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PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

CASABLANCA BOULEVARD BRIDGE, QEW INTERCHANGE STRUCTURE 18X-0196/B0

GRIMSBY, ONTARIO

CONTRACT NO. 2022-E-0020_0021 WO#1

GEOCRES NO.: 30M04-129

Location: Lat: 43.207082°, Long: -79.595377°

Client Name: Ontario Ministry of Transportation c/o HDR Inc.
Date: May 4, 2026

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PART A. FOUNDATIONS INVESTIGATION REPORT

1. INTRODUCTION

This section of the report presents the factual findings obtained from a foundation investigation conducted by Thurber Engineering Ltd. (Thurber) to support the replacement of the Casablanca Boulevard Underpass (Structure ID 18X-0196/B0) crossing over QEW Highway in Grimsby, Ontario. Thurber was retained by HDR Inc. on behalf of the Ministry of Transportation Ontario (MTO) to carry out this foundation investigation under Assignment Number 2022-E-0020_21 WO#1.

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

This report addresses the replacement of the existing structure, based on the results of Thurber's 2025 investigation, discussed herein, as well as the results from an earlier investigation available from the online Geocres Library, as follows:

- MTO GEOCRES No. 30M04-039: Report titled "Foundation Investigation Report for Proposed Underpass of Q.E.W. at Ofield Road, Twp. Of North Grimsby, District #4 (Hamilton), W.J. 66-F-54, W.P. 224-63", prepared by Department of Highways Ontario, dated July 1966.

It is a condition of this report that Thurber's performance of its professional services is subject to the attached Statement of Use and Interpretation.

2. SITE AND PROJECT DESCRIPTION

2.1 General

The Casablanca Boulevard Underpass is oriented in a north-south direction crossing over the QEW highway which is generally orientated in the west-east direction. The new interchange and underpass structure is proposed to accommodate future widening of the QEW as well as increased traffic volumes. The structure is currently understood to be in good condition but would need to be replaced to meet design requirements of the new interchange.

The properties surrounding the project area is mainly in commercial use with minor residential accommodations.

The general topography of this region consists of relatively flat-laying ground sloping gradually towards Lake Ontario. In general, the natural ground surface in the immediate vicinity of the structure between Elev. 83.8 m and 86.2 m.

The existing underpass has been constructed on embankment fill, with embankment heights of up to 7.3 m approaching the underpass structure. The abutment foreslopes and embankment side slopes are oriented at approximately 2 horizontal to 1 vertical (2H:1V) and did not show signs of instabilities at the time of the field investigation.

Photographs of the project area showing the condition of the site at the time of the field investigation are included in Appendix A.

The proposed replacement structure is a two span steel girder structure with a width of about 38.6 m and span lengths of 41 m, and a skew angle of about 22 degrees, as shown in the preliminary General Arrangement (GA) drawings presented in Appendix H.

2.2 Site Geology

The site is located primarily within the Iroquois Plain Physiographic region, as delineated in *The Physiography of Southern Ontario*[1] (Chapman and Putnam, 1984).

Based on *Surficial Geology of Southern Ontario*[2], the surficial deposits at the site consist of glaciolacustrine fine-textured clay and silt till deposits. South of the project site the surficial composition is Paleozoic bedrock.

According to *Paleozoic Geology of Southern Ontario*[3], the bedrock geology consists of red shale interbedded with calcareous layers (siltstone, sandstone, and limestone) of the Queenston Formation.

3. INVESTIGATION PROCEDURES

3.1 Borehole Drilling

Thurber's field investigation, which comprised of advancing a total of two boreholes, was carried out from October 19 to 21, 2025. A summary of the borehole program is provided in Table 3-1

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[1] Chapman, L.J. and Putnam, D.F., 1984: The Physiography of Southern Ontario, Ontario Geological Survey Special Volume 2, Third Edition. Accompanied by Map P.2715, Scale 1:600,000.

[2] Ontario Geological Survey, 2010: Surficial geology of Southern Ontario; Ontario Geological Survey, Miscellaneous Release--Data 128-REV

[3] Armstrong, D.K. and Dodge, J.E.P., 2007: Paleozoic geology of southern Ontario; Ontario Geological Survey, Miscellaneous Release--Data 219.

below. The approximate locations of the boreholes are shown on Drawing 73956-1 provided in Appendix C. Borehole details are provided in the Record of Borehole sheets included in Appendix D.

