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PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
PROPOSED TWINNING AND NORTH APPROACH EMBANKMENT
HIGHWAY 62 NORRIS WHITNEY BRIDGE EA
BELLVILLE, ONTARIO
SITE No. 28-28
G.W.P. 4194-15-00

GEOCRES Number: 31C-308

Latitude: 44.152439°
Longitude: -77.388059°

Report

to

WSP

Date: July 8, 2021
File: 11566

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents a summary of the factual findings from a preliminary foundation investigation carried out at the proposed north approach embankment for bridge twinning by Thurber Engineering Ltd. for the existing Norris Whitney Bridge on Highway 62 in the Geographic Township of Ameliasburgh – Municipality of Belleville and Prince Edward County, Ontario.

The purpose of the investigation was to explore the subsurface conditions at selected locations at the north approach which will be located in a landfill area and, based on the data obtained, provide a borehole location plan, records of boreholes, stratigraphic profiles, and a written description of the subsurface conditions. A model of the subsurface conditions was developed to describe the geotechnical conditions influencing the preliminary design of the north approach embankments. No boreholes were advanced for the proposed bridge twinning. Preliminary foundation recommendations for the twinning bridge are included based on archived subsurface data.

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

There is no Geocres information directly related to the north approach area. In preparation of this report, reference has been made to information on subsurface conditions summarized in a memorandum prepared by Thurber for the rehabilitation of the existing bridge. This document includes foundation references and archive design drawings, and its title is as follows:

- Technical Memorandum titled "Preliminary Foundation Investigation and Design, Norris Whitney Bridge Rehabilitation (Site 28-28)", prepared by Thurber, File 19-4406-20, dated April 15, 2016 (Reference 1). This document is included in Appendix H.

2 SITE DESCRIPTION

The Norris Whitney Bridge carries Highway 62 over the Bay of Quinte between Belleville at the north end and the hamlet of Rossmore at the south end. The RFP information indicates that the existing bridge was constructed in 1982 and consists of an 11-span steel plate I-girder bridge with a total length of 881 m (58 m + 9 x 85 m + 58 m). The bridge has an overall deck width of 12 m and accommodates two lanes of traffic and a pedestrian sidewalk.

The natural terrain in the vicinity of the bridge is generally flat. It is understood that Zwick's island, located at the north end of the bridge on the Bay of Quinte, is the location of a closed landfill site which was operational in the 1950's and 60's. A dyking system was reportedly used to construct the landfill with dykes built out into the bay and refuse deposited behind the dykes. The landfill closed in 1971 and converted to parkland owned and operated by the City of Belleville.

Visual observation at the site and cross-sections provided by WSP indicate the presence of a berm at the east-facing slope of the existing north approach embankment. There is no available documentation of the presence and purpose of this berm.

It is understood that the new twinning bridge, which will carry the NBL, is to be located to the north of the existing bridge. Given the configuration of the shoreline and in order to minimize encroachment onto the landfill area, an additional bridge span is proposed such that the north approach and abutment would be closer to the northerly limits of the landfill. The actual limits of the landfill are unknown. Selected site photographs are included in Appendix D.

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

3 SUBSURFACE CONDITIONS

A previous site investigation was conducted by MTO in 1977 for the proposed construction of the existing high level bridge. A summary of the subsurface conditions previously established is included in Reference 1 in Appendix H.

The current investigation, consisting of a preliminary site investigation for the proposed NBL north approach embankment, was completed between November 16 and 17, 2020. A total of 3 boreholes (NW20-01 to NW20-03) were advanced to depths ranging from 8.5 m to 13.4 m during the investigation. Bedrock coring was completed in Borehole NW20-01 located near the proposed north abutment to confirm bedrock. The approximate locations of the completed boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix E. The coordinates and elevations of the boreholes are provided on the individual Record of Borehole Sheets in Appendix A.

Prior to commencing the site investigation, utility clearances were obtained for all borehole locations.

A track-mounted CME55 drill rig was used in conjunction with hollow-stem augers with an outside diameter of 108 mm (4-1/4-inch) and NQ (47.6 mm inside diameter) coring to advance the boreholes through the overburden and bedrock, respectively. In general, soil samples were obtained at selected intervals using a 50 mm diameter split spoon sampler in conjunction with the Standard Penetration Testing (SPT) in general accordance with ASTM D1586. An MTO 'N' size vane was used to carry out in-situ vane shear tests at selected locations in the silty clay. Thin-walled Shelby tube samples of the cohesive soils were also recovered where appropriate.

The drilling, sampling and in-situ testing 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. Results of field drilling and sampling are presented on the Record of Borehole sheets in Appendix A.

Groundwater levels were observed in the open boreholes during drilling operations and measured in standpipe piezometers installed in Boreholes NW20-01 and NW20-02. The piezometers consisted of 19 mm and 50 mm diameter Schedule 40 PVC pipes with 3.0 m long slotted screens enclosed in a column of filter sand. All boreholes without standpipe piezometer were backfilled upon completion of drilling in general accordance with O.Reg. 903. Once the final readings are taken, the two piezometers installed at the site will be decommissioned in general accordance with O.Reg. 903.

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

4 LABORATORY TESTING

The recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. Selected soil samples were also subjected to grain size analysis and Atterberg Limits testing. All the laboratory tests were carried out in accordance with MTO and/or ASTM Standards, as appropriate. The results of the laboratory testing are summarized on the Record of Borehole sheets and figures in Appendices A and B.

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

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5 DESCRIPTION OF SUBSURFACE CONDITIONS

Details of the encountered soil and rock stratigraphy are presented on the Record of Borehole sheets included in Appendix A, and on the Borehole Locations and Soil Strata drawings in Appendix E. A general description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole Sheets governs any interpretation of the site conditions. It must be recognized and anticipated that subsurface conditions may vary between and beyond the borehole locations.

In general, the subsurface stratigraphy encountered in the boreholes typically consisted of compact to dense cohesionless sand and gravel fill and stiff to very stiff silty clay fill overlying native stiff silty clay and hard silty clay till. The cohesive soils are underlain by limestone bedrock at about 9 m to 10 m depth. Deleterious materials including decayed wood fibres, rubber pieces, charcoal fragments, debris/brick pieces, and metal fragments were noted within the fill.

A detailed description of the subsurface conditions is presented in the following sections.

5.1 Topsoil

Topsoil was encountered at ground surface in Boreholes NW20-02 and NW20-03. The measured thickness of the topsoil ranged from 100 mm to 150 mm.

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

5.2 Asphalt

Asphalt was encountered at ground surface in Borehole NW20-01, which was advanced through the existing pathway east of the highway. The measured thickness of the asphalt was 75 mm.

5.3 Fill

Granular fill consisting of gravelly sand to sand and gravel containing crushed gravel was encountered underlying the asphalt in Borehole NW20-01 and underlying topsoil in Borehole NW20-02. A layer of sandy silt fill was encountered underlying topsoil in Borehole NW20-03. Occasional roots and rootlets, decayed wood fibres, rubber pieces, occasional boulder fragments, metal and charcoal fragments, debris and brick pieces were noted within the granular fill. Occasional roots and rootlets were noted within the sandy silt fill. These inclusions in the fill likely represent part of the refuse dump. Hydrocarbon odour was noted below 1.8 m depth in Borehole NW20-01.

The thickness of the granular fill ranged from 1.7 m to 4.5 m and this fill extended to depths between 4.6 m and 6.1 m below ground surface (Elevations 72.3 to 70.8). The sandy silt fill was approximately 1.2 m thick and its base was located at 1.4 m depth (Elev. 75.5 m).

The SPT-N values recorded in the granular fill ranged from 10 blows per 0.3 m of penetration to 50 blows per 0.125 m of penetration indicating compact to very dense conditions. The SPT-N values recorded in the sandy silt fill ranged from 13 to 15 blows per 0.3 m of penetration indicating a compact state. Measured moisture contents in the granular fill ranged from 2 percent to 38 percent. The high values are associated with fill with higher organic content.

Silty clay fill was encountered underlying the gravelly sand fill in Borehole NW20-01 and underlying the sandy silt fill in Borehole NW20-03. Occasional crushed gravel pieces, metal fragments, brick pieces and hydrocarbon odour were noted within the silty clay fill. The silty clay fill ranged from 1.2 m to 3.2 m in thickness and the base of this fill was encountered at depths between 3.0 m and 4.6 m (Elevations 73.9 and 72.3).

The SPT-N values recorded in the silty clay fill ranged from 9 to 36 blows per 0.3 m of penetration indicating a stiff to hard consistency. The moisture contents of the clayey fill ranged from 10 to 42 percent, with the higher values associated with high organic contents.

Organic contents measured from selected fill samples between 1 m and 4 m depths ranged from 11 percent to a high of 63 percent.

The results of grain size distribution analyses carried out on selected samples of the granular and cohesive fill are presented on the Record of Borehole Sheets and Figures B1 and B2 in Appendix B. The results of the grain size distribution analyses are summarized below:

<table>
<tr><th rowspan="2">Soil Particle</th><th colspan="3">Percentage (%)</th></tr>
<tr><th colspan="2">Gravelly Sand to Sand and Gravel Fill</th><th>Silty Clay Fill</th></tr>
<tr><td>Gravel</td><td colspan="2">27 to 52</td><td>9</td></tr>
<tr><td>Sand</td><td colspan="2">44 to 45</td><td>44</td></tr>
<tr><td>Silt</td><td>20</td><td rowspan="2">4</td><td>35</td></tr>
<tr><td>Clay</td><td>8</td><td>12</td></tr>
</table>

5.4 Silty Clay

A native deposit of brown to grey silty clay containing trace sand was encountered underlying the granular fill in Boreholes NW20-01 and NW20-02, and underlying the silty clay fill in Borehole NW 20-03. The thickness of the silty clay ranged from 1.3 m to 4.5 m and appeared to be decreasing towards the north. The base of the deposit extended to depths from 5.9 m to 10.4 m below ground surface (Elevations 71.0 to 66.5).

The SPT-N values recorded in the silty clay ranged from 5 to 24 blows per 0.3 m of penetration. In general, higher SPT-N values were measured near the top of the deposit, indicative of a crust. The lower 'N' values indicated the presence of firm zones within this deposit. Undrained shear strengths measured by field vane shear tests in the clay ranged from 55 to 105 kPa indicating a stiff to very stiff consistency. Two thin-walled Shelby tube samples were recovered near the bottom of the silty clay in Boreholes NW20-01 and NW20-02.

The moisture contents of the silty clay ranged from 30 percent to 70 percent.

The results of grain size distribution analyses carried out on selected samples of the silty clay are presented on the Record of Borehole Sheets and Figure B3 in Appendix B. The results of the grain size distribution analyses are summarized below:

Soil Particle	Percentage (%)
Gravel	0
Sand	2 to 10
Silt	23 to 33
Clay	65 to 74

The results of Atterberg Limits tests carried out on selected samples of the silty clay are presented on the Record of Borehole Sheets and Figure B5 in Appendix B and are summarized below:

Soil Property	Percentage (%)
Liquid Limit	56 to 68
Plastic Limit	21 to 36
Plasticity Index	31 to 35

The results of the Atterberg Limit tests indicate that the silty clay has high plasticity (group symbol of CH).

5.5 Silty Clay Till

A deposit of brown to grey silty clay till was encountered underlying the silty clay in Borehole NW20-03 at a depth of 5.9 m. The till was sandy and contained trace gravel, with occasional inferred cobbles and boulders. Borehole NW20-03 was terminated within this deposit upon auger refusal at a depth of 8.5 m (Elevation 68.4).

The SPT-N values recorded in the till ranged from 30 to 33 blows per 0.3 m of penetration indicating a hard consistency. The moisture contents of the silty clay till ranged from 10 to 12 percent.

The results of a grain size distribution analysis carried out on a sample of the silty clay till are presented on the Record of Borehole Sheets and Figure B4 in Appendix B. The results of the grain size distribution analysis are summarized below:

Soil Particle	Percentage (%)
Gravel	4
Sand	16
Silt	58
Clay	22

The results of an Atterberg Limits test carried out on a sample of the silty clay till are presented on the Record of Borehole Sheets and Figure B6 in Appendix B, and are summarized below:

Soil Property	Percentage (%)
Liquid Limit	28
Plastic Limit	14
Plasticity Index	14

The results of the Atterberg Limit tests indicate that the silty clay till has low plasticity (CL).

5.6 Limestone Bedrock

The soils described above were found to be underlain by bedrock in Borehole NW20-01 consisting of limestone with shale interbeds of the Simcoe Group. Limestone bedrock was proved by coring in Borehole NW20-01 underlying the native silty clay deposit at a depth of 10.4 m (Elevation 66.5). Both Boreholes NW20-02 and 20-03 were terminated upon auger refusal that can be interpreted as probable bedrock at depths of 9.1 m to 8.5 m (Elevations 67.0 to 68.4).

This limestone is typically fossiliferous, argillaceous and laminated, and varies from medium to thickly bedded. The recovered rock cores are grey to dark grey in colour and described as horizontally bedded and moderately weathered. The rock cores contain frequent shale interbeds typically ranging between 10 mm and 60 mm in thickness. Rock core photos are presented in Appendix C.

The Total Core Recovery (TCR) and Solid Core Recovery (SCR) in the core runs was 100 percent, while the Rock Quality Designation (RQD) values ranged from 82 percent to 97 percent indicating that the rock is of good to excellent quality. The Fracture Index (FI) of the rock, expressed as number of fractures per 0.3 m of core, ranged from 0 to 3.

Unconfined compressive strengths (UCS) interpreted from point load tests conducted on selected rock cores typically varied from 69 MPa to 138 MPa. The UCS of the rock, determined from two laboratory unconfined compression tests, ranged from 105 MPa to 123 MPa. The combined results indicate a generally strong to very strong rock. A point load test conducted on a rock core sample in Borehole NW20-01 correlated to a strength of 33 MPa, indicating a medium strong zone. Results of the point load tests are included in Appendix B.

5.7 Groundwater Conditions

The water levels measured in the open boreholes and standpipe piezometers installed during the investigation are summarized in Table 5.2 below.

Table 5.2 - Piezometer Details and Groundwater Levels

Borehole	Measurement Date	Water Level (m)		Notes
		Depth	Elevation	
NW 20-01	November 18, 2020	2.4	74.5	Piezometer
NW 20-02	November 17, 2020 November 18, 2020	1.8 1.6	74.3 74.5	Piezometer
NW 20-03	November 17, 2020	2.3	74.6	Open Borehole

The above groundwater levels represent short term readings and seasonal fluctuations of the groundwater level are to be expected. The technical memorandum (Reference 1) quoted that the water level at the nearby Bay of Quinte was at Elevation 74.9 at the time of the original 1977 site investigation for the existing bridge. It is anticipated that the groundwater level in the north approach area would be governed by the bay water level, and that the groundwater level may be at higher elevations after spring snowmelt or periods of heavy rainfall.

6 MISCELLANEOUS

The boreholes were drilled using a track-mounted CME 55 drill rig. George Downing Estate Drilling Ltd. of Hawkesbury, Ontario supplied and operated the drilling and sampling equipment for the field program.

Full time supervision of the field activities, including obtaining utility clearances, was carried out by Thurber Engineering. Overall supervision of the field program was performed by Ms. Rocio Reyna, P.Eng. of Thurber.

Interpretation of the field data and preparation of the report was performed by Mr. Geoff Lay, P.Eng. and Mr. Keli Shi, 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 presents preliminary geotechnical recommendations in support of the design of the proposed northbound lane (NBL) north approach embankment. Preliminary discussions on foundation design of the twinning bridge are also included.

Based on the plan and cross-section drawings, up to 5.3 m of fill will be placed for the new NBL approach embankment. The new fill will be placed along the east side of the existing north approach embankment from approximately 85 m north to 240 m north of the existing north abutment, decreasing in height towards the north. It is understood that this north approach embankment will be constructed within the area of an old landfill.

The recommendations provided in this memo are based on the embankment cross-sections provided by WSP dated November 18, 2020, and on the factual data obtained during the course of this investigation. The recommendations are for preliminary design purposes only.

8 NORTH APPROACH FILL STABILITY ANALYSIS

Slope stability analyses were performed to assess the global stability of the proposed high fills at the critical embankment section at Station 21+250 along the proposed north approach for the twinning bridge. The stability analyses were carried out using the commercially available software Slope/W, developed by GEO-SLOPE International Ltd., employing the

Morgenstern-Price method. Target minimum factors of safety of 1.3 and 1.5 are adopted for short-term and long-term conditions, respectively.

The results of the slope stability analyses are presented in Appendix F and summarized below.

Station	Fill Height (m)	Slope Inclination	Fill Material	Loading Condition	Factor of Safety	Figure No.
21+250	5.3	2H : 1V	Compacted Granular A or B Type II	Short-Term	1.4	1
		2H : 1V	Compacted Granular A or B Type II	Long-term	1.5	2
		2H : 1V	Compacted Earth Fill	Short-Term	1.4	3
		2H : 1V	Compacted Earth Fill	Long-term	1.4	4
		2.25H : 1V	Compacted Earth Fill	Short-Term	1.5	5
		2.25H :1 V	Compacted Earth Fill	Long-term	1.6	6

Based on the results of the analysis, the following preliminary recommendations are provided for the new fill:

- Slope inclinations not steeper than 2 horizontal to 1 vertical (2H : 1V) may be used for the embankments up to 5.3 m in height and constructed with compacted OPSS Granular A or B Type II.
- Slope inclinations not steeper than 2.25 horizontal to 1 vertical (2.25H : 1V) may be used for the embankments up to 5.3 m in height and constructed with compacted inorganic earth fill that is free of high plastic clay.

9 NORTH APPROACH FILL SETTLEMENT ASSESSMENT

Settlement analyses for the new north approach fill embankment were carried out to estimate the magnitude of settlement of the foundation soils anticipated during construction and post-construction (long-term) under the weight of the new fill materials. The settlement analyses were performed using the commercially available software Settle3D (Version 3) developed by Rocscience Inc.

The currently applicable MTO embankment settlement criteria for design (July 2010) stipulates that acceptable post-construction settlements are as follows:

- 25 mm for the first 20 m behind the new abutments,
- 50 mm between 20 m and 50 m behind the new abutments,
- 75 mm between 50 m and 75 m behind the new abutments,
- 100 mm for greater than 75 m behind the abutments.

The results of the settlement analysis are presented in Appendix G. The estimated settlements consist of two components as follows:

- Settlement of the existing fill containing landfill debris which should occur relatively quickly as the new approach fill is placed.
- Settlement of the underlying clay foundation subgrade which will be time dependent.

The analyses were conducted using parameters for the clay foundation based on correlation with index properties and soil shear strengths established during current and previous site investigations. In the absence of consolidation tests, settlements have been estimated for two different sets of clay compression/recompression ratios. In the first analysis, a compression ratio and recompression ratio of 0.25 and 0.025 were used, while in the second analysis, a compression ratio and recompression ratio of 0.35 and 0.035 were used.

