DATUM Geodetic | DATE 2015.05.21 - 2015.05.21 | CHECKED BY MEF

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT PLOT	SAMPLES 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
114.6	GROUND SURFACE										
0.0	**ROOTMAT:** (50mm)										
	Silty **SAND**, some clay Loose Brown Moist		1	SS	7		114		○		
113.8			2	SS	100/ 0.225				○		
0.8	END OF BOREHOLE AT 0.8m ON PROBABLE BEDROCK. BOREHOLE BACKFILLED TO SURFACE WITH CUTTINGS.										

$+^3$, $\times^3$: Numbers refer to Sensitivity

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

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RECORD OF BOREHOLE No R-04

METRIC

GWP# 4049-11-00	LOCATION Kingston Rd. 38 Interchange N 4 905 128.8 E 298 986.7	ORIGINATED BY CAM
HWY 401	BOREHOLE TYPE Hand Shovel	COMPILED BY AN
DATUM Geodetic	DATE 2015.05.21 - 2015.05.21	CHECKED BY MEF

SOIL PROFILE			SAMPLES			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/m 3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
112.3	GROUND SURFACE										
0.0	Visible limestone at surface.										

+ 3 , × 3 : Numbers refer to Sensitivity

15 20 5 10 (%) STRAIN AT FAILURE

ONTMT4S 1256.GPJ 2015TEMPLATE(MTO).GDT 10/8/15

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THURBER

RECORD OF BOREHOLE No R-05

METRIC

GWP# 4049-11-00
LOCATION Kingston Rd. 38 Interchange N 4 905 227.0 E 299 061.2
ORIGINATED BY CAM

HWY 401
BOREHOLE TYPE Hollow Stem Augers
COMPILED BY AN

DATUM Geodetic
DATE 2015.05.21 - 2015.05.21
CHECKED BY MEF

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
105.0	GROUND SURFACE										
0.0 / 0.0	**ROOTMAT:** (50mm)										
	Silty **SAND** Loose to Compact Brown Moist		1	SS	9						
			2	SS	22		104		○ (approx. 8)		
103.3			3	SS	100/ 0.200						
1.7	END OF BOREHOLE AT 1.7m ON PROBABLE BEDROCK. BOREHOLE DRY ON COMPLETION. BOREHOLE BACKFILLED TO SURFACE WITH BENTONTIE HOLEPLUG AND CUTTINGS TO SURFACE.										

$+^3, \times^3$: Numbers refer to Sensitivity

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

Appendix B

Point Load Test Results and Rock Core Photographs

POINT LOAD TEST SHEET

THURBER

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 :** 38-1	**Tester :**	CAM

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	0.1	A	11.94	47.6	50.0	93.6	Limestone	Strong
2	1	0.2	D	6.62	47.6	70.0	65.0	Limestone	Strong
3	1	1.1	D	11.02	47.6	80.0	108.2	Limestone	Very Strong
4	1	1.3	A	8.89	47.6	70.0	53.7	Limestone	Strong
5	1	1.8	D	11.47	47.6	80.0	112.7	Limestone	Very Strong
6	1	2.4	D	11.14	47.6	120.0	109.4	Limestone	Very Strong
7	2	3.1	D	3.07	47.6	100.0	30.2	Shale Seam	Medium Strong
8	2	3.1	A	12.53	47.6	50.0	98.2	Limestone	Strong
9	2	3.9	D	7.05	47.6	80.0	69.2	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.

THURBER

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 :** 38-2	**Tester :**	CAM

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	1.7	A	7.41	47.6	50.0	58.1	Limestone	Strong
2	1	2.0	D	8.12	47.6	70.0	79.8	Limestone	Strong
3	2	2.4	D	6.74	47.6	75.0	66.2	Limestone	Strong
4	2	3.0	D	3.67	47.6	95.0	36.0	Shale Seam	Medium Strong
5	2	3.0	A	8.80	47.6	50.0	69.0	Limestone	Strong
6	2	3.6	D	7.17	47.6	130.0	70.4	Limestone	Strong
7	3	4.0	D	6.74	47.6	80.0	66.2	Limestone	Strong
8	3	4.3	D	8.70	47.6	95.0	85.4	Limestone	Strong
9	3	4.7	D	11.55	47.6	130.0	113.4	Limestone	Very Strong
10	3	5.3	D	9.78	47.6	135.0	96.1	Limestone	Strong
11	3	5.4	A	11.84	47.6	60.0	80.6	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.