Table 3-1: Borehole Details

Borehole No.	MTM NAD 83 Northing (m)	MTM NAD 83 Easting (m)	Ground Surface Elev. (m)	Borehole Depth (m)	Borehole Termination Elev. (m)	Monitoring Well Installed
25-01	4785175.4	297069.1	91.5	12.5	79.0	Yes
25-02	4785309.1	297036.4	91.6	15.2	76.4	Yes

The borehole locations were established in the field by Thurber using a portable GNSS receiver and verified relative to existing site features. The ground surface elevations at the borehole locations were determined using a Trimble R12 GNSS receiver. The ground surface elevations were derived from GNSS observations and Natural Resources Canada Geoid Model HT2.0, which is a height transformation model that modifies a gravimetric geoid model (CGG2000) to fit the historical CGVD28:78 datum. Borehole location coordinates are presented in the Modified Transverse Mercator (MTM) coordinate system (MTM NAD 83 Zone 10).

The borehole locations were cleared of utilities prior to commencement of drilling.

Boreholes were advanced using hollow stem augers and HQ rock coring advancement methodologies powered by a truck mounted D50 Turbo drill rig supplied and operated by Elements GEO Corp.

Soil samples were obtained at selected intervals using 50 mm outside diameter split-spoon samplers driven in conjunction with the Standard Penetration Test (SPT) ASTM D1586.

Rock coring was carried out using HQ core barrel in conjunction with HW casing. All rock cores were logged, and the Total Core Recovery (TCR), Solid Core Recovery (SCR), Rock Quality Designation (RQD), and Fracture Index (FI) were determined for each core. The cores were then photographed, wrapped with parafilm wrap, and packaged in wooden core boxes.

The field investigation was carried out under the full-time supervision of Thurber's technical staff. All boreholes were logged in the field. Soil and rock samples were identified in the field, placed in labelled containers and transported back to Thurber's laboratories in Oakville for further examination and testing.

3.2 Groundwater Monitoring

The groundwater conditions were assessed during drilling by visual examination of the soil, the sampler, and the drill rods as the samples were retrieved.

Monitoring wells were installed in both boreholes to permit monitoring of the groundwater levels at the site after completion of drilling operations. The monitoring wells consisted of 50 mm diameter PVC pipes with slotted screens sealed at target depths and with installation details as summarized in Table 3-2.

Table 3-2: Monitoring Well Details

Borehole ID	Ground Surface Elevation (m)	Monitoring Well Tip		Slotted Screen Length (m)	Mid-Screen Elevation (m)
		Depth (m)	Elev. (m)		
25-01	91.5	12.5	79.0	1.5	79.8
25-02	91.6	9.1	82.5	1.5	83.3

Water levels were measured in the monitoring wells using a groundwater level reader.

The monitoring wells remain installed for long term groundwater level monitoring during subsequent stages of design. Well records were submitted to Ministry of the Environment, Conservation and Parks (MECP) and the wells were tagged in accordance with Ontario Regulation 903 (Reg. 903). The monitoring wells should be decommissioned by a licensed well contractor when no longer required.

3.3 Laboratory Testing

Visual identification and natural moisture content determination were completed on all retained soil samples. Laboratory testing was selected in general accordance with the MTO Guideline for Foundation Engineering Services (GFES), Section 5 (April 2022). Routine geotechnical laboratory testing consisted of grain size distribution testing and Atterberg Limits testing, where appropriate, in accordance with MTO and ASTM standards. Geotechnical laboratory testing was carried out at Thurber's laboratory in Oakville.

Point Load Testing and Uniaxial Compressive Strength (UCS) tests were carried out on selected rock cores for estimating the strength of the intact rock. Point load strength testing was carried out in Thurber's laboratory. UCS testing was carried out by Geomechanica Inc.

Selected soil samples were also submitted for analytical testing to assess the corrosion potential of soil to metal and the potential for sulphate attack on the subsurface concrete structures. The analysis was carried out by SGS Canada Inc., an independent Canadian Association for Laboratory Accreditation (CALA) accredited laboratory. The results of the analytical testing are discussed in Section 5 and laboratory Certificate of Analysis are included in Appendix E.

Results of the geotechnical soil and rock laboratory testing are presented on the Record of Borehole sheets in Appendix D and in detail in Appendices E and F, respectively.

4. DESCRIPTION OF SUBSURFACE CONDITIONS

Details of the encountered soil stratigraphy are presented on the Record of Borehole sheets included in Appendix D. Photographs of the retrieved bedrock core samples are provided in Appendix G. A general summary of the stratigraphy, based on the conditions encountered in the boreholes, is given in the following paragraphs. However, the factual data presented on the Record of Borehole sheets takes precedence over this general description and must be used for interpretation of the site conditions. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological changes. The subsoil conditions will vary between and beyond the borehole locations. In summary, the subsurface conditions encountered at the site consist of embankment fill overlying a deposit of firm to hard silty clay which is underlain by shale bedrock.