Based on these parameters, the estimated foundation settlement beneath the maximum embankment height at the proposed north approach is expected to range between 75 mm and 115 mm after a 6-month waiting period following completion of fill placement, and between 90 mm and 135 mm after 20 years. In addition, the embankment settlement due to fill compression is estimated at 0.5 percent of the fill height. It is anticipated that approximately 50 percent of the total fill compression (or 0.25 percent of the fill height) will occur during construction, and that the remaining 50 percent, or approximately 10 to 15 mm, will occur after construction.

It is anticipated that construction of the new approach embankment could result in 10 mm to 15 mm settlement of the adjacent existing approach fill. As such, the pavement of the latter may require asphalt resurfacing.

To meet MTO's embankment settlement criteria, it is recommended that a minimum waiting period of 6 months be allowed after completion of fill placement for foundation settlement to take place prior to abutment foundation and approach slab construction, and paving.

10 EMBANKMENT CONSTRUCTION

All embankment fill must be constructed with adequate quality control in accordance with OPSS.PROV 206 and OPSS.PROV 501 requirements.

Where new fill is placed against an existing embankment slope or on a sloping ground surface steeper than 3H : 1V, the existing slope should be benched in accordance with OPSD 208.010.

Prior to fill placement, the subgrade must be adequately prepared to receive the new fill. All vegetation, topsoil, organics, soft/loosened or wet soils should be sub-excavated. Based on available information, the site was previously used as a landfill. Various types of waste materials (decayed wood fibres, circular rubber pieces, occasional boulder fragments, charcoal fragments, brick pieces and metal fragments) and debris were encountered within the fill during the investigation. Where exposed at the subgrade level, it is recommended that these materials be sub-excavated and removed from the site prior to placing new fill. Subgrade preparation at this site should be addressed during detail design.

It is also recommended that all permanent and temporary slope surfaces be vegetated and seeded in accordance with current MTO practice with reference to OPSS.PROV 804. Surface runoff and precipitation must be prevented from flowing perpendicularly down any slope surface. Erosion protection measures will have to be taken as necessary to maintain surficial slope stability.

11 FOUNDATION ALTERNATIVES

The Thurber technical memorandum (Reference 1) presents descriptions of the foundation types used for the existing bridge and are summarized as follows:

- Both abutments are supported on vertical and battered steel HP 310 x 110 piles reportedly driven to bedrock.
- All the piers are supported on composite piles, each of which consists of a steel tube pile enclosing three HP 310 x 110 piles socketted at least 0.5 m into bedrock. The tube piles had been reportedly driven to bedrock.

A draft Structural Design Report (SDR) titled “Norris Whitney Bridge, Highway 62 – Bridge Twinning”, dated April 7, 2020 was prepared by WSP. The SDR indicates that the new twinning bridge will be located to the east of the existing bridge with an 8.5 m clearance between the two structures. It also indicates that it is expected that the new pier structures and foundations will be generally similar to those of the existing bridge, i.e. H-piles driven to bedrock at the abutments and composite tube piles/H-piles with the H-piles socketted into bedrock at the piers. The draft SDR also indicates that each new bridge pier foundation is designed to be staggered with the adjacent existing pier in order to avoid contact between new and old piles. Moreover, an additional north approach span has been added such that the new north approach and north abutment would not be located directly on the known landfill area by the north shoreline.

Based on preliminary subsurface information obtained at the north approach area during the current preliminary investigation, and previously reported subsurface information available for the existing bridge, foundation alternatives that can be considered for the new twinning bridge are discussed in the following.

- We concur that steel H-piles driven to bedrock is feasible for use as foundation support at the north abutment, provided that the fill preloading/waiting period is implemented prior to pile installation. Alternatively, steel H-piles or pipe piles socketted into bedrock may also be considered. For the south abutment, similar alternatives may also be considered.
- At the piers, foundation construction will have to be carried out within dewatered cofferdam enclosures. The composite tube pile/H-pile combination is feasible. It is considered that the tube pile would increase the lateral resistance of the pile group and also serve the purpose of a casing for installation of the socketted H-piles. However, consideration may also be given to using concrete filled steel pipe piles socketted into bedrock.

- In addition, caissons (drilled shafts) socketted into bedrock may also be considered as an alternative to the above. At the piers, caissons with permanent steel casings may be used, where the casings would serve the purposes of temporary liners and providing partial water cutoff during construction, and would increase the lateral resistance of the foundation unit. It may also be possible to design a continuous caisson and column system without using a cap.

12 INVESTIGATION FOR DETAIL DESIGN

There is no GEOCRES information available for the new bridge and approach alignment. The subsurface conditions depicted by the three boreholes at the north approach and abutment area from the preliminary investigation is insufficient and incomplete to be used for detail design of the new works. It will be necessary to carry out additional site investigation and field testing to support the preparation of foundation design recommendations for detail design of the new twinning bridge and its approach fills. In particular, boreholes must be advanced for the new piers in the water and at the south approach and abutment area.

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

- 2 BHs at each foundation element advancing to a minimum of 3 m below refusal.
- Where bedrock is encountered, all of the boreholes at each foundation element shall be cored for a minimum depth of 3 m.
- 1 BH at each bridge approach embankment within 20 m of the abutment, advancing to 3 m into a competent stratum or 10 m below the base of the fill. Where bedrock is encountered, no coring is required.

The three boreholes advanced for this preliminary investigation can be incorporated into the detail investigation program. In order to satisfy the minimum MTO requirements above, the borehole configuration for detail design should be as follows:

- 2 BHs at each abutment area within existing bridge for a total of 4 BHs.

- 2 BHs at each pier location for a total of 22 BHs assuming 11 piers; however, the actual number of required boreholes may be reduced upon discussion with MTO, given the close proximity of previous boreholes for the existing bridge and the relatively high cost of advancing boreholes over open water.

- 1 BH at each approach area for a total of 2 BHs.

13 CLOSURE

Engineering analysis and preparation of the report were carried out by Mr. Geoff Lay, P.Eng. and Dr. Sydney Pang, 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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Appendix A

Records of Boreholes

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

NOTE: Hierarchy of Soil Strength Prediction

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

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE

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

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

▼ Water Level

C_{pen} Shear Strength Determination by Pocket Penetrometer

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

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

UNIFIED SOILS CLASSIFICATION

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

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

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

SYMBOLS

CLAYSTONE

SILTSTONE

SANDSTONE

COAL

Bedrock (general)

DISCONTINUITY SPACING

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

TERMS

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

STRENGTH CLASSIFICATION

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

THURBER

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RECORD OF BOREHOLE No NW 20-01

METRIC

W.P. 4194-15-00	LOCATION Norris Whitney Bridge, MTM NAD83-9 N 4 890 598.6 E 233 788.1		ORIGINATED BY GA
DIST Eastern HWY 62	BOREHOLE TYPE Hollow Stem Augers/NQ Coring		COMPILED BY AN
DATUM Geodetic	DATE 2020.11.16 - 2020.11.17	LATITUDE 44.151935 LONGITUDE -77.387723	CHECKED BY RPR

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
76.9	GROUND SURFACE					
0.0 / 0.1	**ASPHALT:** (75mm)					
	SAND and **GRAVEL**, crushed gravel pieces, Very Dense, Brown to Grey, Moist (FILL)	1	SS	50/0.125		
76.3 / 0.6	Gravelly **SAND**, some silt, trace clay, Compact to Dense, Brown to Grey, Moist (FILL)	2	SS	22		27 45 20 8
		3	SS	36		
75.1 / 1.8	Silty **CLAY**, with sand, occasional crushed gravel pieces, hydrocarbon odour, Very Stiff, Dark Brown to Black, Moist (FILL)	4	SS	15		
73.9 / 3.0	**SAND** and **GRAVEL**, trace silt, trace clay, Compact, Brown to Grey, Moist (FILL)	5	SS	14		
	Wet	6	SS	30		52 44 4 (SI+CL)
70.8 / 6.1	Silty **CLAY**, trace to some sand, Stiff, Grey, Wet	7	SS	13		
	Field vane: sensitivity 1.8					
	Firm	8	SS	5	wP–wL	0 10 23 67
	Field vane: sensitivity 6.5					
		1	TW			

Elevation scale: 76, 75, 74, 73, 72, 71, 70, 69, 68, 67

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

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

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

ONTMT4S2 MTO-11566.GPJ 2017TEMPLATE(MTO).GDT 6/4/21

RECORD OF BOREHOLE No NW 20-01

METRIC

W.P. 4194-15-00 LOCATION Norris Whitney Bridge, MTM NAD83-9 N 4 890 598.6 E 233 788.1 ORIGINATED BY GA

DIST Eastern HWY 62 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY AN

DATUM Geodetic DATE 2020.11.16 - 2020.11.17 LATITUDE 44.151935 LONGITUDE -77.387723 CHECKED BY RPR

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page							
66.5 10.4	Silty **CLAY**, trace sand Firm Grey Wet Coring started at 10.4m						FI	
	LIMESTONE, moderately weathered, grey to dark grey, with shale interbeds, laminated, horizontally bedded: (Simcoe Group) Horizontal fractures at 10.54m, 10.68m, 10.87m, 11.65m and 11.74m	1	RUN			66	2 1 2 1 3	RUN #1 TCR=100% SCR=100% RQD=82% UCS=103MPa (Average-PLT's) UCS test value=122.5MPa at 10.7m
63.5 13.4	Shale interbeds: 10.88m - 10.92m (40mm) 12.69m - 12.72m (30mm) 12.76m - 12.79m (30mm) 12.89m - 12.91m (20mm) 13.00m - 13.06m (60mm) 13.21m - 13.23m (20mm) 13.24m - 13.28m (40mm) 13.33m - 13.35m (20mm) 13.40m - 13.41m (10mm) Horizontal fractures at 17.10m, 12.40m, 12.72m and 13.40m	2	RUN			65 64	1 1 1 0 1	RUN #2 TCR=100% SCR=100% RQD=97% UCS=107MPa (Average-PLT's) UCS test value=105.6MPa at 12.2m UCS=33.0MPa at 13.3m (PLT)

END OF BOREHOLE AT 13.4m. MONITORING WELL WAS INSTALLED 1.0m NORTH OF THE BOREHOLE. AUGERED TO 5.2m.

Monitoring well installation consists of 50mm diameter Schedule 40 PVC pipe with a 3.05m slotted screen.

WATER LEVEL READINGS

DATE	DEPTH(m)	ELEV.(m)
2020.11.18	2.4	74.5

Chart column headings: 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, × ³: Numbers refer to Sensitivity   20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No NW 20-02

METRIC

W.P. 4194-15-00	LOCATION Norris Whitney Bridge, MTM NAD83-9 N 4 890 610.2 E 233 806.4		ORIGINATED BY GA
DIST Eastern HWY 62	BOREHOLE TYPE Hollow Stem Augers		COMPILED BY AN
DATUM Geodetic	DATE 2020.11.17 - 2020.11.17	LATITUDE 44.152042 LONGITUDE -77.387495	CHECKED BY RPR

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH (kPa)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
76.1	GROUND SURFACE					
0.0 / 0.1	**TOPSOIL:** (100mm)					
	SAND and **GRAVEL**, crushed gravel pieces, occasional roots and rootlets Dense to Very Dense Brown Moist (FILL)	1	SS	33		
	Decayed wood fibres, circular rubber pieces, occasional boulder fragments, occasional charcoal fragments, debris and brick pieces	2	SS	57		Organic Content 63.2%
		3	SS	28		
	Occasional metal fragments, odour Compact Wet					
	Some silt	4	SS	10		
	Rockfill layer, decayed wood fibres	5	SS	20		
71.5 / 4.6	Silty **CLAY**, trace sand Stiff Grey Moist to Wet	6	SS	14		
	Firm	7	SS	6		0 2 24 74
					3.4	
		1	TW			
					>100	Vane unable to turn
	Spoon bouncing at 9.1m					
67.0 / 9.1	END OF BOREHOLE AT 9.1m UPON AUGER REFUAL. BOREHOLE OPEN TO 9.1m AND WATER LEVEL AT 1.5m UPON COMPLETION OF DRILLING.	8	SS	50/ 0.0		

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No NW 20-02

METRIC

W.P.	4194-15-00	LOCATION	Norris Whitney Bridge, MTM NAD83-9 N 4 890 610.2 E 233 806.4	ORIGINATED BY	GA
DIST	Eastern HWY 62	BOREHOLE TYPE	Hollow Stem Augers	COMPILED BY	AN
DATUM	Geodetic	DATE	2020.11.17 - 2020.11.17 LATITUDE 44.152042 LONGITUDE -77.387495	CHECKED BY	RPR

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page										
	Well installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.05m slotted screen.										

WATER LEVEL READINGS

DATE	DEPTH(m)	ELEV.(m)
2020.11.17	1.8	74.3
2020.11.18	1.6	74.5

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

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

RECORD OF BOREHOLE No NW 20-03

METRIC

W.P. 4194-15-00 — LOCATION Norris Whitney Bridge, MTM NAD83-9 N 4 890 696.4 E 233 756.5 — ORIGINATED BY GA

DIST Eastern HWY 62 — BOREHOLE TYPE Hollow Stem Augers — COMPILED BY AN

DATUM Geodetic — DATE 2020.11.17 - 2020.11.17 — LATITUDE 44.152812 — LONGITUDE -77.388131 — CHECKED BY RPR

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
76.9	GROUND SURFACE							
0.0	**TOPSOIL:** (150mm)							
0.2	Sandy **SILT**, trace gravel, trace to some clay, occasional roots and rootlets; Compact; Brown; Moist; (FILL)	1	SS	13				
		2	SS	15		76		
75.5 / 1.4	Silty **CLAY**, with sand, trace gravel, occasional brick pieces; Very Stiff to Stiff; Dark Brown to Black; Moist; (FILL)	3	SS	20		75		9 44 35 12
					∇			
		4	SS	12				Organic Content 34.8%
	Decayed wood fibres, metal fragments					74		
	Layer of black clayey silt; Wet	5	SS	9				Organic Content 10.9%
						73		
72.3 / 4.6	Silty **CLAY**, trace sand; Very Stiff; Brown; Moist	6	SS	24		72		0 2 33 65
71.0 / 5.9	Silty **CLAY**, sandy, trace gravel, occasional cobbles and boulders; Hard; Brown to Grey; Wet; (TILL)					71		
		7	SS	33				
						70		
	Some sand	8	SS	30		69		4 16 58 22
68.4 / 8.5	END OF BOREHOLE AT 8.5m UPON AUGER REFUSAL. BOREHOLE OPEN TO 8.5m AND WATER LEVEL AT 2.3m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.3m, SAND TO 0.5m, THEN TOPSOIL TO SURFACE.							

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

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

15 20 5 10 (%) STRAIN AT FAILURE

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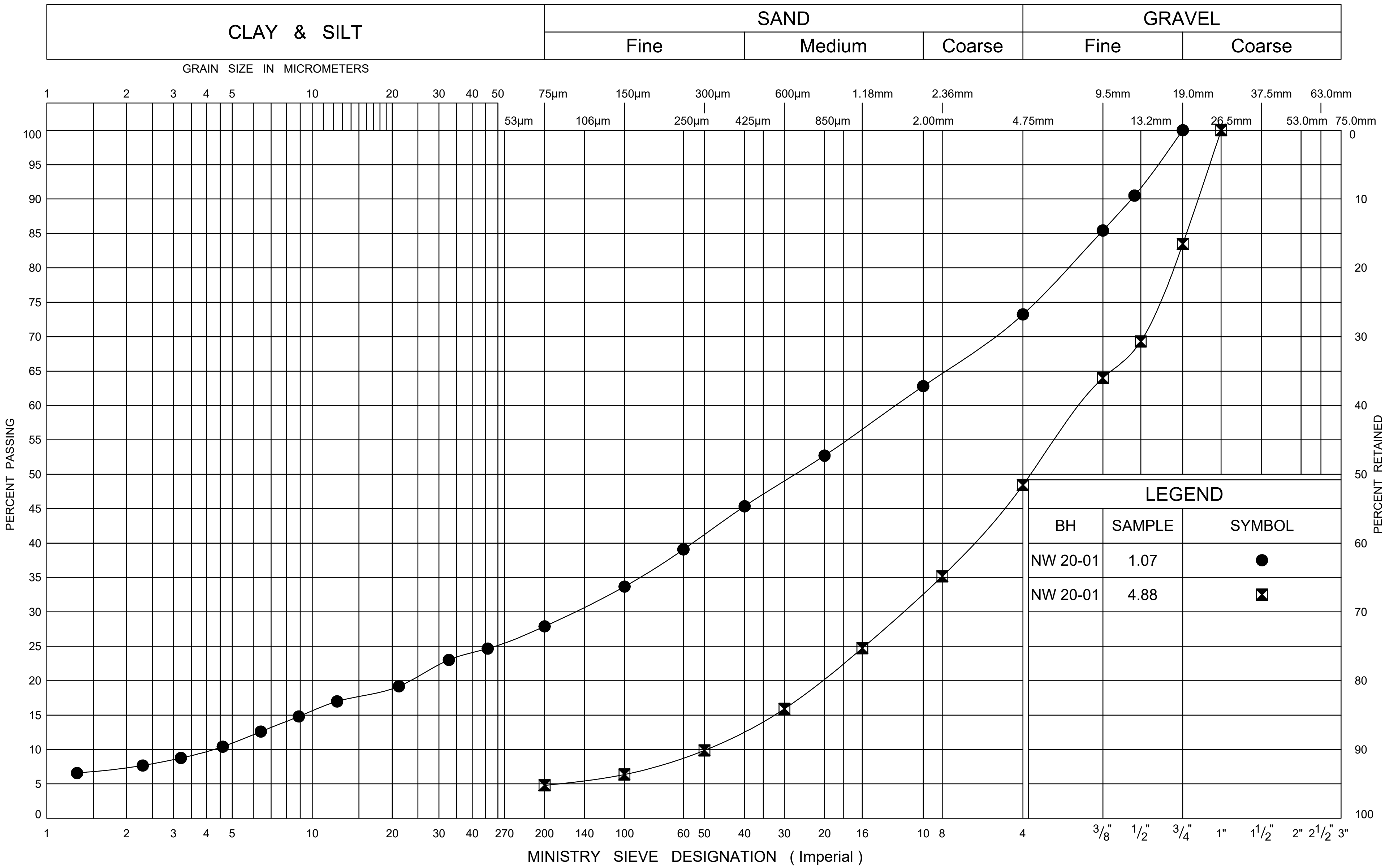
Appendix B