THURBER

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 :** 38-3	**Tester :**	CAM

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	UCS (MPa)	Rock Type	Notes
1	1	0.4	D	4.04	47.6	70.0	39.7	Weathered	Medium Strong
2	1	0.5	A	11.74	47.6	50.0	92.0	Limestone	Strong
3	2	1.4	D	13.27	47.6	120.0	130.3	Limestone	Very Strong
4	2	2.3	D	8.13	47.6	70.0	79.9	Limestone	Strong
5	2	2.6	A	17.61	47.6	60.0	119.9	Limestone	Very Strong
6	2	3.1	D	5.23	47.6	90.0	51.4	Limestone	Strong
7	2	3.4	D	12.25	47.6	115.0	120.3	Limestone	Very 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.

Rock Core from Borehole 38-01

Rock Core from Borehole 38-02

Rock Core from Borehole 38-03

Appendix C

Site Photographs

Photograph 1 – West side of existing bridge, looking northeast near new south abutment; note exposed bedrock surface

Photograph 2 – West of existing bridge, looking north along approximate alignment of new underpass

Photograph 3 – South approach to existing bridge, looking north

Photograph 4 – Looking north along approximate alignment of new approach

Photograph 5 – West side of existing bridge, looking southeast; note rock cut

Photograph 6 – West of existing bridge, looking south along approximate alignment of new underpass near new north abutment, showing rock cut