A more detailed description of the subsurface conditions encountered in the boreholes are presented below.

4.1 Asphalt

Approximately 225 mm and 760 mm of asphalt were encountered at the surface in Boreholes 25-01 and 25-02 which were both advanced through the existing Casablanca Boulevard pavement in the southeast and northwest quadrants of the structure site, respectively.

4.2 Fill

Approximately 6.8 m and 7.5 m of fills associated with the approach embankments were encountered below the asphalt in Boreholes 25-01 and 25-02, respectively. The fill layers extend to about 7.0 m and 8.3 m (Elevations 84.5 and 83.3 m) behind the south and north abutments. The upper portion of the fill consists of silty sand to sand and gravel, and the lower portion of the fill is generally silty clay.

4.2.1 Non-Cohesive Fill

Deposits of cohesionless fill were encountered in both boreholes directly below the asphalt. The Cohesionless fill was generally described as brown/grey, compact to very dense, and varied in composition from silty sand to sand and gravel. The cohesionless fill was encountered at depths of 0.2 and 0.8 m (Elev. 90.9 and 91.3 m) and was penetrated at 2.2 and 2.7 m (Elev. 88.9 and 89.3 m).

SPT N-values recorded in the cohesionless fill ranged from 13 to 73 blows, indicating compact to very dense materials. Moisture contents recorded in the cohesionless fill ranged from 4% to 9%.

The results of the grain size distribution analyses carried out on selected samples of the cohesionless fill are presented on Figure 1 in Appendix E and their respective borehole log in Appendix D. The results of the grain size analyses are summarized in the table below.

Soil Particle	Percentage (%)
Gravel	25 to 42
Sand	36 to 40
Silt and Clay	22 to 35

4.2.2 Cohesive (Silty Clay) Fill

Cohesive fills were encountered in both boreholes below the cohesionless fill. The cohesive fill was generally described as firm to very stiff, grey/brown, and composition of clay, silty, some sand to sandy, and trace gravel. The cohesive fill was encountered at depths of 2.2 and 2.7 m (Elev. 88.9 and 89.3 m) and extended to a depth of 7.0 and 8.3 m (Elev. 84.5 m and 83.3 m) in Boreholes 25-01 and 25-02, respectively.

SPT N-values recorded in the cohesive fill ranged from 4 to 17 blows. Moisture contents recorded in the clay fill ranged from 12% to 24%.

The results of the grain size distribution analyses carried out on selected samples of the cohesive fill are presented on Figure 2 in Appendix E and their respective borehole log in Appendix D. The results of the grain size analyses are summarized in the table below.

Soil Particle	Percentage (%)
Gravel	1 to 5
Sand	14 to 24
Silt	39 to 45
Clay Size Particles	31 to 40

The results of Atterberg Limits testing carried out on selected fill samples indicate that the samples are primarily low plasticity silty clay (CL). The results of the testing are presented in Figure 4 in Appendix E and are summarized in the table below.

Index Property	Percentage (%)
Liquid Limit	24 to 35
Plastic Limit	14 to 21
Plasticity Index	10 to 14

4.3 Silty Clay

A native layer of silty clay was encountered below the cohesive fill in both boreholes. The silty clay was encountered at depths of 7.0 and 8.3 m (Elev. 84.5 m and 83.3 m) and extended to depths of 8.1 and 9.3 m and (Elev. 83.4 m and 82.3 m), respectively.

SPT N-values recorded in the native silty clay typically ranged from 8 to 28 blows, indicating a stiff to very stiff consistency. However, N-values greater than 100 blows per 100 mm of penetration were encountered near the base of the layer. Moisture contents recorded in the silty clay ranged from 10% to 26%.

The results of the grain size distribution analyses carried out on selected samples of the silty clay are presented on Figure 3 in Appendix E and their respective borehole log in Appendix D. The results of the grain size analyses are summarized in the table below.

Soil Particle	Percentage (%)
Gravel	0 to 1
Sand	14 to 16
Silt	51 to 56
Clay Size Particles	30 to 32

The results of Atterberg Limits testing carried out on a selected silty clay sample indicate that the sample is low plasticity silty clay (CL). The results of the testing are presented in Figure 5 in Appendix E and are summarized in the table below.

Index Property	Percentage (%)
Liquid Limit	28
Plastic Limit	18
Plasticity Index	10

4.4 Shale Bedrock

Shale bedrock was encountered below the overburden at depth of 8.1 m and 9.3 m (Elev. 83.4 m and 82.3 m) in Boreholes 25-01 and 25-02, respectively.