Geotechnical Laboratory Test Results

78 12 M



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GRAIN SIZE DISTRIBUTION

Gravelly SAND/SAND and GRAVEL FILL

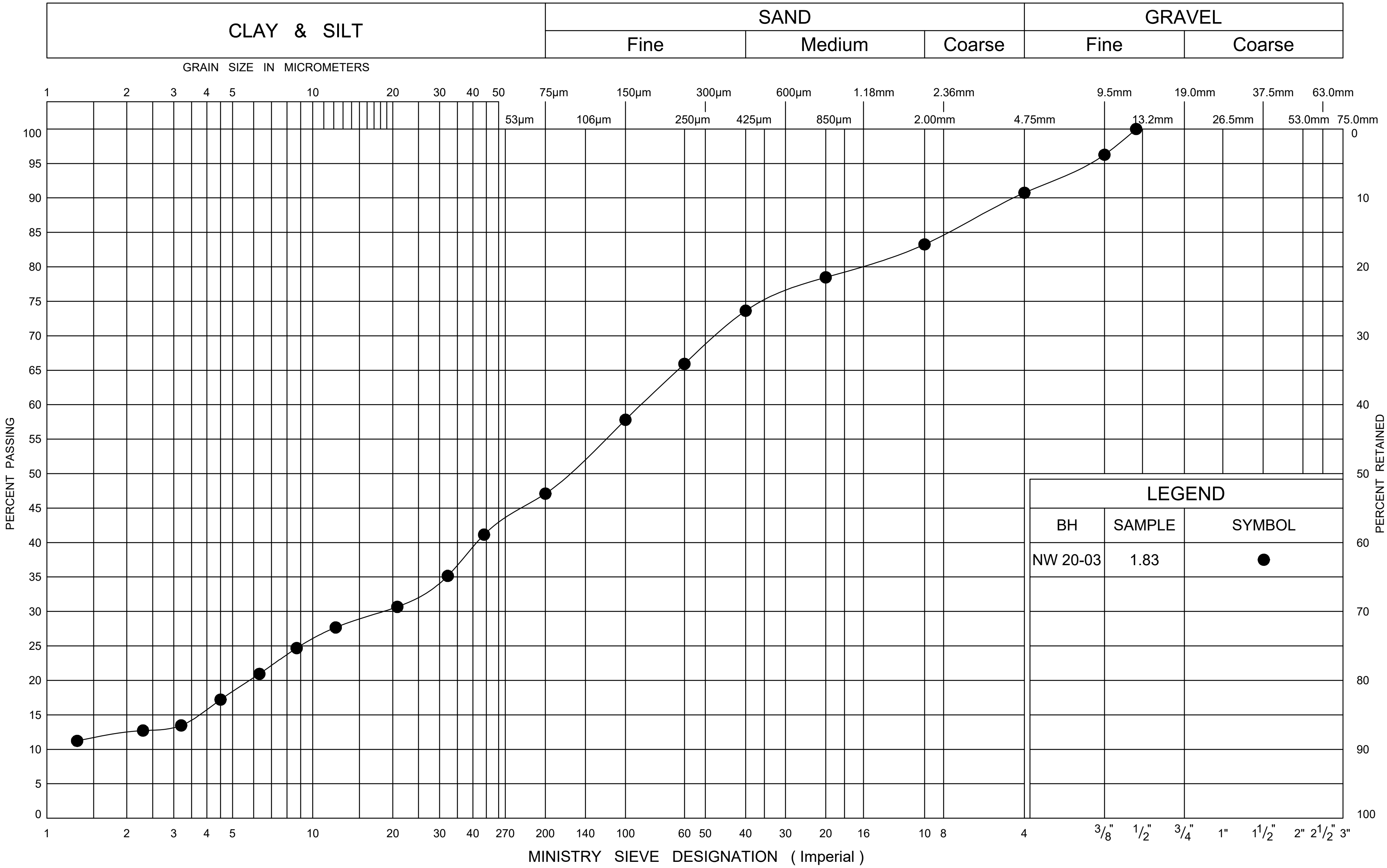
FIG No B1

W P 4194-15-00

Norris Whitney Bridge

ONTARIO MOT GRAIN SIZE 2 MTO-11566.GPJ ONTARIO MOT.GDT 1/13/21

78 12 M



LEGEND		
BH	SAMPLE	SYMBOL
NW 20-03	1.83	●

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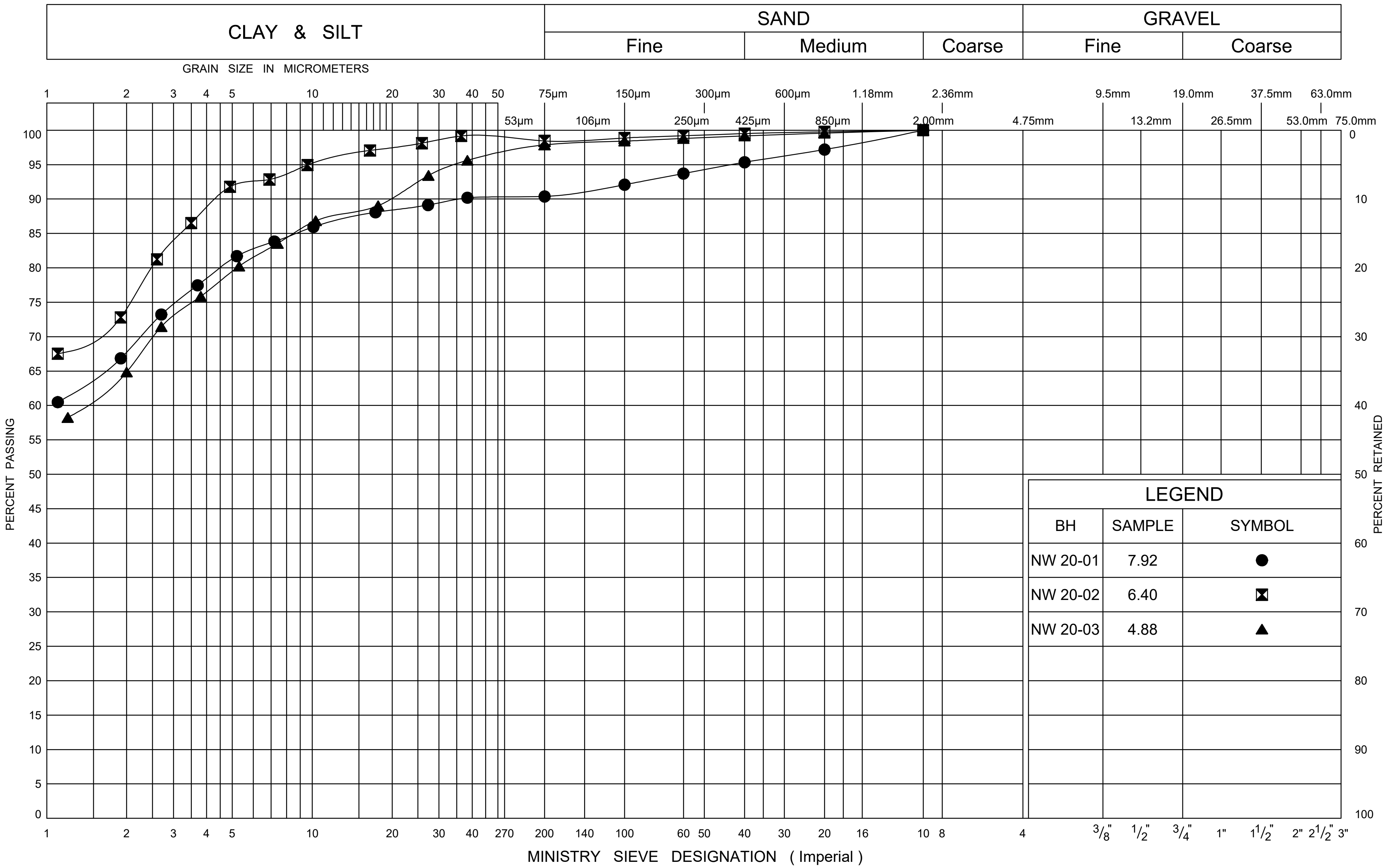
GRAIN SIZE DISTRIBUTION

Silty CLAY FILL

FIG No B2	
W P 4194-15-00	
Norris Whitney Bridge	

ONTARIO MOT GRAIN SIZE 2 MTO-11566.GPJ ONTARIO MOT.GDT 1/13/21

78 12 M



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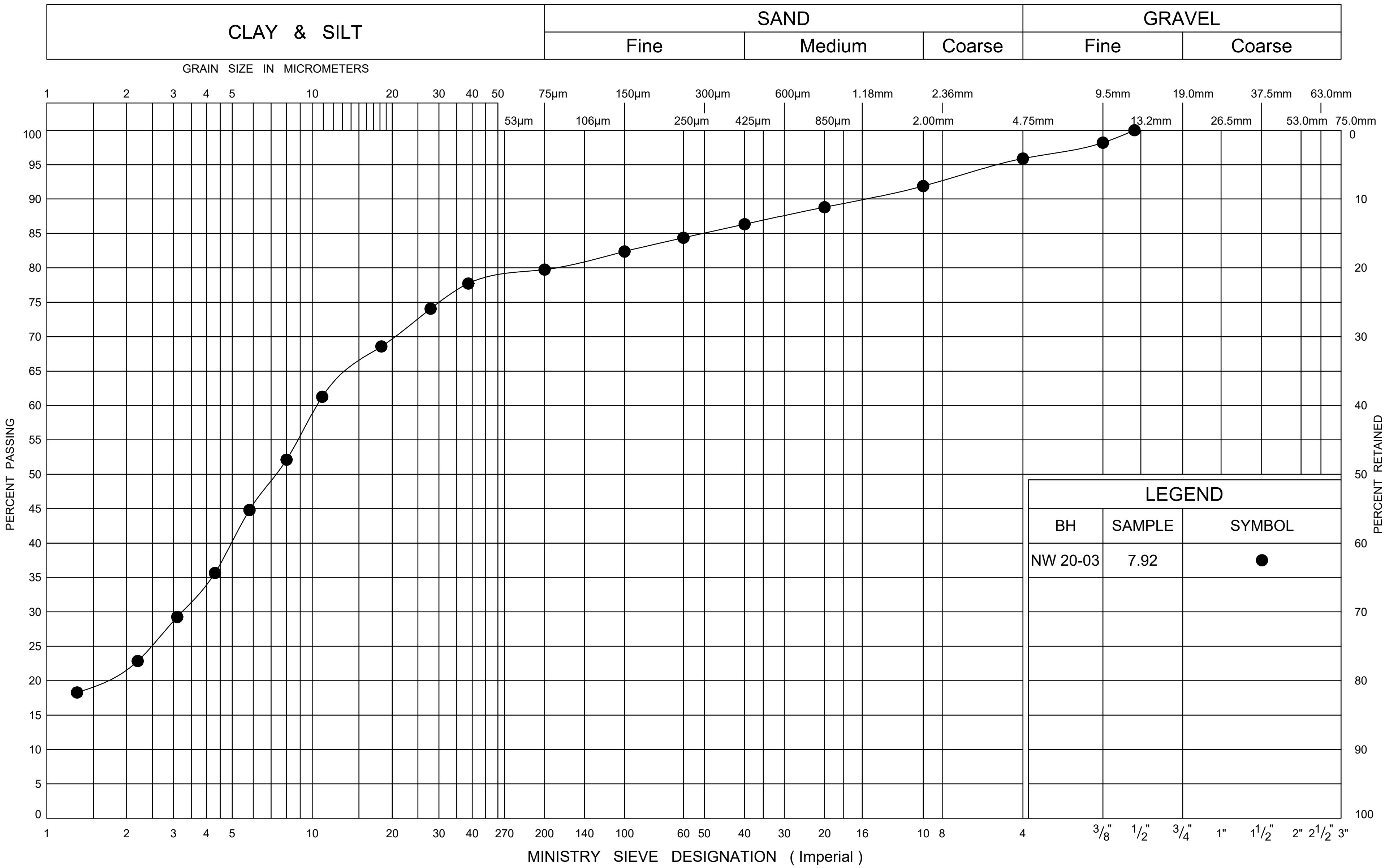
GRAIN SIZE DISTRIBUTION

Silty CLAY

FIG No B3

W P 4194-15-00

Norris Whitney Bridge



LEGEND

BH	SAMPLE	SYMBOL
NW 20-03	7.92	●

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GRAIN SIZE DISTRIBUTION

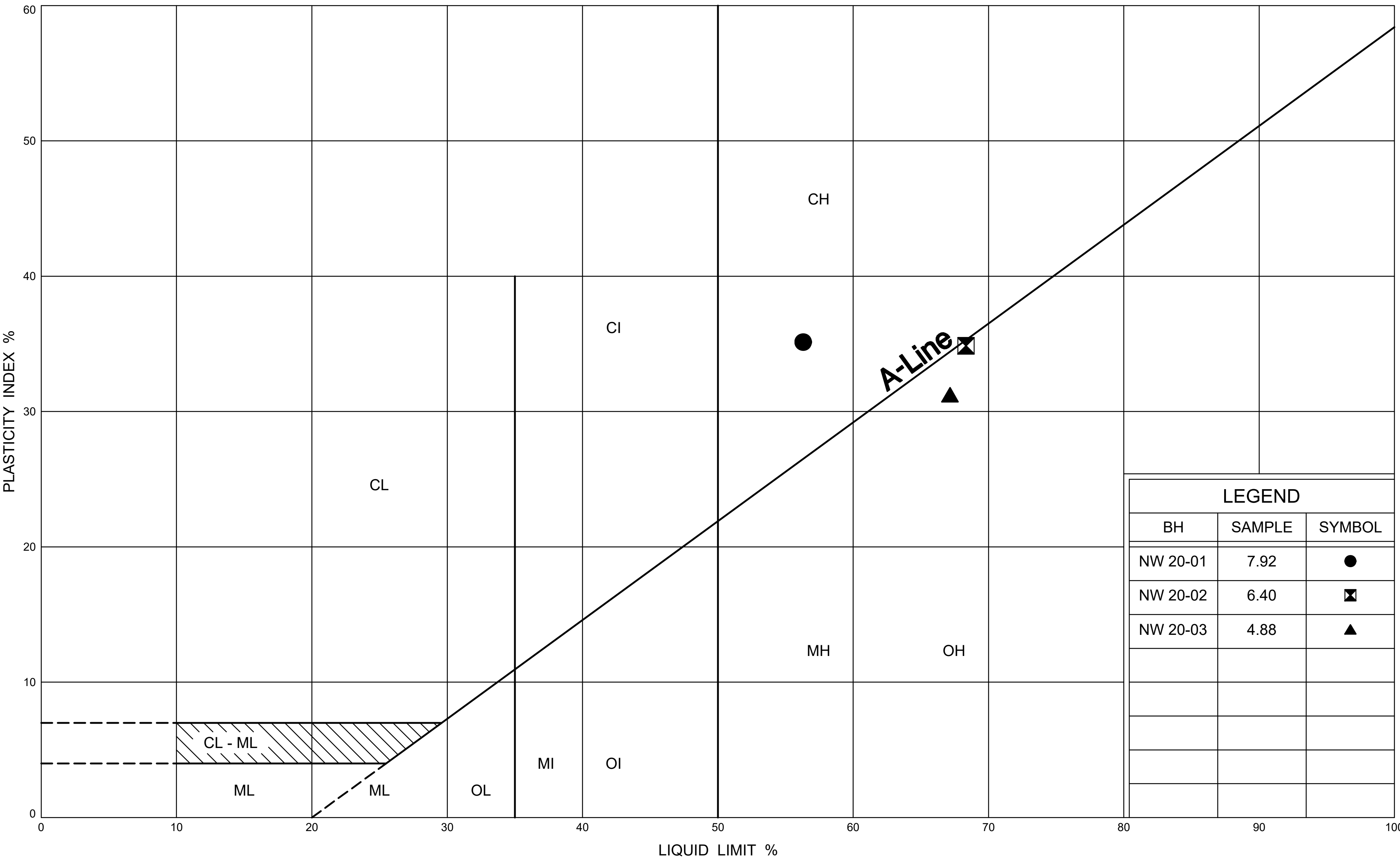
Silty CLAY TILL

FIG No B4

W P 4194-15-00

Norris Whitney Bridge

Oct 75, FF - S - 21



LEGEND

BH	SAMPLE	SYMBOL
NW 20-01	7.92	●
NW 20-02	6.40	⧗
NW 20-03	4.88	▲

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PLASTICITY CHART

Silty CLAY

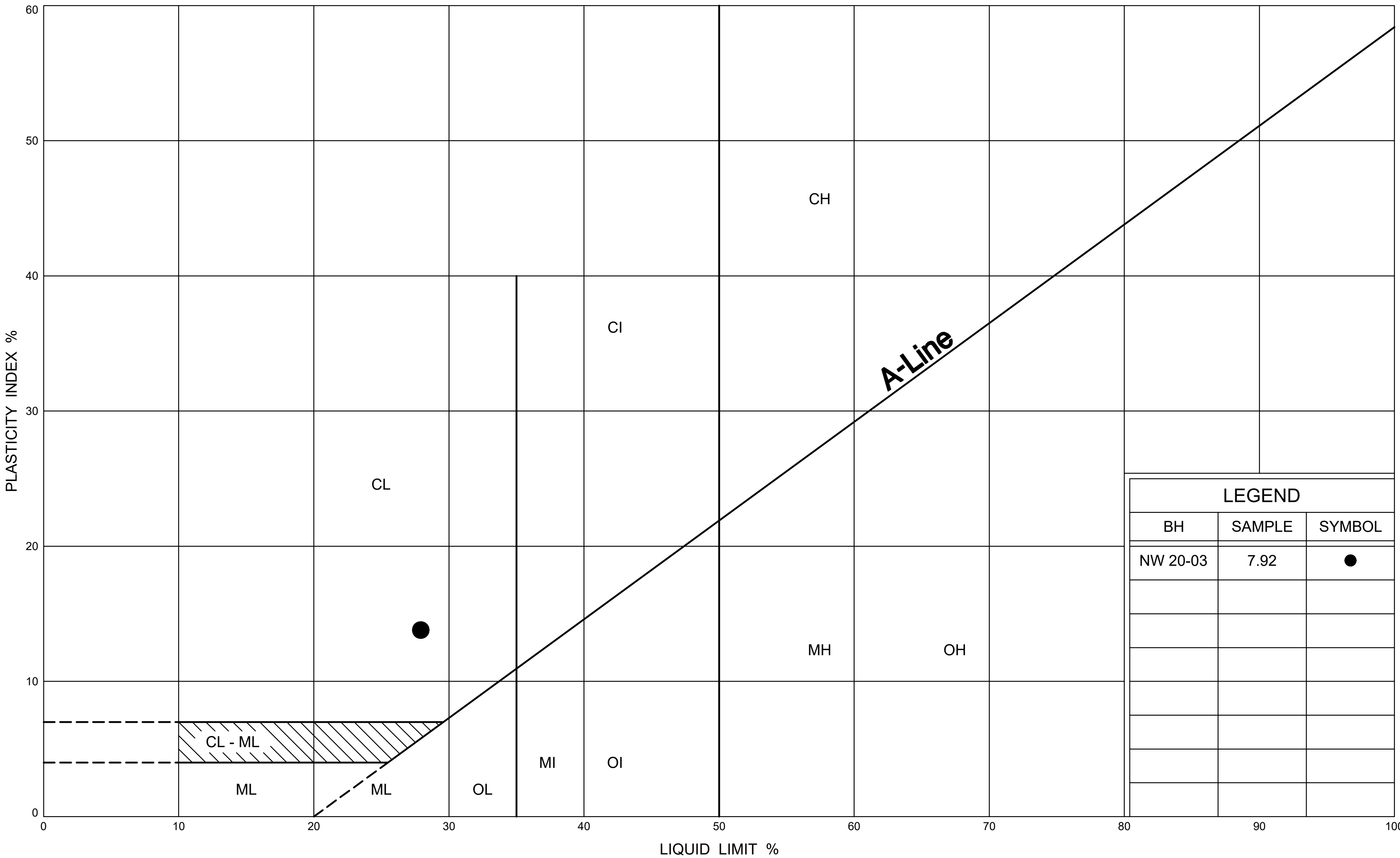
FIG No B5

W P 4194-15-00

Norris Whitney Bridge

ONTARIO MOT PLASTICITY CHART MTO-11566.GPJ ONTARIO MOT.GDT 1/13/21

Oct 75, FF - S - 21



LEGEND		
BH	SAMPLE	SYMBOL
NW 20-03	7.92	●

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PLASTICITY CHART
Silty CLAY TILL

FIG No B6

W P 4194-15-00

Norris Whitney Bridge

ONTARIO MOT PLASTICITY CHART MTO-11566.GPJ ONTARIO MOT.GDT 1/13/21

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	11566	**Date Drilled:**	16-Nov-20
Client:	WSP Canada Group Ltd.	**Date Tested:**	03-Dec-20
Project Name:	Hwy 401 Belleville	**Tester:**	GA
Core Size:	NQ **BH No :** NW20-01	**Reviewed by:**	RPR

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	$I_{s(50)}$ (MPa)	UCS (MPa)	Rock Type	Rock Strength (after Hoek & Brown, 1997)
1	1	10.5	A	21.5	46.8	65.9	5.7	137.6	Limestone	Very Strong
2	1	11.1	A	10.0	41.8	66.7	2.9	69.2	Limestone	Strong
3	1	11.7	D	10.3	47.0	154.5	4.3	103.3	Limestone	Very Strong
4	2	11.9	A	15.4	46.7	68.7	4.0	95.9	Limestone	Strong
5	2	12.5	A	17.1	46.7	60.0	4.9	118.0	Limestone	Very Strong
6	2	13.3	D	3.2	46.4	122.7	1.4	33.1	Limestone/Shale	Medium Strong
7										
8										
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34										

* It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing

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

* Correlation factor to obtain UCS values is 24.