Photograph 7 – North approach to existing bridge, looking south

Photograph 8 – Looking south along approximate alignment of new bridge approach

Appendix D

Borehole Locations and Soil Strata Drawing

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

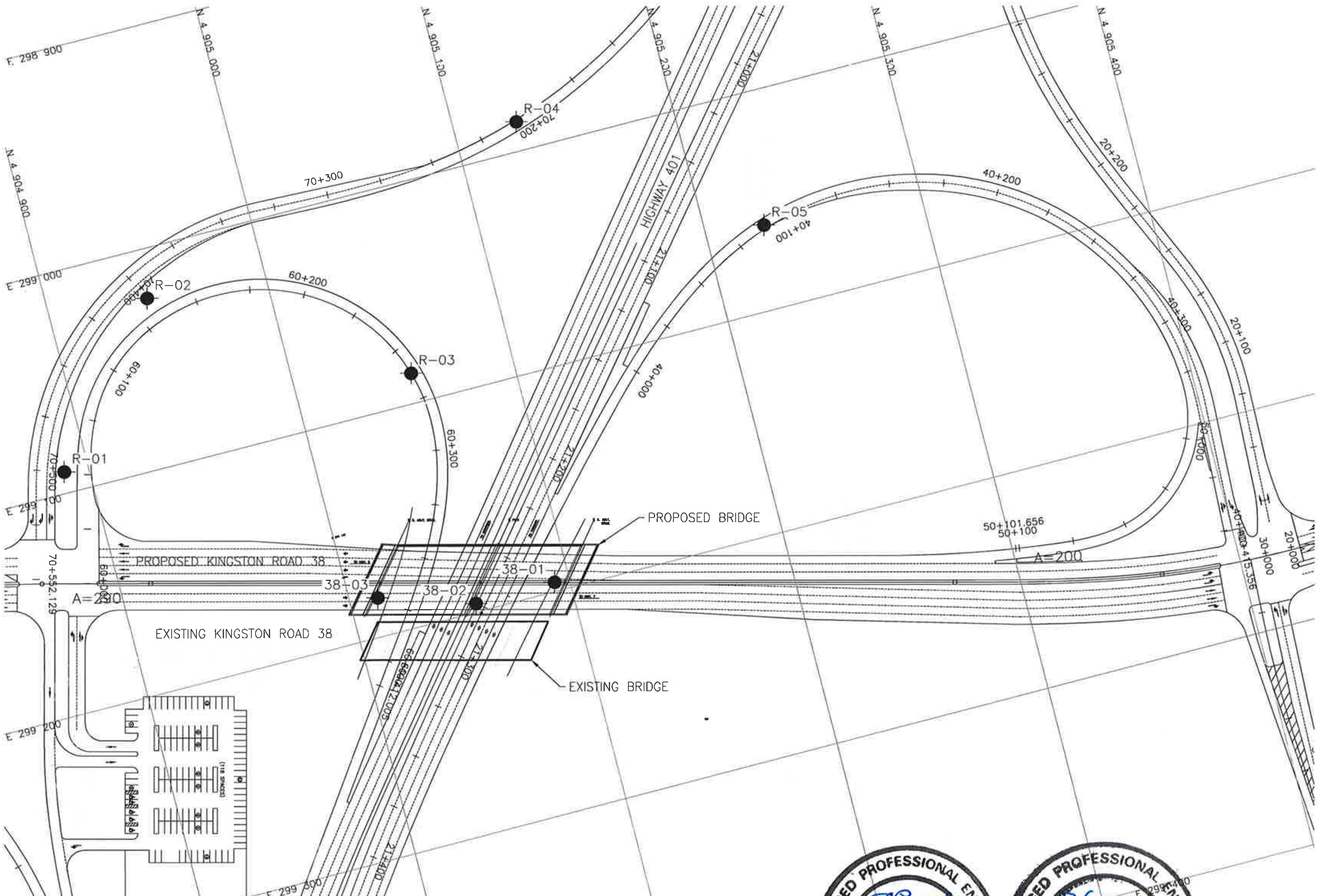
CONT No
WP No 4049-11-00

SHEET

HIGHWAY 401 AND KINGSTON ROAD 38 INTERCHANGE
BOREHOLE LOCATIONS PLAN

MMM GROUP

THURBER ENGINEERING LTD.



PLAN

40 0 40 80m
SCALE 1:2000

KEYPLAN

LEGEND

Symbol	Description
	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
38-01	111.8	4 905 093.0	299 195.8
38-02	110.6	4 905 055.6	299 196.1
38-03	116.3	4 905 012.8	299 182.3
R-01	124.5	4 904 888.9	299 090.3
R-02	123.7	4 904 945.3	299 022.6
R-03	114.6	4 905 053.3	299 086.3
R-04	112.3	4 905 128.8	298 986.7
R-05	105.0	4 905 227.0	299 061.2

-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-240

LICENSED PROFESSIONAL ENGINEER
A. PIASCIK
Mar 15/16
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Mar 15, 2016
PROVINCE OF ONTARIO

REVISIONS	DATE	BY	DESCRIPTION		
DESIGN AP	CHK PKC	CODE	LOAD	DATE MAR 2016	
DRAWN MFA	CHK AP	SITE 7-57	STRUCT	FIGURE D1	

FILENAME: H:\Drafting\19\5161\256\ted1256-BoreholePlan(KingstonRd38).dwg
PLOTDATE: 3/14/2016 3:09 PM