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Appendix C

Rock Core Photographs

PHOTOGRAPHS OF ROCK CORES

BOREHOLE NW 20-01
RUNS 1 AND 2

TOP

Run 1

Run 2

BOTTOM

Run #	Depth (m)
1	10.3 – 11.9
2	11.9 – 13.4

PHOTOGRAPHS OF ROCK CORES

BOREHOLE NW 20-01
RUNS 1 AND 2

TOP

Run 1

Run 2

BOTTOM

Run #	Depth (m)
1	10.3 – 11.2
2	11.9 – 12.8

PHOTOGRAPHS OF ROCK CORES

BOREHOLE NW 20-01
RUNS 1 AND 2

TOP

Run 1

Run 2

BOTTOM

Run #	Depth (m)
1	11.2 – 11.9
2	12.8 – 13.4

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Appendix D

Selected Site Photographs

THURBER

Photo 1- Norris Whitney Bridge, northeast side
Location of Borehole NW20-01
Photo taken on November 18, 2020

Photo 2- Norris Whitney Bridge, northeast side
Location of Borehole NW20-01
Photo taken on November 18, 2020

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Photo 3- Norris Whitney Bridge, northeast side
Location of Borehole NW20-03
Photo taken on November 18, 2020

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Appendix E

Borehole Locations and Soil Strata Drawing

MINISTRY OF TRANSPORTATION, ONTARIO

METRIC

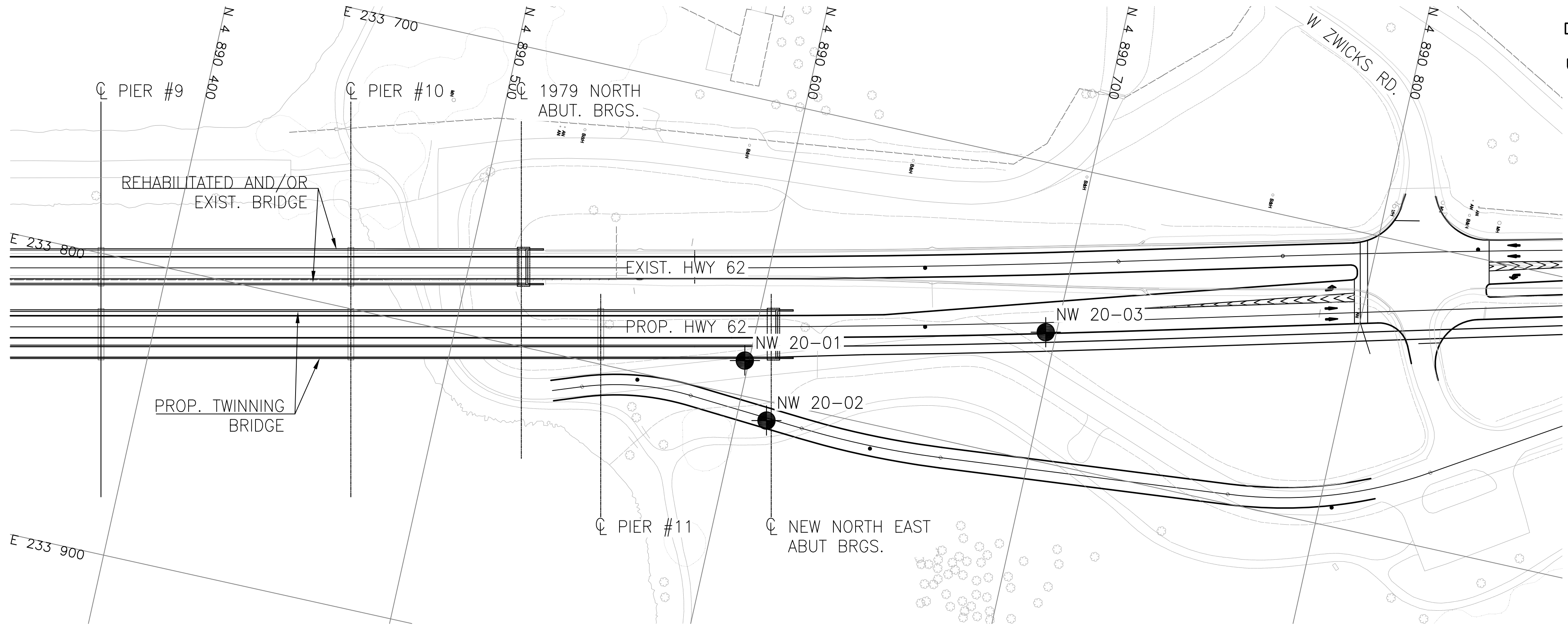
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

CONT No
WP No 4194–15–00

SHEET

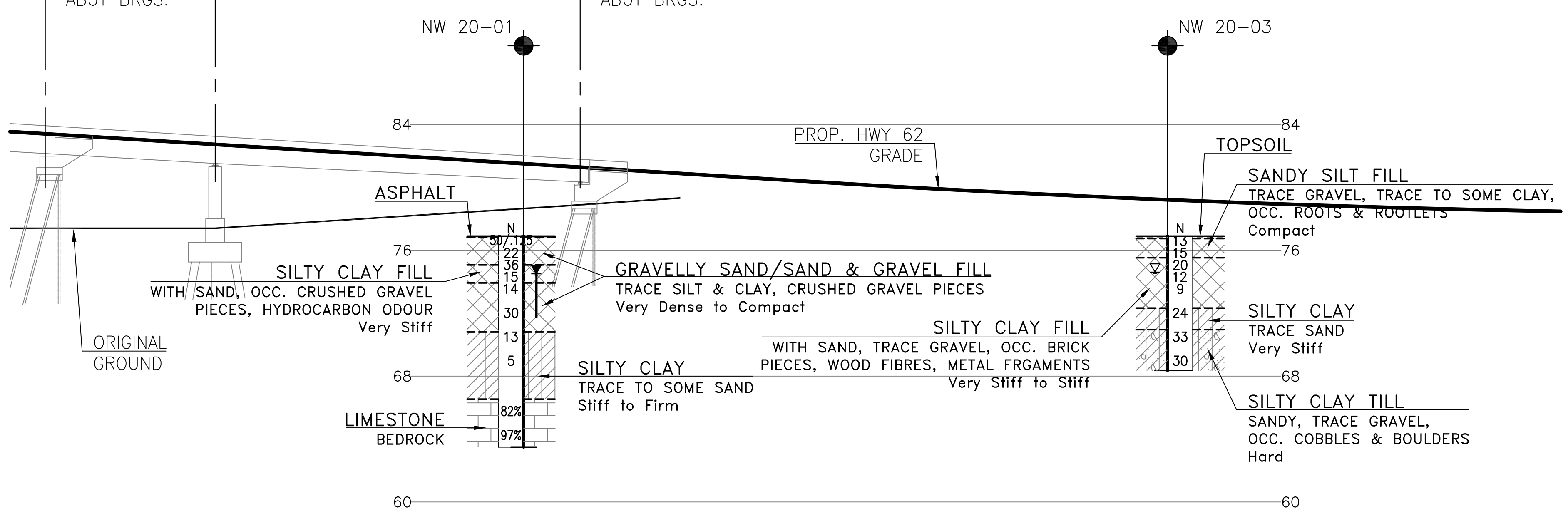
HIGHWAY 62
NORRIS WHITNEY BRIDGE TWINNING AT THE NORTH EMBANKMENT
BOREHOLE LOCATIONS AND SOIL STRATA

THURBER ENGINEERING LTD.



PLAN

40 0 40 80m
SCALE 1:2000



PROFILE ALONG PROP. ℄ HWY 62

20 0 20 40m H 1:1000
8 0 8 16m V 1:400

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
NW 20–01	76.9	4 890 598.6	233 788.1
NW 20–02	76.1	4 890 610.2	233 806.4
NW 20–03	76.9	4 890 696.4	233 756.5

-NOTES-

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

GEOCRES No. 31C-308

REVISIONS

DATE	BY	DESCRIPTION			
DESIGN RPR	CHK SKP	CODE	LOAD	DATE JUL 2021	
DRAWN AN	CHK RPR	SITE 28–28	STRUCT	DWG 1	

FILENAME: H:\Drafting\11000\11566\TED-11566-PLPR-NW.dwg
PLOTDATE: 7/8/2021 10:19 AM

MINISTRY OF TRANSPORTATION, ONTARIO

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

CONT No

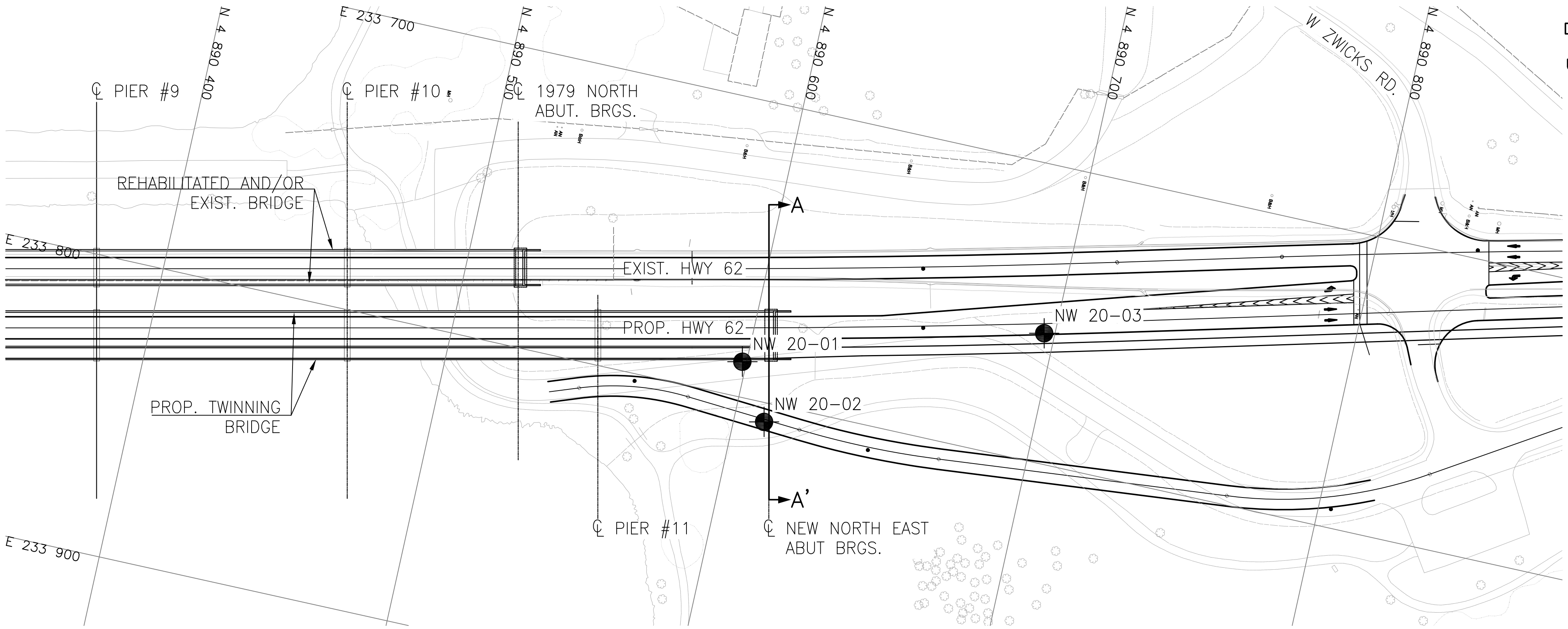
WP No 4194–15–00

HIGHWAY 62
NORRIS WHITNEY BRIDGE TWINNING AT THE NORTH EMBANKMENT
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

wsp

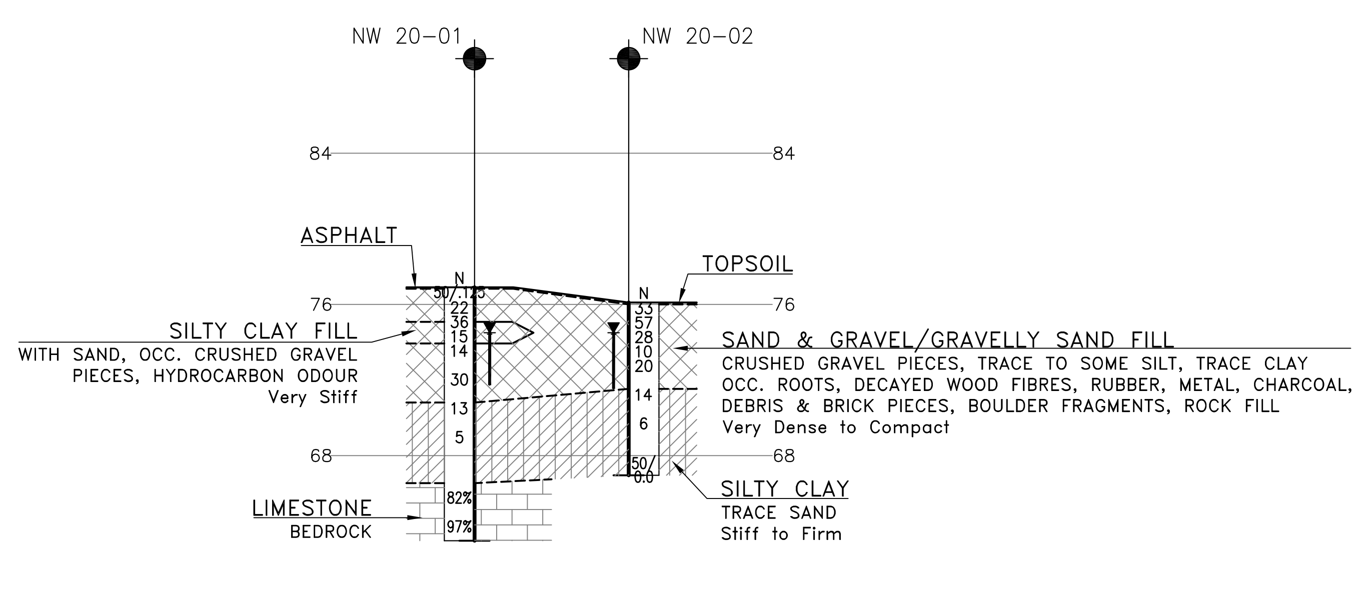
THURBER ENGINEERING LTD.



PLAN

40 0 40 80m

SCALE 1:2000



SECTION A–A'

20 0 20 40m H 1:1000

8 0 8 16m V 1:400

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
NW 20–01	76.9	4 890 598.6	233 788.1
NW 20–02	76.1	4 890 610.2	233 806.4
NW 20–03	76.9	4 890 696.4	233 756.5

-NOTES-

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

GEOCRES No. 31C-308

REVISIONS	DATE	BY	DESCRIPTION

DESIGN RPR	CHK SKP	CODE	LOAD	DATE JUL 2021	
DRAWN AN	CHK RPR	SITE 28–28	STRUCT	DWG 2	

FILENAME: H:\Drafting\11000\11566\TED-11566-PLPR-NW.dwg
PLOTDATE: 7/8/2021 10:19 AM

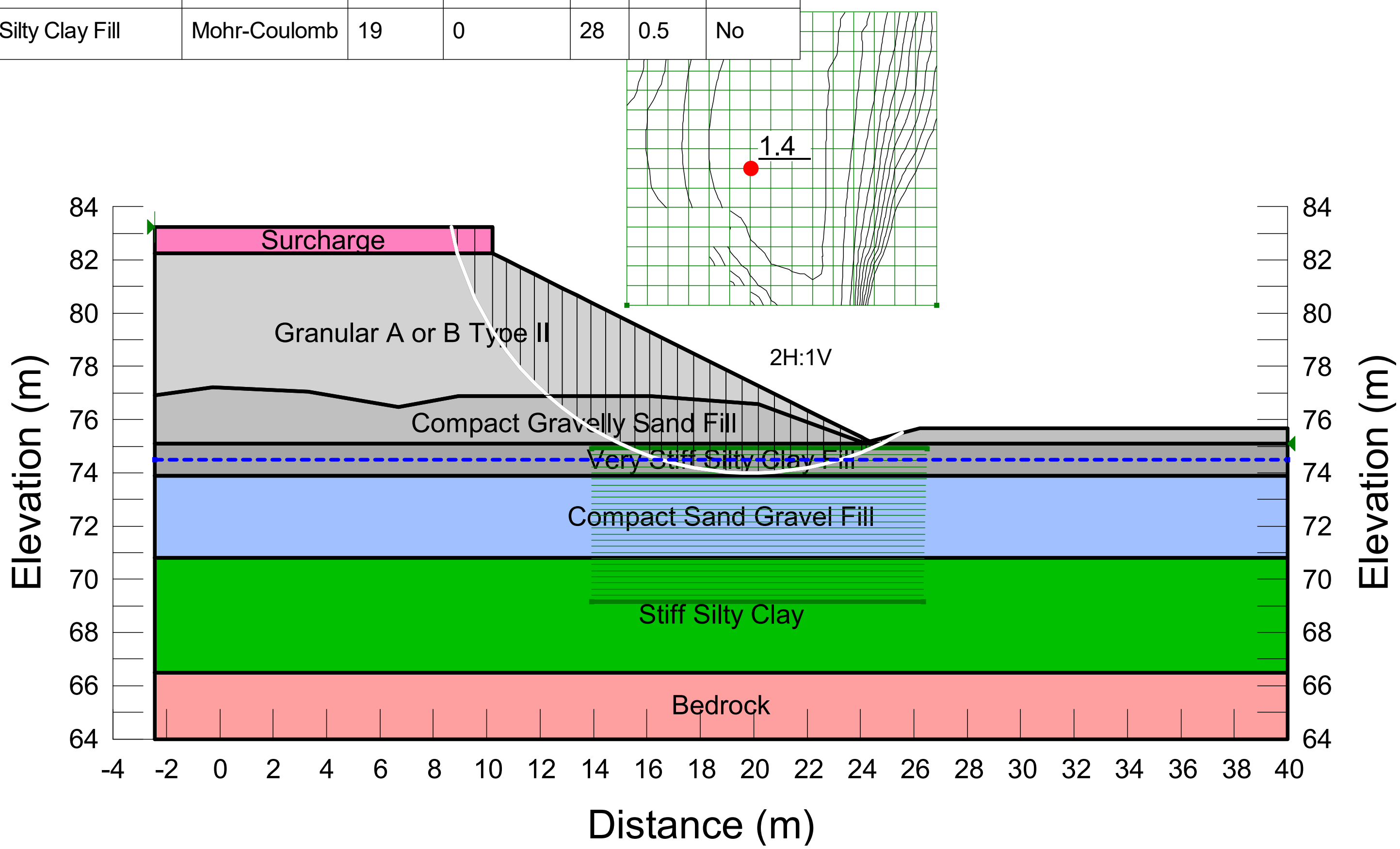
Appendix F

Selected Stability Analyses Results

NORRIS WHITNEY BRIDGE
NORTH APPROACH (STA 21+250)
2H:1V GRANULAR FILL (SHORT-TERM)

FIGURE 1

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	B-bar	Add Weight
	Bedrock	Bedrock (Impenetrable)				0	No
	Compact Gravelly Sand Fill	Mohr-Coulomb	21	0	31	0	No
	Compact Sand Gravel Fill	Mohr-Coulomb	21	0	30	0	No
	Granular A or B Type II	Mohr-Coulomb	22	0	35	0	Yes
	Stiff Silty Clay	Mohr-Coulomb	18	0	21	0.75	No
	Surcharge material	Mohr-Coulomb	12	0	0	0	Yes
	Very Stiff Silty Clay Fill	Mohr-Coulomb	19	0	28	0.5	No



NORRIS WHITNEY BRIDGE
NORTH APPROACH (STA 21+250)
2H:1V GRANULAR FILL (LONG-TERM)

FIGURE **2**

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	B-bar	Add Weight
	Bedrock	Bedrock (Impenetrable)				0	No
	Compact Gravelly Sand Fill	Mohr-Coulomb	21	0	31	0	No
	Compact Sand Gravel Fill	Mohr-Coulomb	21	0	30	0	No
	Granular A or B Type II	Mohr-Coulomb	22	0	35	0	Yes
	Stiff Silty Clay	Mohr-Coulomb	18	0	21	0	No
	Surcharge	Mohr-Coulomb	12	0	0	0	Yes
	Very Stiff Silty Clay Fill	Mohr-Coulomb	19	0	28	0	No

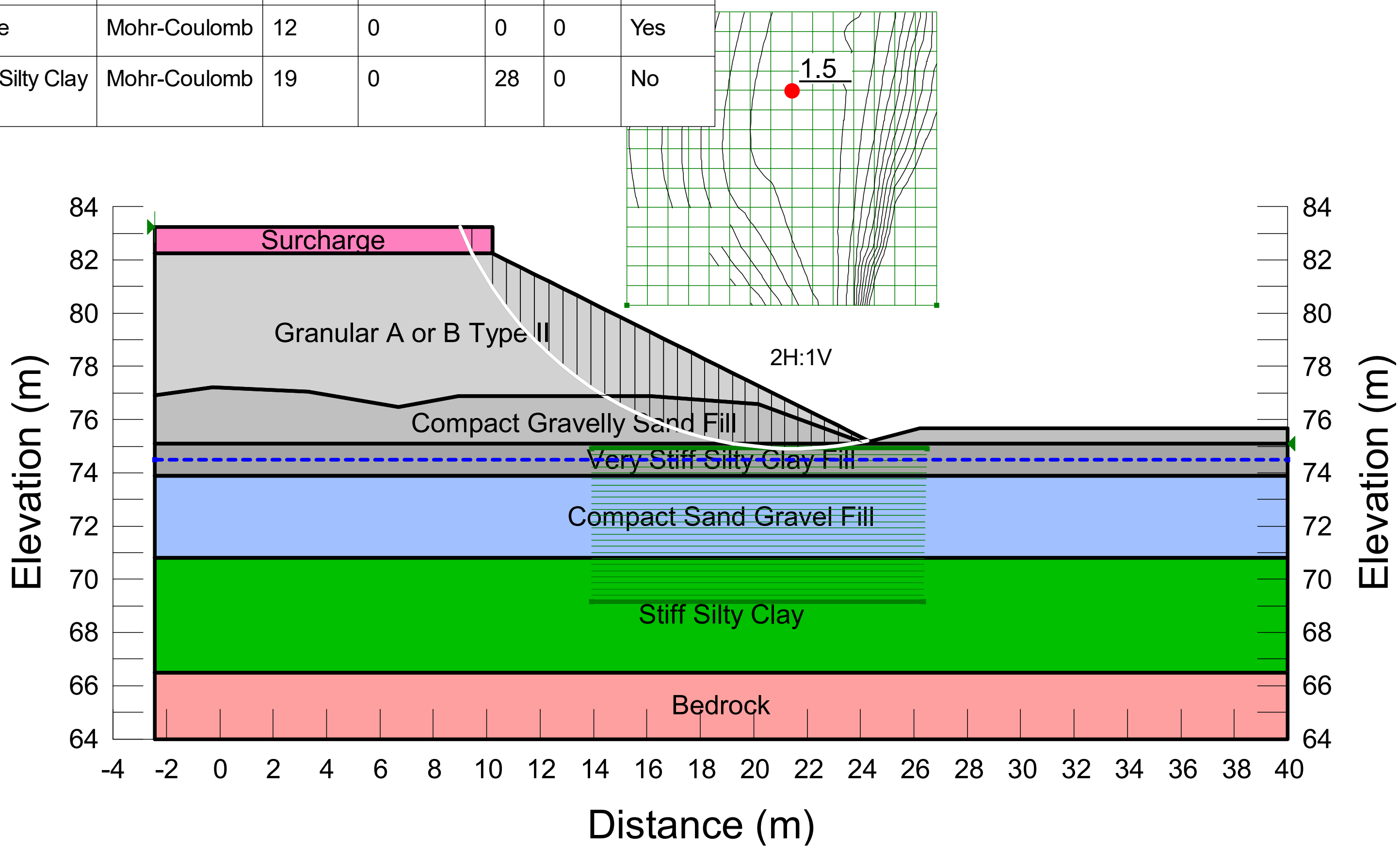
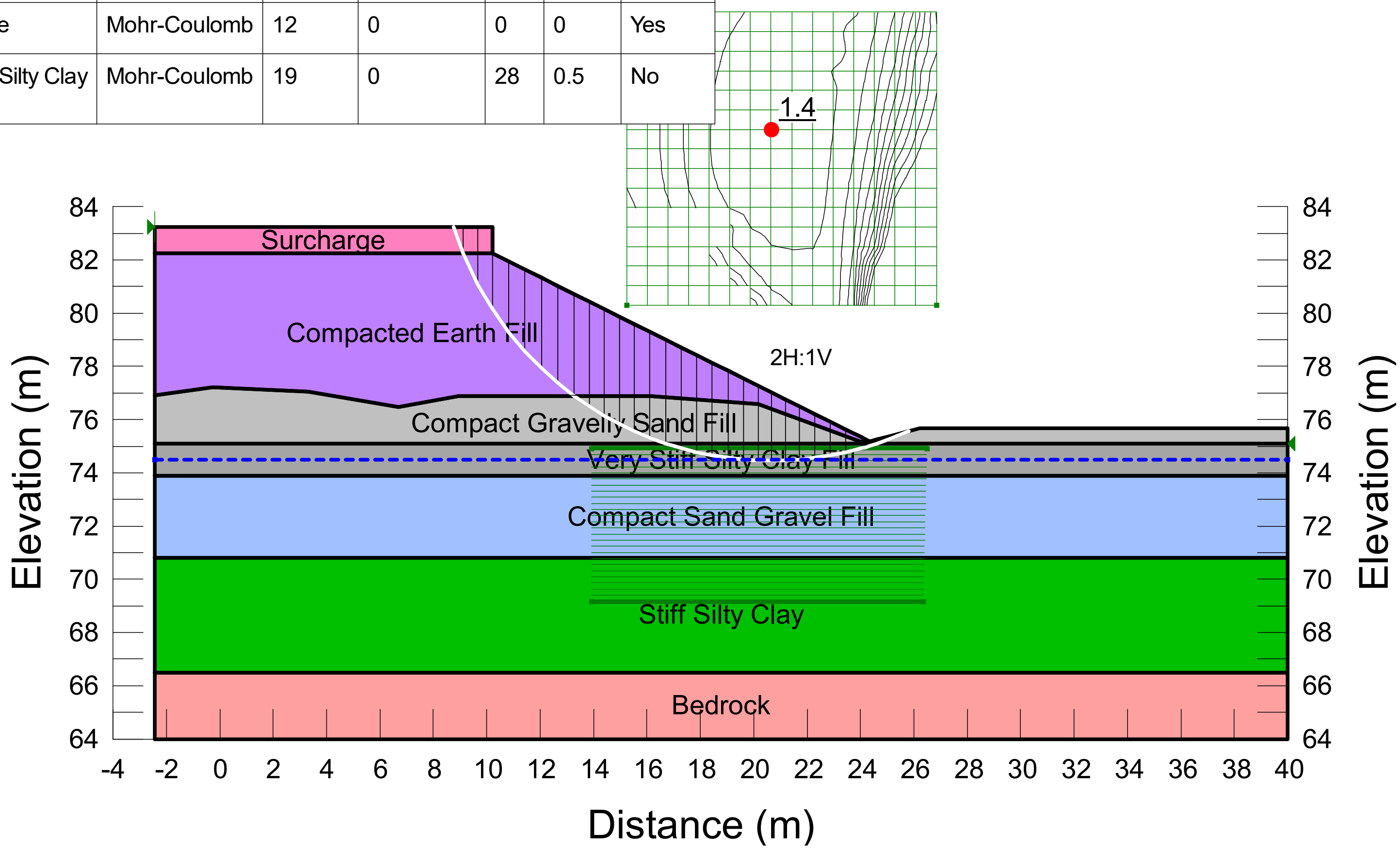


FIGURE 3

NORRIS WHITNEY BRIDGE
NORTH APPROACH (STA 21+250)
2H:1V EARTH FILL (SHORT-TERM)

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	B-bar	Add Weight
	Bedrock	Bedrock (Impenetrable)				0	No
	Compact Gravelly Sand Fill	Mohr-Coulomb	21	0	31	0	No
	Compact Sand Gravel Fill	Mohr-Coulomb	21	0	30	0	No
	Compacted Earth Fill	Mohr-Coulomb	21	0	30	0	Yes
	Stiff Silty Clay	Mohr-Coulomb	18	0	21	0.75	No
	Surcharge	Mohr-Coulomb	12	0	0	0	Yes
	Very Stiff Silty Clay Fill	Mohr-Coulomb	19	0	28	0.5	No



NORRIS WHITNEY BRIDGE
NORTH APPROACH (STA 21+250)
2H:1V EARTH FILL (LONG-TERM)

FIGURE **4**

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	B-bar	Add Weight
	Bedrock	Bedrock (Impenetrable)				0	No
	Compact Gravelly Sand Fill	Mohr-Coulomb	21	0	31	0	No
	Compact Sand Gravel Fill	Mohr-Coulomb	21	0	30	0	No
	Compacted Earth Fill	Mohr-Coulomb	21	0	30	0	Yes
	Stiff Silty Clay	Mohr-Coulomb	18	0	21	0	No
	Surcharge	Mohr-Coulomb	12	0	0	0	Yes
	Very Stiff Silty Clay Fill	Mohr-Coulomb	19	0	28	0	No

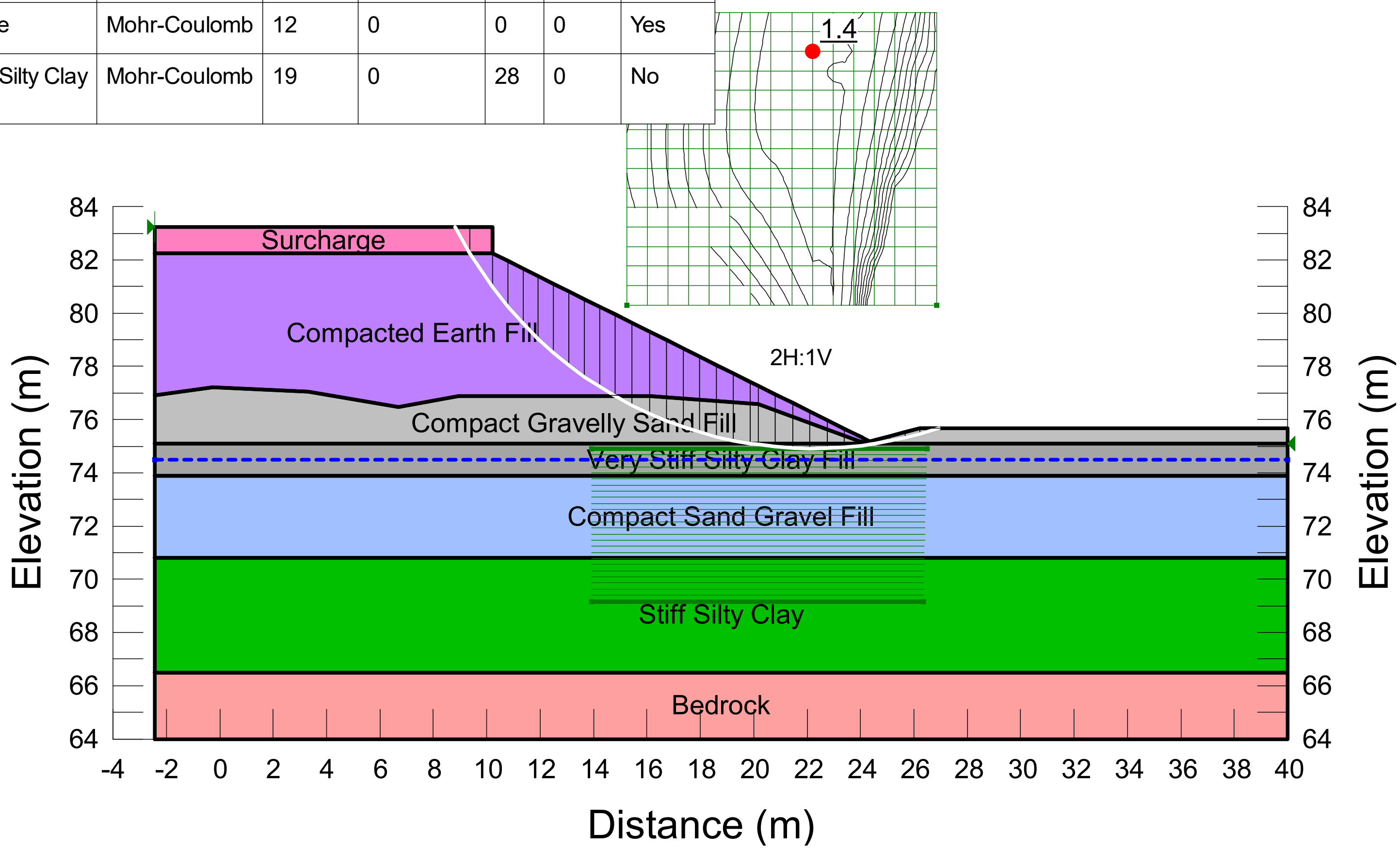
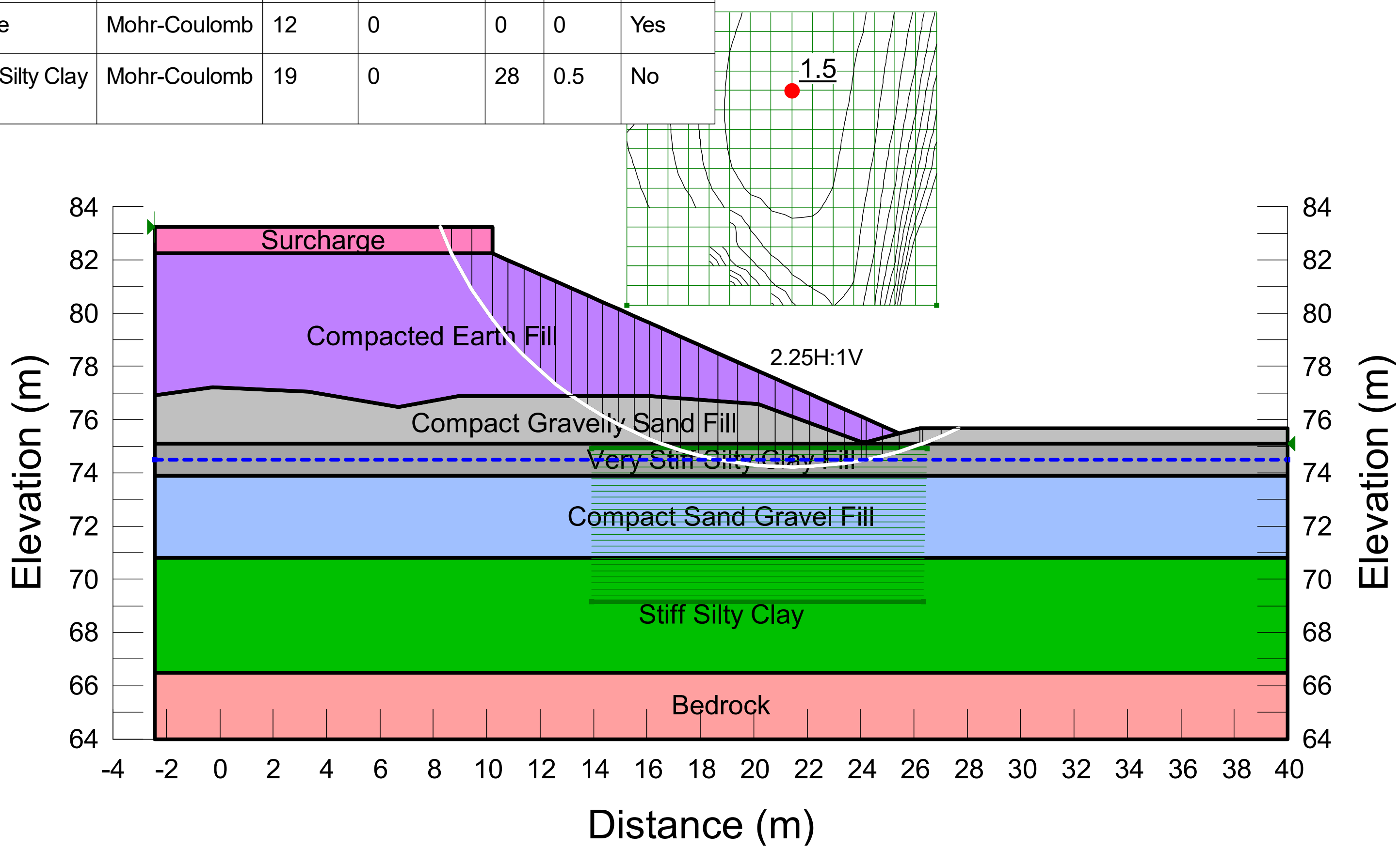