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

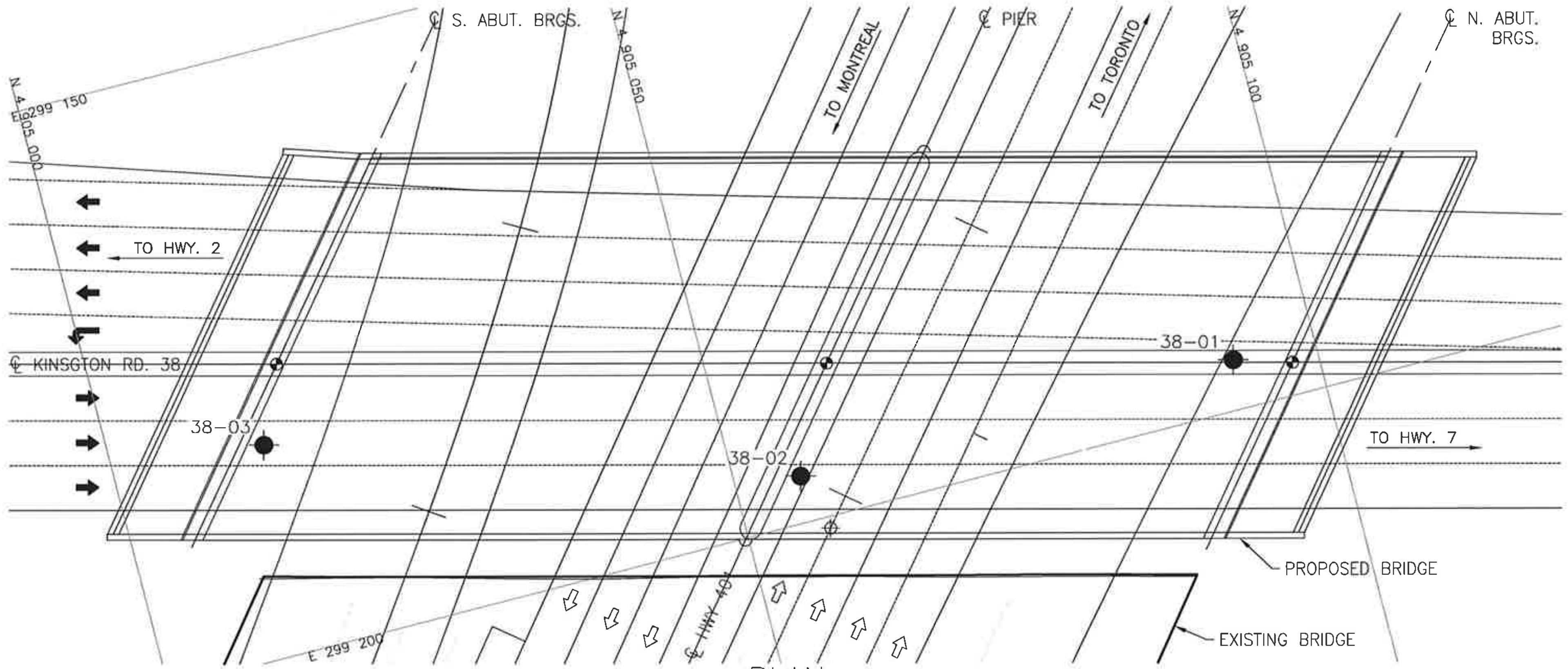
CONT No
WP No 4049-11-00

HIGHWAY 401
KINGSTON ROAD 38
PROPOSED BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

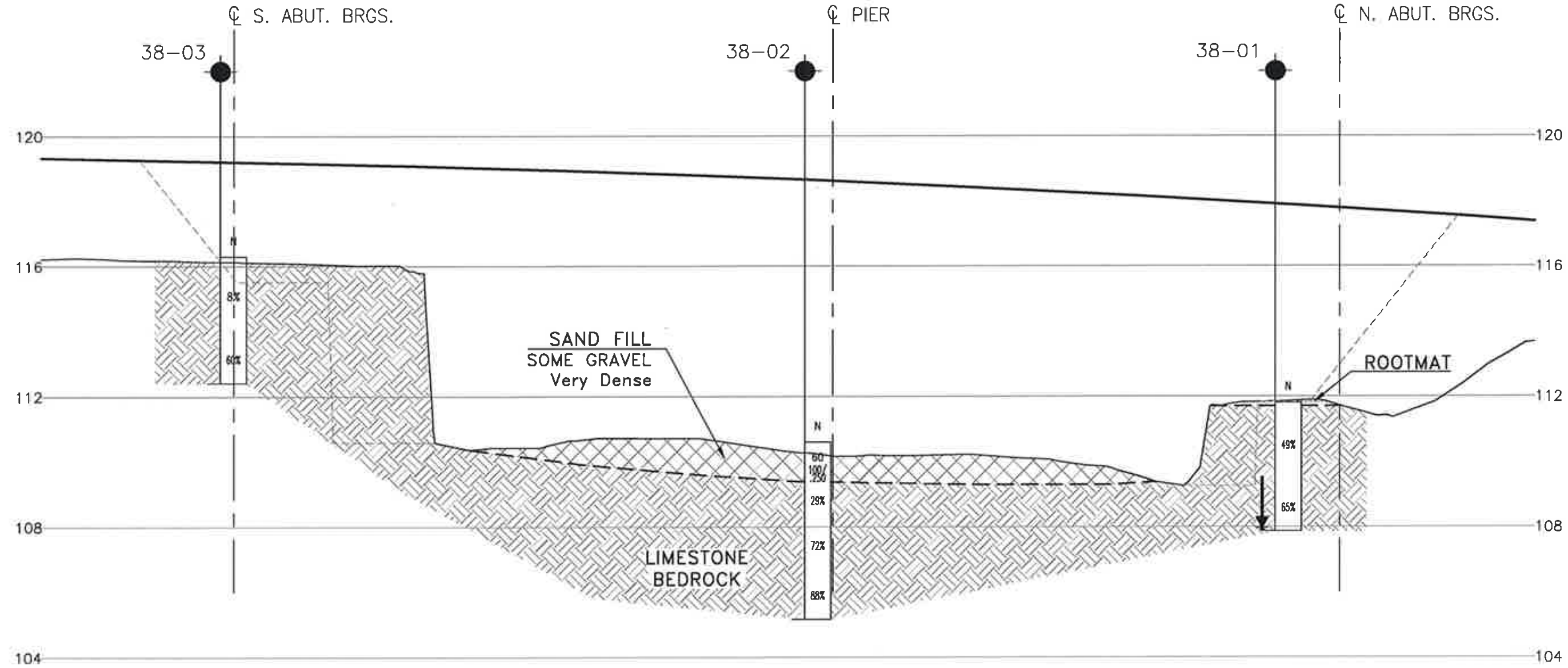
MMM GROUP

THURBER ENGINEERING LTD



PLAN

10 0 10 20m
SCALE 1:500



PROFILE

10 0 10 20m H 1:500
4 0 4 8m V 1:200

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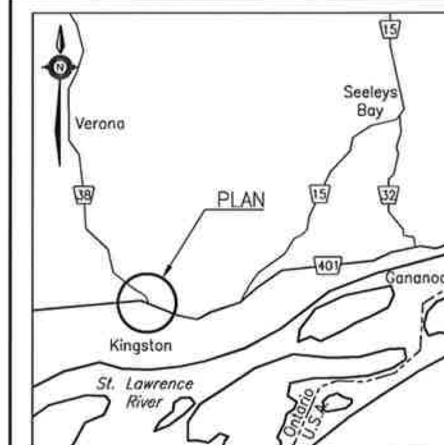
P. K. CHATTERJI
Mar 15, 2016
PROVINCE OF ONTARIO



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
38-01	111.8	4 905 093.0	299 195.8
38-02	110.6	4 905 055.6	299 196.1
38-03	116.3	4 905 012.8	299 182.3