FIGURE **5**

NORRIS WHITNEY BRIDGE
NORTH APPROACH (STA 21+250)
2.25H:1V EARTH FILL (SHORT-TERM)

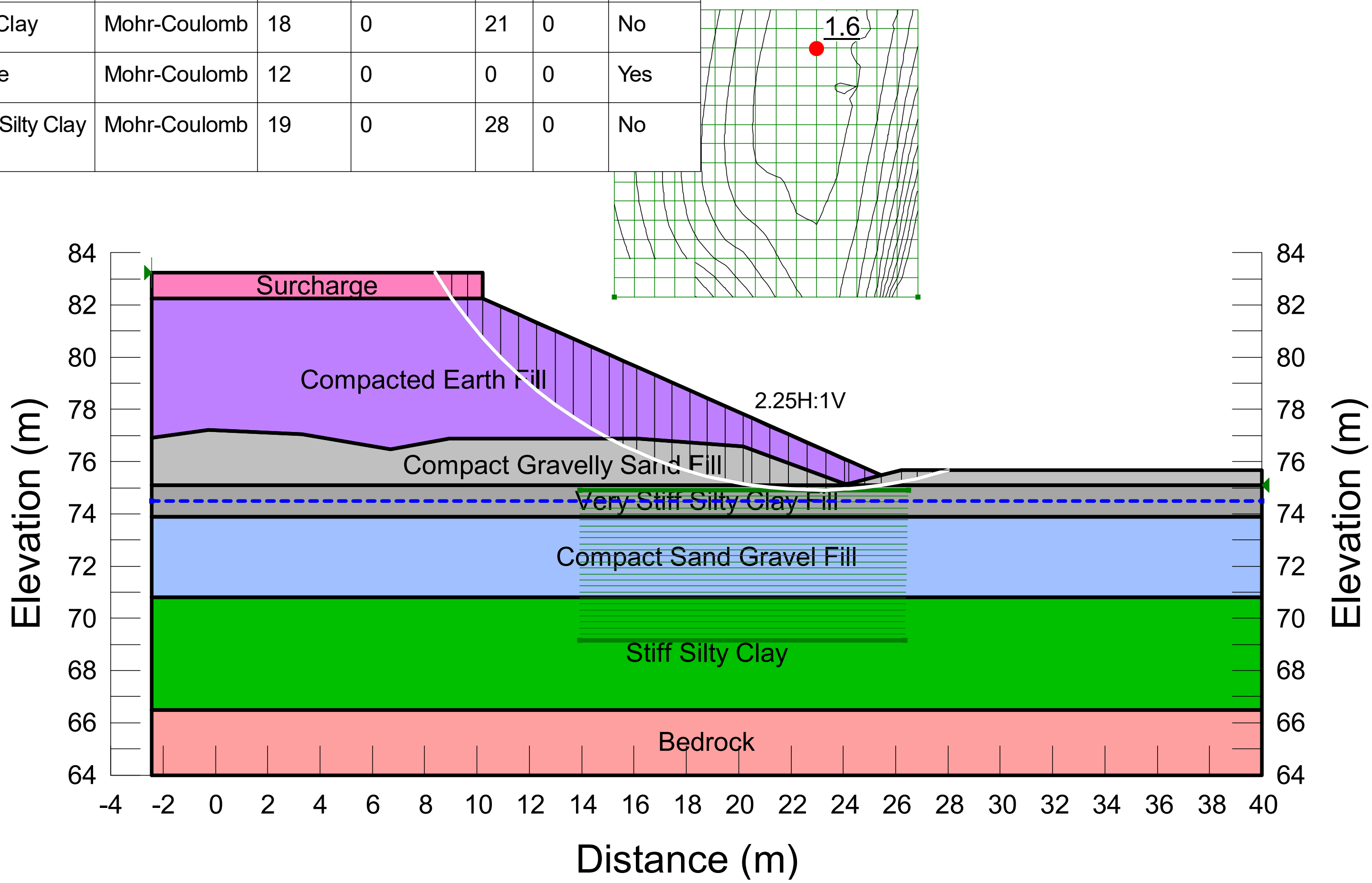
Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	B-bar	Add Weight
	Bedrock	Bedrock (Impenetrable)				0	No
	Compact Gravelly Sand Fill	Mohr-Coulomb	21	0	31	0	No
	Compact Sand Gravel Fill	Mohr-Coulomb	21	0	30	0	No
	Compacted Earth Fill	Mohr-Coulomb	21	0	30	0	Yes
	Stiff Silty Clay	Mohr-Coulomb	18	0	21	0.75	No
	Surcharge	Mohr-Coulomb	12	0	0	0	Yes
	Very Stiff Silty Clay Fill	Mohr-Coulomb	19	0	28	0.5	No



NORRIS WHITNEY BRIDGE
NORTH APPROACH (STA 21+250)
2.25H:1V EARTH FILL (LONG-TERM)

FIGURE **6**

Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	B-bar	Add Weight
	Bedrock	Bedrock (Impenetrable)				0	No
	Compact Gravelly Sand Fill	Mohr-Coulomb	21	0	31	0	No
	Compact Sand Gravel Fill	Mohr-Coulomb	21	0	30	0	No
	Compacted Earth Fill	Mohr-Coulomb	21	0	30	0	Yes
	Stiff Silty Clay	Mohr-Coulomb	18	0	21	0	No
	Surcharge	Mohr-Coulomb	12	0	0	0	Yes
	Very Stiff Silty Clay Fill	Mohr-Coulomb	19	0	28	0	No



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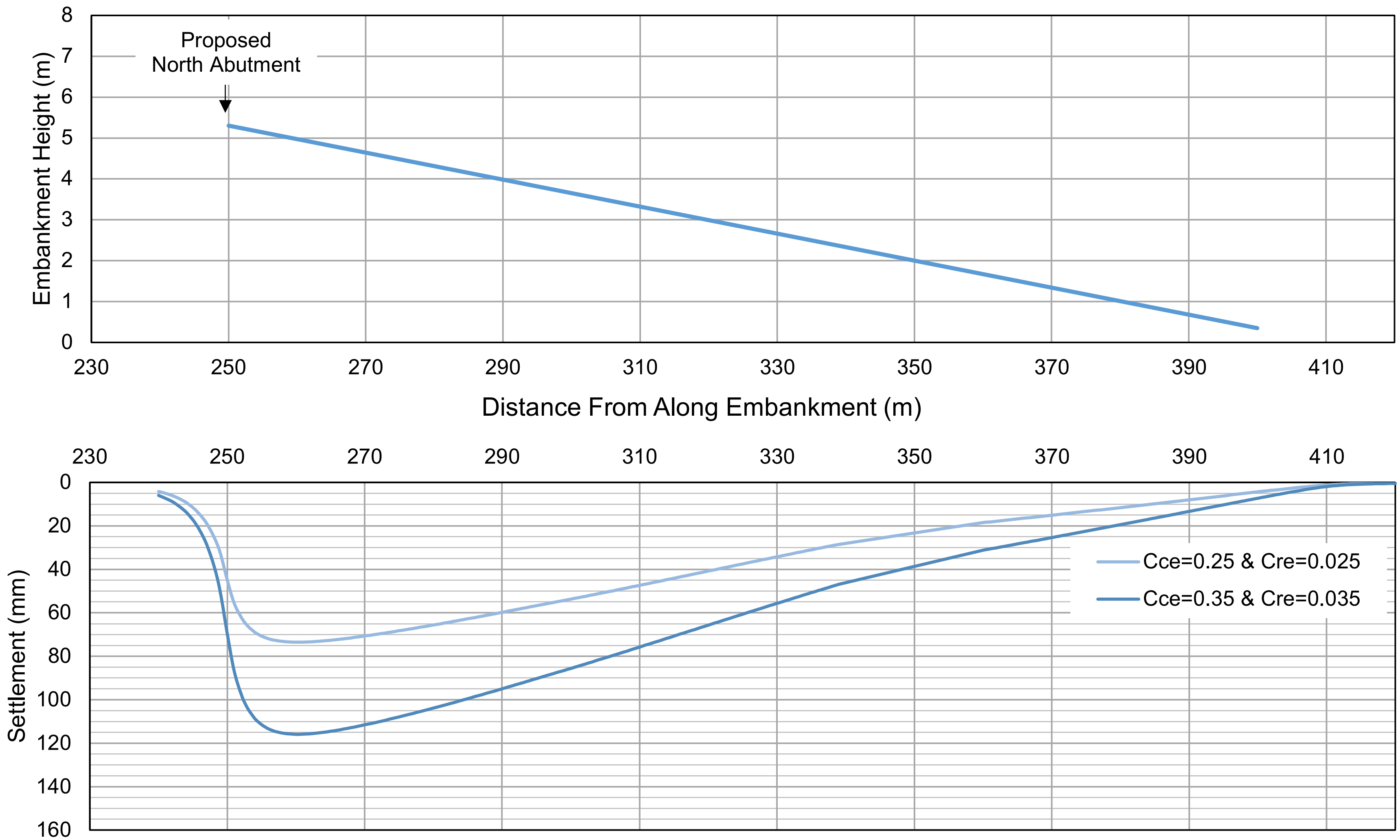
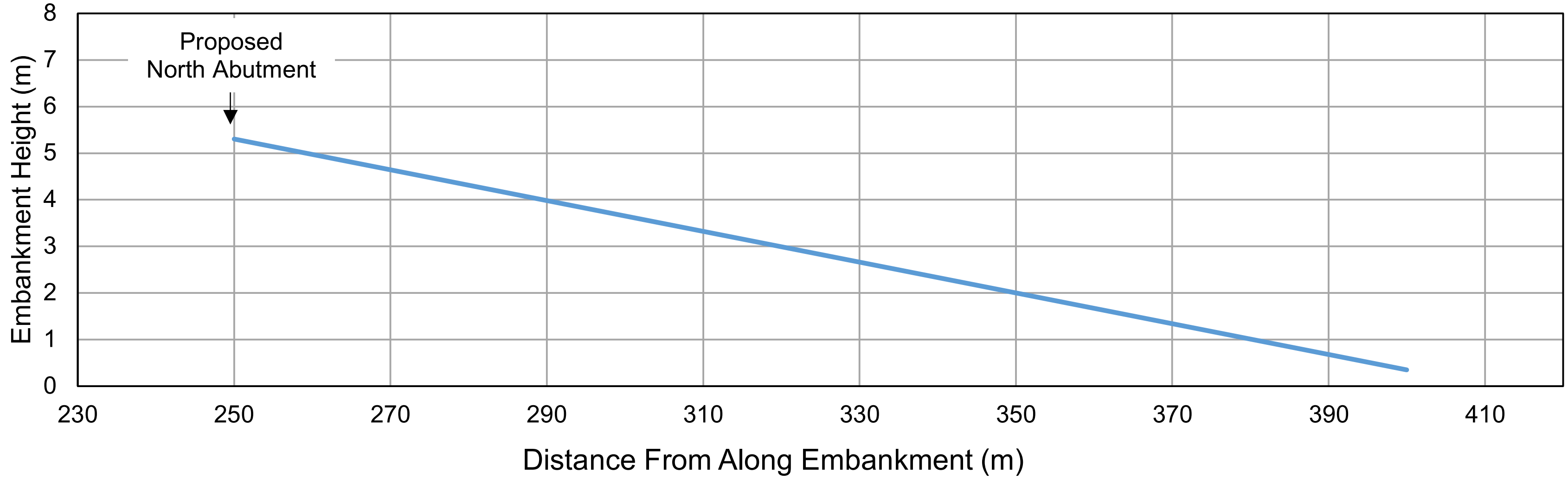
Appendix G

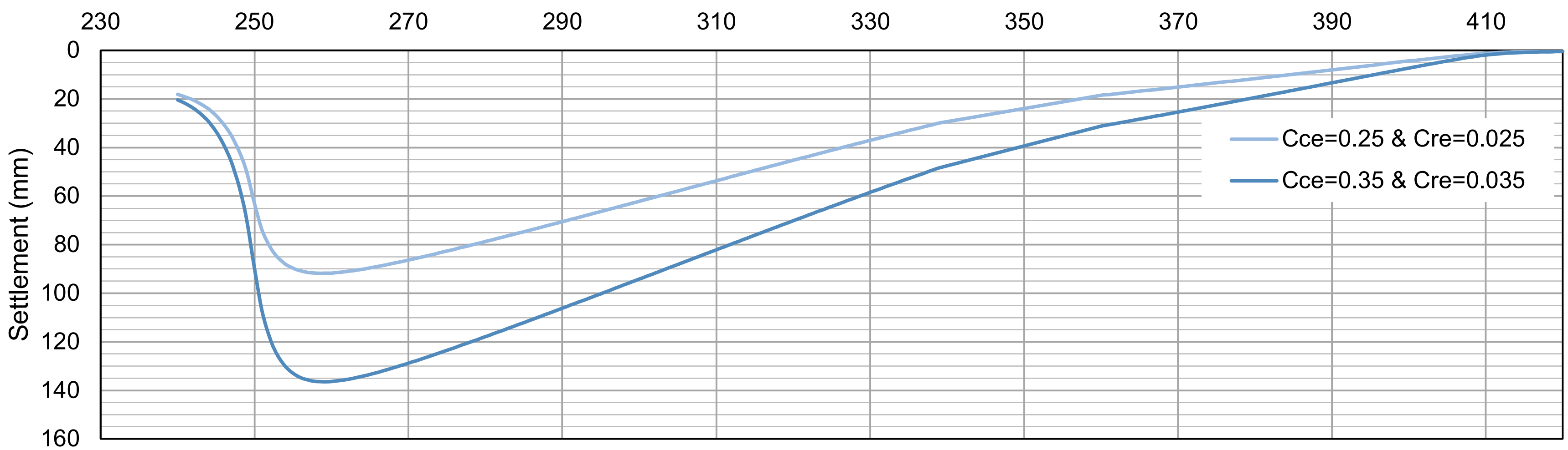
Settlement Analyses Results



Estimated Ground Settlement

THURBER ENGINEERING LTD.	**Estimated Ground Settlement Along NBL Embankment Centerline**	**After 6 months**
	11566 - Norris Whitney	**Figure 7**





Estimated Ground Settlement

THURBER ENGINEERING LTD.	Estimated Ground Settlement Along NBL Embankment Centerline	After 20 years
	11566 - Norris Whitney	Figure 8

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Appendix H

Preliminary Foundation Investigation and Design Memo, dated April 15, 2016

THURBER ENGINEERING LTD.

MEMORANDUM

To: Christopher Schueler, P.Eng.
AECOM

Date: April 15, 2016

From: Murray Anderson, P.Eng.
Alastair Gorman, P.Eng.
(Reviewed by P.K. Chatterji, P.Eng.)

File: 19-4406-20

**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN
NORRIS WHITNEY BRIDGE REHABILITATION (SITE 28-28)**

1 INTRODUCTION

This memo presents a brief summary of the factual findings from a foundation review carried out for the existing Norris Whitney Bridge on Highway 62 in the Geographic Township of Ameliasburgh – Municipality of Belleville and Prince Edward County, Ontario. It also presents preliminary geotechnical recommendations for use in assessment of the existing foundations at the site. It is noted that the proposed structural alternatives for the rehabilitation are not yet defined.

The recommendations provided in this memorandum are for planning, structure evaluation and preliminary design purposes only. Additional investigation and analysis may be required in any subsequent detail design phase of the project.

The following reference numbers apply to this site:

- Current W.P. 4082-13-01
- Site No. 28-28
- GEOCRES No. 31C-135
- Historic W.P. 134-74-01

2 SITE DESCRIPTION

The Norris Whitney Bridge carries Highway 62 over the Bay of Quinte between Belleville at the north end and the hamlet of Rossmore at the south end. The RFP information indicates that the existing bridge was constructed in 1982 and consists of an 11 span steel plate I-girder bridge with a total length of 881 m (58 m + 9 x 85 m + 58 m). The bridge has an overall deck width of 12 m and accommodates two lanes of traffic and a pedestrian sidewalk.

The General Layout drawing for the existing bridge (Contract No. 80-34) indicates that the top of pavement elevation on the bridge rises from Elev. 86.9 at the north abutment to Elev. 100.3 at the highest point, and then falls to Elev. 79.2 at the south abutment. The highest point is located between the fourth and fifth piers from the north end, and road grade at this location is approximately 25.6 m above the water level indicated on the drawing (Elev. 74.89).

The natural terrain in the vicinity of the bridge is generally flat. In the GEOCRES report, the depth of water in the bay was reported to be typically 3.0 to 4.5 m, locally about 10.7 m within the former main navigation channel adjacent to the south shore. A discontinuous causeway from a former crossing alignment runs parallel to the west side of the existing bridge.

3 SUBSURFACE CONDITIONS

The site is located across the boundary of two physiographic regions known as the Napanee Plain on the north shore and the Prince Edward Peninsula on the south shore. The Napanee Plain is a flat to undulating limestone plain largely stripped of overburden. The Prince Edward Peninsula is a low plateau of limestone projecting into the eastern part of Lake Ontario. Geologic maps indicate that the bedrock consists of interbedded limestone and shale of the Verulam Formation.

A site investigation was completed by the Ministry of Transportation and Communications between May 24 and June 17, 1977 for the proposed construction of the current high level bridge. A total of 21 boreholes and eight dynamic cone penetration tests (DCPTs) were advanced to depths ranging from 0.5 to 20.7 m during the investigation. The boreholes were terminated upon refusal on probable bedrock or after recovery of at least one run of bedrock core. The available GEOCRES files are attached in Appendix A.

The soil conditions encountered in the boreholes were variable as a result of dredge and fill operations previously carried out in the area. Up to 4.6 m of mixed fill was encountered in boreholes drilled on the north shore, and up to 13.9 m of organic clay and/or sandy gravel fill (related to the former causeway crossing) was encountered on the bay bottom, typically increasing in thickness to the south. These materials were not present adjacent to the south shore where dredging was carried out in the former main navigation channel.

The fill and organic soils on the north half of the site are underlain by a 3.4 to 4.0 m thick layer of native sand to sandy gravel, overlying a 2.7 to 4.0 m thick layer of clay to clayey silt. A 0.6 to 5.5 m thick layer of glacial till comprising sand and gravel with various proportions of silt and clay was encountered below all of the above soils along essentially the full alignment.

Standard Penetration Test (SPT) N-values recorded in the various units and the corresponding consistency/relative density of the layers are summarized below:

Soil Unit	SPT N-values (blows/0.3 m)	Relative Density	Consistency
Fill	2 to 120, typically 2 to 32	Very loose to dense	-
Organic Clay	0 to 19, typically 1 to 10	-	Very soft to stiff
Sand to Sandy Gravel	5 to 53	Loose to very dense	-
Clay to Clayey Silt	2 to 24	-	Soft to very stiff
Glacial Till	15 to 80 (for 75mm), locally 3 to 9	Compact to very dense	Very stiff to hard

The undrained shear strength of the organic clay measured by field and laboratory vane testing ranged from 1.4 to 46 kPa. Similar testing of the clay and clayey silt yielded undrained shear strengths of 37 to 135 kPa.

Bedrock or refusal on probable bedrock was encountered below the organic clay, sandy gravel and glacial till in all but four of the boreholes at depths of 0.5 to 19.0 m below the ground or water surface. In general, the bedrock surface falls from approximate Elev. 76.7 (0.5 m depth) some 50 m south of the south abutment, to Elev. 55.9 (19.0 m depth) approximately 230 m north of the south abutment, and then rises to Elev. 68.4 (8.1 m depth) approximately 165 m north of the north abutment. The inferred bedrock elevations at the south and north abutments are approximate Elev. 69.5 and 65.5, respectively.

Bedrock was proven by recovering up to 1.5 m of rock core in 11 of the boreholes. The bedrock was described as sound limestone with shaly sections. RQD values ranged from 50% to 97%, indicating a fair to excellent quality rock.

Groundwater levels at about 0.3 and 0.9 m depth (Elev. 75.0) were observed in two boreholes drilled on land. The water level in the bay was at Elev. 74.9 at the time of the fieldwork. The water was typically 3.0 to 4.5 m deep at the borehole locations in the bay, locally about 10.7 m within the former main navigation channel adjacent to the south shore.

4 SITE OBSERVATIONS

Foundations engineering staff from Thurber visited the site to observe conditions related to the geotechnical performance.

No obvious signs of settlement or distress of the existing bridge foundations were observed.

The approach embankments appeared to be stable, with no obvious signs of instability or bulging. Rock protection is provided at the abutments along the shoreline. Possible erosion was noted at a drain outlet on the east side of the north approach embankment.

Photographs of the structure and the approaches are attached in Appendix B.

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5 EXISTING FOUNDATIONS

The archive design drawings indicate that the bridge is supported on steel pile foundations bearing on bedrock. The layout of the piles is as follows:

- The south abutment is supported on nine HP 310x110 piles driven to bedrock. The front row of piles contains five piles battered forward at 1H:3.5V, and the hind row contains two inner piles inclined forward at 1H:3.5V and two outer piles inclined backwards at 1H:8V.

- Each pier is supported by a pile cap carried on 10 to 14 composite piles comprising three HP 310x110 piles enclosed in steel tube piles (1219 mm outside diameter by 12.7 mm thick) filled with tremie concrete after installation of the H-piles. The H-piles are socketed at least 500 mm into sound bedrock in a 1050 to 1200 mm diameter socket filled with tremie concrete. The tube piles are driven to bedrock. The perimeter composite piles are inclined outwards at a batter of 1H:4.5V, and the two interior piles, where present, are vertical.

- The north abutment is supported on ten HP 310x110 piles driven to bedrock. The front row of piles contains six piles battered forward at 1H:3V, and the hind row contains two inner piles inclined forward at 1H:3V and two outer piles inclined backwards at 1H:8V. Additionally, a vertical H-pile is driven to bedrock under each wing wall.

An underwater inspection of the pile caps was carried out in 2007 by Harmer Podolak Engineering Consultants Inc. The inspection involved visual examination of the submerged portion of the concrete pile caps at the piers, between the water line and the bay bottom. At four of the piers, the top 0.6 to 5.1 m of the tube piles below the pile caps was accessible for examination. In general, the pile caps were assessed to be in good condition with light to medium scaling and localized spalls. The exposed tube piles were considered to be in good condition with light surface rust. One perforation was noted on the north side of a pile at the sixth pier from the south.

6 ASSESSMENT OF EXISTING FOUNDATIONS

The archive information and site observations indicate that the existing bridge is founded on piles to bedrock. The foundations appear to be performing satisfactorily, and it can be assumed that the foundations will continue to perform satisfactorily in the future provided they are structurally sound.

The RFP document suggests that the required rehabilitation work will consist of concrete deck repairs, waterproofing, paving, expansion joint replacement, substructure repairs, and barrier wall repair/replacement. In this case, no appreciable increase in the loading is anticipated.

If a significant (greater than 10%) increase in loading on the foundations due to rehabilitation of the bridge is subsequently planned, it will be necessary to carry out further assessment and possibly site investigation and field testing to support the preparation of foundation design recommendations.

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7 EXCAVATION AND ROADWAY PROTECTION

If the selected rehabilitation strategy requires excavation in the approach fills behind the abutments, it is recommended that site investigation and field testing be carried out in each approach fill in order to characterize the fill and bedrock, and to select parameters for the design of roadway protection. One borehole within each approach fill and within the probable extent of excavation is considered to be appropriate. The boreholes should extend for the full depth of fill or to twice the depth of excavation, whichever is the greater, or terminate on bedrock if encountered within this depth.

8 CLOSURE

The factual subsurface information used in the preparation of this memorandum was taken from the report by The Ministry of Transportation and Communications titled “Feasibility Foundation Investigation Report for Bay of Quinte Crossing at Belleville, Hwy. 14, District 8, Kingston”, WP 134-74-01, Site 28-28, undated.

The memorandum was prepared by Mr. Murray Anderson, P.Eng., Senior Foundations Engineer and was reviewed by Mr. Alastair Gorman, P.Eng. and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.

Murray Anderson, P.Eng.
Associate, Senior Foundation Engineer

Alastair Gorman, P.Eng.
Associate, Senior Foundation Engineer

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

Attachments

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Appendix A

GEOCRES Report, Correspondence, and Archive Drawings

DOCUMENT MICROFILMING IDENTIFICATION

G.I.-30 SEPT. 1976

GEOCRES No. 31C-135

DIST. 8 REGION

W.P. No. 134-74-01

CONT. No. 80-34

W. O. No.

STR. SITE No. 28-28

HWY. No. 14

LOCATION Bay of Quinte, Belleville

No of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

ENGINEERING MATERIALS OFFICE
SOIL MECHANICS SECTION

W P 134-74-01 DIST 8

HWY 14 STR SITE 28-28

Bay of Quinte Crossing
at Belleville

DISTRIBUTION

T.C. Kingsland
E.R. Saint
S.C. Radbone
R.W. Franks (2)

K.G. Bassi
G.A. Wrong
B.J. Giroux
R.S. Pillar

R. Hore

R. Forest)
J. Anderson) cover only
G. Sloan)

Files

GEOCRES 31C-135 DATE

FEASIBILITY
FOUNDATION INVESTIGATION REPORT

For

Bay of Quinte Crossing at Belleville
Hwy. 14, District 8, Kingston
W.P. 134-74-01, Site 28-28

INTRODUCTION

The Ministry of Transportation and Communications has proposed to replace the existing Hwy. 14 crossing of the Bay of Quinte. The crossing is presently accomplished by means of a causeway and two steel through truss type structures, one structure being a swing bridge for the purpose of accommodating marine traffic on the waterway. A route location study resulted in the decision to build a high-level structure, east of, and parallel to the existing crossing.

Because of the large scope of the project, the Soil Mechanics Section was requested to carry out a feasibility study to determine the subsurface conditions along the line of the proposed crossing. The request was contained in a memorandum dated March 22, 1977 from Mr. T.C. Kingsland, Kingston Regional Structural Planning Engineer. An investigation was subsequently carried out to establish the subsoil, bedrock and groundwater conditions existing at the site.

The pertinent factual data were provided immediately after completion of the fieldwork. In addition, recommendations were also provided verbally. This report contains the factual data obtained from the field and laboratory work, together with recommendations pertaining to the design and construction of the structure and associated approach fills.

FIELD INVESTIGATION

The fieldwork was carried out during the period of May 24, 1977 to June 17, 1977. A total of 21 boreholes, 8 accompanied by dynamic cone penetration tests, were put down to depths of 67 feet below the bay water surface. The borings on the bay were put down by means of diamond drilling techniques using NX casing operating from a drum-floating raft. The remaining boreholes were advanced by means of a muskeg vehicle equipped with hollow stem continuous flight augers. Bedrock was proven by obtaining up to 5 feet of BXL size rock core.

SITE DESCRIPTION: PAST AND PRESENT CONDITIONS

The proposed crossing on the Bay of Quinte will connect Zwick's Park in the Town of Belleville with the Hamlet of Rossmore in Prince Edward County (Refer to Photo 1 in the Appendix).

The north approach will extend through generally flat terrain immediately east of existing Hwy. 14 within Zwick's Park (Refer to Photo 2 in the Appendix). Maps of the area prior to 1890 show the area now occupied by Zwick's Park to be largely open water except for an island. This piece of land, about 4 acres in area, was named Zwick's Island and now comprises the southwestern portion of Zwick's Park. The island was joined to the mainland by means of a causeway as part of the original 1890 Bay of Quinte Bridge. It is believed that this causeway resulted in the development of the swamp areas as shown in maps as early as the 1920's. In the 1960's the swamp area was used for sanitary landfill. The western portion has been sodded and extensively developed into a park for day users. The municipality of Belleville is presently placing clean landfill east of Hwy. 14 with the intention of developing it for recreational uses.

On the Prince Edward County side the approaches will extend partially through residential areas within the Hamlet of Rossmore (Refer to Photo 3 in the Appendix).

At the location of the proposed crossing, the Bay of Quinte is some 3000 feet wide. The depth of the main channel, at the swing bridge location, is up to 35 feet deep. Elsewhere along the proposed crossing, the bay is 10 to 15 feet deep.

The existing crossing from Zwick's Park to Rossmore, some 2750 feet, is accomplished by means of an earth causeway and two steel through truss type structures. The main structure adjacent to the Rossmore side is composed of one swing span and three fixed spans (128' fixed, 200' swing, 165' fixed and 128' fixed); and about 1000 feet north of the swing span, is a two fixed span structure (99', 108').

The original structure completed in 1891, was composed of about seventeen spans between Zwick's Island and Rossmore. The present causeway was constructed in the 1920's by placing dredged material between the piers. The original superstructure was removed and the causeway was brought up to final grade leaving the original piers in place. These piers are constructed of stone, timber and concrete cribwork and are believed to be supported on timber piles. The causeway was completed by placing 10-20 ton armour stone on the side slopes for protection against wave action (Refer to Photo 5 in the Appendix).

GEOLOGY

The site borders the physiographic regions of the "Napanee Plains" and the "Prince Edward Peninsula". These regions are characterized by a thin veneer of glacial drift underlain by generally flat to undulating limestone of the Trenton-Black River Formation. At this site, limestone and shale bedrock outcrops appear on the Prince Edward County shore in the vicinity of existing Hwy. 14 (Refer to Photo 4 in the Appendix).

SUBSOIL DESCRIPTION

General

Subsoil across the site is quite variable as a result of dredging and landfill operations carried out in this area in the past. The parent subsoil consists of 11 to 13 feet of sandy gravel or medium to coarse sand underlain by 9 to 13 feet of clay or clayey silt which in turn overlies a 5 to 18 foot thick deposit of glacial till. The glacial till is underlain by limestone bedrock. On the Prince Edward County shore subsoil consists of 2 to 11 feet of sandy gravel overlying bedrock. In Zwick's Park the parent subsoil is overlain by up to 15 feet of fill material, whereas, within the bay the parent subsoil is overlain by a thin veneer up to 9 feet thick of very soft organic clay.

East of the existing causeway from the Prince Edward County side to about 1000 feet north of the shoreline, the parent subsoil beneath the bay has been dredged, in some locations down to the bedrock surface. In a few locations, up to 45 feet of organic clay has been recently deposited within the dredged areas. However, within the main channel the current has kept the dredged areas relatively free of organic clay deposits.

The locations and elevations of the borings, together with a stratigraphical profile and sections inferred from borehole data, are shown on Drawing No. 1347401-A.

A brief description of the various subsoil and bedrock types encountered and the groundwater conditions are presented in the paragraphs to follow.

Fill Material

Fill material was encountered in all borings put down in the Zwick's Park area and in the existing causeway. Fill material was also encountered in two boreholes (B.H.'s 17 & 20) put down in the bay adjacent to the causeway.

Fill material in Zwick's Park: The fill material encountered in Zwick's Park is estimated to be 3 to 15 feet thick. This fill material varies in composition from a sandy gravel to a sand with silt and inclusions and/or pockets of clayey silt. Typical grain size distribution curves for the fill material are shown in envelope form on Figure 1 of the Appendix. The results of Atterberg Limit testing on representative samples from the cohesive zones of clayey silt are plotted on the Plasticity Chart,Figure 2. The Atterberg Limits indicate that the clayey silt pockets are inorganic and of low plasticity. This fill material also contains inclusions of wood chips and organics. In these areas the organic content was found to be as high as 7% by weight.

Standard Penetration testing carried out in the fill material in Zwick's Park gave a range of 'N' values of 2 to 32 blows per foot indicating this fill has undergone slight to moderate compaction.

Fill material in the causeway: The fill material within the causeway and adjacent to it was found to be up to 41 feet deep. The composition of this fill material is a gravelly sand with a trace of silt. The result of grain size distribution testing is shown in an envelope form on Figure 1. This fill material contains a trace of shells and wood chips in isolated zones. The organic content in these zones was found to be as high as 13% by weight; however, this high organic content is attributed to the presence of wood chips and is not indicative of the deposit as a whole.

The range of Standard Penetration Test 'N' values for this fill material is 8 to 92 blows per foot, indicating that the material has been subject to a non-uniform compactive effort.

Organic Clay

This material comprises the bay bottom, being generally a thin veneer up to 9 feet thick covering the parent subsoil. In some areas where the bay bottom has been dredged for causeway fill, the bay has filled up to 46 feet of organic clay within these dredged areas. In one boring in Zwick's Park area a deposit of organic clay about 7 feet thick was encountered immediately below the fill. The material in this deposit is black, being generally plastic and composed of organic clay. Where the very deep deposits of organic clay were encountered in the bay bottom, the organic material contains appreciable amounts of silt and sand. The organic content of the deposit as determined by laboratory testing ranges from 3 to 26% by weight.

The results of laboratory and field testing are summarized below:

Moisture Content, Bulk Density and Atterberg Limits

		Range	Average
Natural Moisture Content	($W\%$)	34-280	167
Liquid Limit	($W_L\%$)	55- 96	74
Plastic Limit	($W_p\%$)	29- 85	65
Plasticity Index	($I_p\%$)	11- 26	17

Undrained Shear Strength Su

		Range	Sensitivity
Laboratory Vane Tests	(psf)	50-490	3
Field Vane Tests	(psf)	30-960	3

The Atterberg Limits indicate that the material is organic and of high plasticity. The natural moisture content generally decreases with depth while the undrained shear strength generally increases with depth. The undrained shear strength indicates that the deposit has a very soft to firm consistency.

Sandy Gravel to Sand

This granular deposit was encountered immediately below the fill in Zwick's Park below the organic clay deposit of the bay bottom south of the park and also immediately below the ground surface on the Prince Edward County side. On the Prince Edward County side this deposit ranges in thickness from 2 to 11 feet. Elsewhere, the thickness of this deposit varies from 11 to 13 feet. This granular stratum is composed of sandy gravel or medium to coarse sand. The results of grain size distribution testing performed on representative samples from this stratum are summarized in envelope form on Figure 3.

Standard Penetration testing gave 'N' values ranging from 4 to 53 blows per foot, generally increasing with depth. Based on these values the deposits are estimated to have a compact to very dense relative density.

Clay

This stratum was encountered in three borings (B.H. #4, 7 & 8) put down in Zwick's Park and also in three borings put down in the bay east of the causeway and north of the fixed span structure (B.H. #10, 11 & 13). This cohesive deposit was found beneath the stratum of sandy gravel to sand and also in some locations beneath the deposit of organic clay. The thickness of the deposit is estimated to be between 9 and 13 feet. The deposit is composed of clay which is somewhat fissured and laminated. In two locations (B.H. #4 & 7) part of this deposit was found to have random layers of clayey silt.

The results of laboratory and field testing on representative samples taken from this stratum are summarized below.

Natural Moisture Content, Atterberg Limits and Bulk Density

		Clay		Clayey Silt Layers	
		Range	Average	Range	Average
Natural Moisture Content	(W%)	42-72	56	30-44	36
Liquid Limit	(W_L%)	58-80	69	22-35	31
Plastic Limit	(W_p%)	19-24	23	14-24	18
Plasticity Index	(I_p%)	39-54	46	8-17	12
Bulk Density	(γPCF)	97-109	104		

Undrained Shear Strengths (Su. P.S.F.)

	Range	Sensitivity
Field Vane Tests	800->2400	3-8
Laboratory Vane Tests	765-2830	2-4
Laboratory Unconfined Tests	825-1940	
Laboratory Quick Triaxial Tests	1740-1880	

Consolidation Tests (3 tests)

Initial Void Ratio	e_o	1.2-2.0
Coefficient of Consolidation	c_c	0.4-1.4
Degree of Preconsolidation	$P'_c-P'_o$ (PSF)	3200-6600

The results of the Atterberg Limit testing are plotted on the Plasticity Chart, Figure 4. The Atterberg Limit testing indicates that the clay deposit is generally inorganic and of high plasticity, whereas the layers of clayey silt are inorganic and of low plasticity. The Natural Moisture Content is generally between the Plastic Limit and the Liquid Limit. The consolidation testing gave a range of preconsolidation pressure of 3200 to 6600 P.S.F. in excess of the existing effective overburden pressure.