-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-240

REVISIONS	DATE	BY	DESCRIPTION

DESIGN AP	CHK PKC	CODE	LOAD	DATE MAR 2016
DRAWN MFA	CHK AP	SITE 7-57	STRUCT	DWG 1

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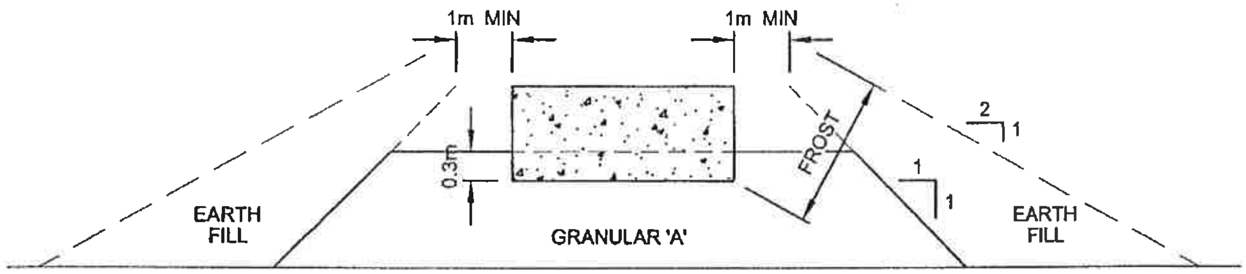
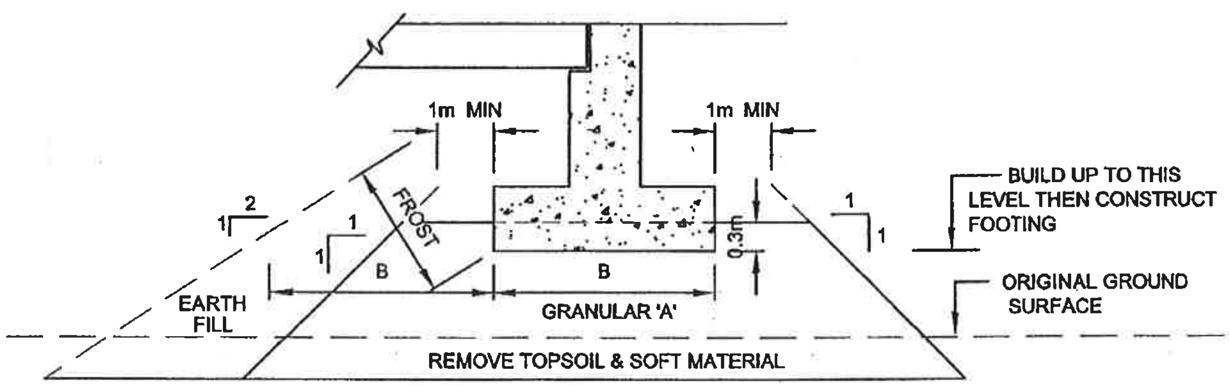
Appendix E
Figure 1 – Abutment on Compacted Fill

March , 2004



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	ABUTMENT ON COMPACTED FILL SHOWING GRANULAR A CORE	THURBER
DRAWN SS		
DATE April , 2004		
APPROVED PKC		DWG. NO. FIGURE 1
SCALE NTS		

TED35146 .DWG

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
Advantages: i. Relative ease of construction. ii. More cost effective than deep foundations.	***Advantages:*** i. Relative ease of construction. ii. More cost effective than deep foundations.	***Advantages:*** i. Facilitate the integral abutment design. ii. Installation less influenced by weather and groundwater compared to spread footing	***Advantages:*** i. Higher geotechnical resistance is available for caissons socketed into bedrock than for socketed piles. ii. Construction of caissons could continue in freezing weather. iii. Less construction space required than for spread footings
Disadvantages: i. Potentially deep excavation at the north abutment ii. May require groundwater control. iii. Not feasible for integral abutment design.	***Disadvantages:*** i. Lower bearing capacity than for footings placed directly on bedrock. ii. Potentially deep excavation at north abutment iii. May require groundwater control. iv. Not feasible for integral abutment design v. Not feasible for the south abutment support.	***Disadvantages:*** i. Higher unit costs than spread footings. ii. Piles will require significant length of coring for sockets iii. Potential difficulties penetrating hard limestone bedrock during coring.	***Disadvantages:*** i. Higher unit cost than for spread footings or H-piles. ii. Temporary liners will be required to install caissons through cohesionless soils. iii. Difficulty in sealing liners at the interface with overburden soils. 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**

THURBER