The undrained shear strength as measured by laboratory and in situ testing ranges from greater than 2400 P.S.F. to 760 P.S.F. decreasing with depth. The sensitivity as measured by vane testing (both laboratory and field testing) indicates that in general the deposit is slightly to moderately sensitive to remoulding. Furthermore, the undrained shear strengths indicate that the consistency of the deposit varies from very stiff in the upper portion changing to firm, generally decreasing with depth.

Clayey Silt

This deposit was encountered in two locations; one in Zwick's Park (B.H. #5) immediately below the sandy gravel to sand deposit and one in the bay (B.H.#12) immediately below the sandy gravel deposit. The thickness of this deposit is estimated to be 9 feet thick. The material in this stratum is clayey silt and a trace of sand with random silt and sand seams. The results of the laboratory and field testing are summarized as follows:

Moisture Content and Atterberg Limits

		Range	Average
Natural Moisture Content	(W%)	22-44	33
Liquid Limit	(W_L%)	33-36	34
Plastic Limit	(W_p%)	15-23	18
Plasticity Index	(I_p%)	10-21	16

Undrained Shear Strength (Su P.S.F.)

	Range	Sensitivity
Field Vane Tests	1000-1600	2-5

The results of the Atterberg Limit testing are shown on Figure 5; the testings indicate that the clayey silt deposit is inorganic and of low plasticity. In general, the testing shows that the natural moisture content is slightly above or below the liquid limit.

Standard Penetration testing gave 'N' values ranging from 13 to 24 blows per foot. Based on these 'N' values,and together with the in situ vane testing, the deposit is estimated to have a stiff to very stiff consistency.

Glacial Till

A deposit of glacial till up to 18 feet thick was encountered in all borings except in the area of the south bank and also in areas where the parent subsoil has been completely dredged. The composition of the glacial till varies widely across the site. Beneath Zwick's Park and adjacent to it, the till is cohesive being a heterogeneous mixture of clayey silt, with sand and gravel. Elsewhere, the till deposit is granular and composed of a heterogeneous mixture of sand, gravel with some silt and clay. In some locations the glacial drift was found to contain occasional cobbles and boulders in the lower portion of the deposit. The results of laboratory testing on representative samples from this deposit are shown on the Plasiticity Chart, Figure 6 and on the Grain Size Distribution Envelope, Fig.7. The Atterberg Limits indicate that the cohesive glacial drift has an inorganic matrix of low plasticity.

The range of 'N' values from the Standard Penetration testings in this deposit is 15 blows per foot to 80 blows for 3 inches. The cohesive glacial till is estimated to have a firm to hard consistency based on 'N' values. Similarly, the relative density of the granular till is estimated to have a compact to very dense relative density that in general increases with depth.

Groundwater Conditions

Observations on the groundwater level were carried out during the fieldwork by measuring in the open boreholes. The measurements place the groundwater table at a depth of 3 feet below the existing ground surface which corresponds to elevation 246. During the time of the field investigation the water level in the bay fluctuated only slightly from elevation 245.6 to elevation 245.8.

DISCUSSION AND RECOMMENDATIONS

The Ministry of Transportation and Communications has proposed to replace the existing crossing of Hwy. 14 and the Bay of Quinte with a new two-lane high level structure about 100 feet east of and parallel to the existing crossing.

A feasibility study was initiated to assess the foundation requirements for the high level bridge and related approaches by carrying out a preliminary sub-surface investigation. The high level structure will be required to have a minimum vertical navigational clearance of 90 feet and a minimum depth of channel of 13 feet. The main navigation channel will be shifted toward the centre of the bay. The alignment of the north approach will meet the existing conditions some 1000 feet north of Zwick's Park shoreline, whereas about 500 feet south of the Rossmore shoreline, the alignment of the south approach will match existing. The existing structures are to be removed, however, the extent of the causeway removal is yet to be reconciled. The grades of the proposed structure are restricted to a maximum of 5%.

The number of spans and span details are as yet to be decided. Furthermore, the locations of the piers and abutments will depend to a large degree upon the extent of the approach fills which are in turn affected by the feasibility and economics of constructing and maintaining the fill slopes. Because of the importance of the approach fills at the feasibility stage in the planning process, this aspect will be discussed first.

Approach Embankments

North approach - Based on the preliminary profile grade as established by the Regional Planning and Design Office, the heights of profile grade above the following existing conditions at the north approach are anticipated.

Height of Profile Grade Above Average Ground Surface	up to 55 feet
Height of Profile Grade Above Bay Bottom	60 - 105 feet

Because of the generally flat terrain, fill heights will not vary significantly in the transverse direction. Longitudinally, the fill height will depend upon the profile grade at that location and upon the sloping nature of the bay bottom. However, the slope of the bay bottom at the north approach is relatively insignificant, being about 2%.

Subsoil at the north approach generally consists of up to 15 feet of fill material overlying 10 to 13 feet of firm to stiff clay or hard clayey silt which in turn overlies 2 to 4 feet of hard glacial till, followed by limestone bedrock. In one location (B.H. #5), a 7 ft. thick pocket of soft organic clay is sandwiched between the lower clayey silt stratum and the 7 ft. overlying fill material. In another location (B.H. #8) 11 feet of dense sand to sandy gravel is encountered between the lower clay stratum and a 4 foot thick deposit of fill material. Subsoil beneath the bay bottom is somewhat less competent, being a surficial veneer of very soft organic clay up to 9 feet thick overlying 10 to 13 feet of compact to dense sandy gravel which in turn overlies 10 feet of firm to stiff clay. The clay in turn is overlying a compact to very dense glacial till.

The fill material is heterogeneous in composition. In some zones it is composed of competent granular fill material but in other areas it is composed of sanitary landfill or topsoil. This fill material will be detrimental to the stability and performance of the approaches and it is, therefore, recommended that the fill material and organic clay be removed entirely within the plan limits of the proposed embankment. Backfill placed underwater should be composed of Granular 'A' to prevent segregation of material. Fill material for the remainder of the approaches should be of acceptable granular material placed and compacted according to current MTC standards.

Stability analysis in terms of total stress have been carried out to determine the stability of fills immediately after construction. In this method of analysis, stability is governed by undrained shear strength properties of the foundation and fill materials. The following data and values were used in carrying out the stability analysis.

Fill Material	γ (pcf)	ϕ^o	Su (psf)
(Tension Cracks 5')			
Granular Material	130	30	0

The subsoil condition beneath the bay and beneath the land are somewhat different, subsoil beneath the land being slightly more competent. For this reason two sets of subsoil data were considered in the analysis. The subsoil conditions also assume that the unacceptable fill material and organic clay will be removed entirely within the plan limits of the embankment and replaced by a granular type of acceptable fill material.

Subsoil Conditions Beneath the Bay (Water Elevation 246)

Elevation (Feet)	γ(PCF)	γ'(PCF)	ϕ^o	Su (PSF)
240-225	130	68	30^o	0
225-220	100	38	0	1000
220-215	100	38	0	750
Below 215	140	78	35	0

Subsoil Conditions Beneath Land

Elevation (Feet)	γ(PCF)	γ'(PCF)	ϕ^o	Su (PSF)
250-235	130	68	30	0
235-225	100	38	0	1500
Below 225	140	78	35	0

The longitudinal stability of the embankments will depend upon the geometry of the forward slope of the embankment, as well as the position and location of the structure's abutment. Furthermore, the longitudinal stability will depend on the location of the toe of the slope since subsoil is not as competent beneath the bay bottom as beneath the land.

The following are recommendations based on the above analysis. They are discussed according to three categories, depending upon the location of the toe of the slope with regard to the shoreline.

Case A: The toe of the slope will not extend within 20 feet of the shoreline.

- Fills up to 40 feet will be stable with forward and side slopes of 2:1.
- Fills up to 50 feet with 20 foot long counterbalancing berms at mid-height on both the forward and side slopes of the embankment would also be stable with slopes of 2:1.
- Fills up to 60 feet with 40 foot long counterbalancing berms at mid-height on both the forward and side slopes of the embankment would be stable with 2:1 slopes.

These recommendations are summarized in Figure 9, together with the critical slip circle and assumed subsoil conditions for 40 foot and 50 foot fill heights.

Case B: The toe of the slope is located within 20 feet of the shoreline but will not extend into the bay.

- Fills up to 30 feet above the average ground surface (assumed to be at elevation 250±) will be stable with forward and side slopes of 2:1.
- Fills up to 45 feet above the average ground surface will require 35 foot long berms at mid-height in both forward and transverse direction of the approaches.

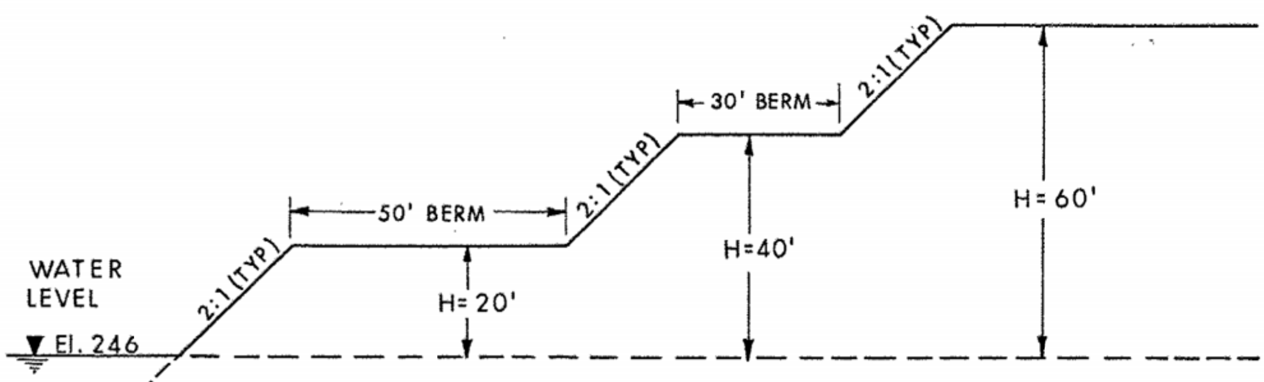
- Fills up to 55 feet above the average ground surface will be stable with 60 foot long counterbalancing berms at mid-height on both the forward and side slopes of the embankment with slopes not steeper than 2:1.

These recommendations are shown on Figure 10, together with the critical slip circles and the assumed subsoil stratigraphy.

Case C: The toe of the slope extends past the shoreline into the bay.

- Fill heights up to 20 feet above the water line will be stable with side slopes of 2:1.
- Fills up to 40 feet above the waterline with 50 foot long counterbalancing berms at midheight on both the forward and side slopes of the embankment would be stable with 2:1 slopes.
- Fill heights of up to 60 feet above the water line would require the following berm configuration for stability against deep seated rotational failure (also see sketch below):

berm at 1/3 height 50 feet long
berm at 2/3 height 30 feet long
all side slopes 2:1 maximum



Case C

A minimum of 50 foot transitition taper should be provided between the different geometrical configuration, i.e. between a Case C and Case A or between a Case A and Case B condition.

Due to the presence of the underlying compressible clay stratum, fill will undergo settlements as a result of the consolidation of the clay deposit. To estimate settlements the stress distribution was computed by the Purdue Method and consolidation characteristics of the clay deposit were based on three laboratory consolidations tests. An estimate of the field e-log p curve was made from the laboratory curve by means of a graphical procedure after Schmertmann, 1953. The calculations indicate that a 40 foot fill with side slopes of 2:1 and no berms will undergo a settlement of approximately 5-6 inches; furthermore, it is estimated that 90% of the settlement will occur within 4 months after construction. Calculations were also carried out for a 60 foot fill with 40 feet mid-height berms and slopes of 2:1. The expected settlement for this fill is 7-9 inches, 90% of which would occur within 4 months after completion of the fill. The above magnitudes of settlement are applicable to fill heights located on the land. The consolidation testing indicated that the clay stratum beneath the bay would undergo about 2-3 times the settlement of clay stratum beneath land subject to the same loading conditions.

If settlements of such magnitudes are detrimental to the performance of the approaches and the pavement, the fills should be constructed and left in place for 4-6 months prior to paving. It is calculated that this preloading period would allow about 90% of the settlement to occur.

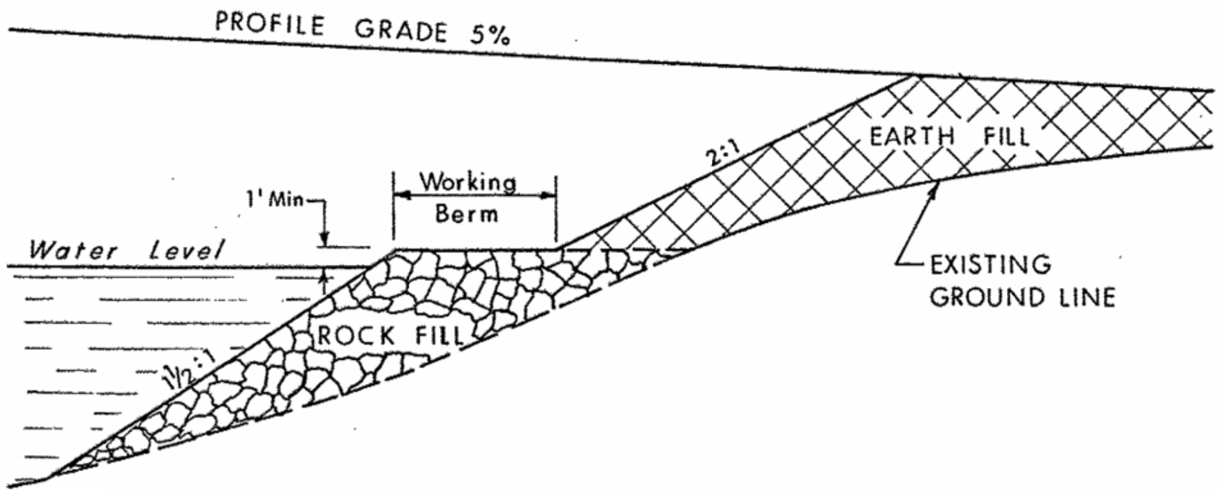
<u>South approach</u>: Based on the preliminary profile grade the height of fills above the following existing conditions at the south approach are anticipated.

Height of Profile Grade Above Average Ground Surface up to 15 feet
Height of Profile Grade Above Bay Bottom at Pier Stop up to 62 feet

Again, generally flat terrain is prevalent at the south approach and the fill heights will not vary appreciably in the transverse directions. However, due to the sloping nature of the bay bottom and the 5% profile grade, fill height will vary considerably in the longitudinal direction, increasing as the approaches extend outward from shore.

Subsoil at the south approach is comprised of 2 to 11 feet of loose to compact sand to sandy gravel overlying limestone bedrock.

The subsoil is such that the anticipated fill heights will be stable with respect to deep seated rotational failure with forward and side slopes of 2 horizontal to 1 vertical. If the forward slopes extend into the bay the embankment should be constructed by placing rockfill to extend to a height of one foot above the water level with side and forward slopes of 1½ to 1. The fill may then be completed by the placing of earthfill with a slope of 2 to 1. It may be advantageous for construction to provide a 10 foot berm between the crest of the rockfill slope and the toe of the earthfill slope. See sketch below.



Furthermore, the earthfill should be protected against wave action by rip-rapping to an elevation as per hydrological requirements.

Structure Foundations

As mentioned earlier, details of the spans and pier locations are as yet not finalized. However, one proposal put forward is that the structure be comprised of 12 spans of equal length (250'), the centre span to cross the relocated main channel where the present 2 fixed-span structure is located. This centre span is required to have a minimum vertical navigational clearance of 90 feet above the water level. Highway grades are to be limited to a maximum of 5%.

At this stage in the design it is felt that detailed recommendations concerning the construction and design of the structure foundations is not warranted. Only concepts or alternatives will be presented at this stage and further elaboration or clarification will be provided by this office as required.

Because of the widely differing subsoil conditions, foundation requirements for the southern and northern portion will be discussed separately.

Southern portion: On the southern portion subsoil consists of up to 46 feet thick deposits of very soft organic clay underlain by up to 5 feet of very dense glacial till overlying bedrock. Within the main channel bedrock is exposed at the bay bottom at a depth of about 35 feet.

Within the southern portion of the proposed line the subsoil conditions are such that virtually no lateral support will be provided to the structure foundations. The structure foundations must provide the sufficient and adequate internal lateral rigidity to be considered in this particular area. The following alternatives are put forward for consideration.

The structure may be supported on large diameter concrete caissons socketted into the bedrock surface. Construction would require that caissons be provided with a permanent liner. The bedrock conditions are such that foundations may be designed for a maximum allowable load of 30 t.s.f. The sizing of the caissons would be based on the slenderness ratio. To reduce the dewatering problems it may be advantageous to extend the caissons to the underside of the deck and in this manner construct the pile caps to serve also as the pier caps.

Alternatively, the foundation may be accomplished by constructing cofferdams and supporting the structure foundation directly on the bedrock surface. In this manner the structure foundation would have to be brought up by means of mass or reinforced concrete. For this scheme proposed, an extensive dewatering scheme would be essential for construction purposes.

Alternatively, the structure may be supported on steel tubular piles keyed into the bedrock surface to provide sufficient lateral resistance. Tubular piles should be sized according to the slenderness ratio.

Northern portion: Subsoil conditions on the northern portion are somewhat more competent. Subsoil consists of a thin veneer of very soft organic clay up to 9 feet thick overlying 11 to 13 feet of sand or sandy gravel followed by 9 to 13 feet of clay which in turn overlies 5 to 18 feet of glacial till. The glacial till overlies limestone bedrock. The following alternatives are provided for consideration.

The structure foundation may be founded on steel 'H' piles, steel tube piles, or concrete caissons founded on the bedrock surface. Piles constructed in this fashion may be designed for maximum allowable load, i.e. 100 tons/pile for a 12 BP 74 steel 'H' pile. Again, concrete caissons may be used but for construction purposes it will be necessary to use a permanent liner.

General considerations: The bedrock depths are quite variable across the stratigraphical profile of the proposed line, being from 33 to 63 feet below the water level. At this stage in the feasibility planning where the type of structure foundation has yet to be decided, there does not appear to be any advantage in the saving of pile lengths, etc., in shifting the alignment slightly to the east or west. However, depending on the type of foundation chosen for the southern portion, it may be advantageous to shift the proposed line to avoid the causeway fill, thus simplifying the construction of the cofferdams. Conversely, if piles are chosen it may be advantageous to shift the alignment toward the causeway fill to take advantage of the lateral support offered to the piles by the causeway fill.

MISCELLANEOUS

The fieldwork was supervised by Mr. M. MacLean, Project Engineer, and Mr. J. White, Student Engineer, using equipment owned and operated by Atcost Soil Drilling Inc., Concord, Ontario.

This report was written by Mr. M. MacLean with the assistance of Miss Y. Jamani, Student Engineer, and was reviewed by Mr. M. Devata, Supervising Engineer.

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