Photographs of the retrieved rock core are provided in Appendix G. The bedrock was visually identified as red shale of the Queenston Formation.

4.4.1 Physical Properties

4.4.1.1 Total Core Recovery

Total Core Recovery (TCR) is the total cumulative length of all core recovered in the core barrel expressed as a percentage of the length of the core run and is recorded on a per run basis. Prior to measuring the recovered length, the core was assembled to align joints and rubble zones were reassembled to the extent practicable. The TCR of the rock cores typically ranged from 85% to 100%. During the investigation, Borehole 25-02 Run 4 had no recovery as the bedrock core was lost during retrieval.

4.4.1.2 Solid Core Recovery

Solid Core Recovery (SCR) is the total cumulative length of all solid, cylindrical pieces of core recovered in the core barrel expressed as a percentage of the length of the core run and is recorded on a per run basis. The SCR of the rock cores recovered typically ranged from 40% to 96%.

4.4.1.3 Rock Quality Designation

Rock Quality Designation (RQD) is the total cumulative length of intact core recovered in the core barrel expressed as a percentage of the length of the core run and is recorded on a per run basis. The RQD of the rock cores typically ranged from 60% to 91%, indicating a fair to excellent quality rock. The RQD of the first run (Run #1) in Borehole 25-02 was zero due to poor rock quality.

4.4.1.4 Harder Interbeds

Interbeds of siltstone and limestone “hard layers” were noted throughout the Queenston Bay Formation. It is noted that petrography was not completed to determine the mineral content and/or crystalline structure of each hard layer. The measured thickness of the hard layers varied in thickness but was generally less than 150 mm.

4.4.1.5 Fracture Index

The fracture index records the number of natural fractures per 0.3 m length of core run. The fracture index of the shale bedrock typically ranged from 0 to 7, with localized zones of greater than 10 in more weathered and/or highly fractured zones.

4.4.1.6 Weathering

In general, the shale bedrock is highly to moderately weathered in the upper 2 m of the bedrock, generally improving with depth to slightly weathered to fresh.

4.4.2 Index Properties

4.4.2.1 Point Load Testing

Point load index strength tests were carried out on selected intact rock core samples. The test results are presented in Appendix F. Determination of the unconfined compressive strength (UCS) was based on the empirical relationship between UCS and point load index strength as follows:

$$\text{Unconfined compressive strength (MPa)} = 24\ I_{S(50)}$$

where $I_{S(50)}$ is the point load index strength in MPa for a 50 mm equivalent diameter core.

A correlation value of 24 was used to estimate the UCS, in accordance with Franklin (1983)[4], as insufficient site specific data is available to develop a value by comparison of the UCS tests and the point load index tests.

The UCS of the shale estimated from the results of point load tests ranged from 5 to 20 MPa, indicating a very weak to weak rock strength classification. Locally, a point load test indicated an estimated UCS value of 25 MPa, indicating a medium strong rock classification. The results are summarized on the Record of Borehole sheets included in Appendix D and on the Point Load Test Sheets in Appendix F.

4.4.2.2 Unconfined Compressive Strength

UCS testing was completed on two shale samples. The results of the UCS testing, including bulk density, are summarized in Table 4-1 and the complete test results are provided in Appendix F.

Table 4-1: Shale Bedrock UCS Test Results

BH No.	Run No.	Sample Depth (m)	Bulk Density (g/cm^3)	UCS (MPa)
25-01	2	9.6 – 9.9	2.585	36.3
25-02	2	11.3 – 11.6	2.611	6.7

4.5 Groundwater Conditions

Monitoring wells were installed in Boreholes 25-01 and 25-02 to monitor groundwater levels. The groundwater levels measured in the monitoring wells are shown on the appended borehole logs and are summarized in Table 4.2 below.

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[4] J. A. Franklin, 1983: Evaluation of Shales for Construction Projects - An Ontario Shale Rating System, Ministry of Transportation and Communications

Table 4-2: Groundwater Level Monitoring

Borehole/ Monitoring Well No.*	Screened Deposit	Mid Screen Depth (m)	Mid Screen Elev. (m)	Date	Groundwater Level	
					Depth m)	Elevation (m)
25-01	Shale	11.7	79.8	Nov. 24, 2025	5.9	85.6
25-02	Silty Clay	8.3	83.3	Nov. 24, 2025	7.1	84.5

The groundwater levels measured are short-term, not stabilized readings and may not reflect groundwater levels at the time of construction as seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater levels may be at a higher elevation after periods of significant and/or prolonged precipitation and spring snow melts.

5. CORROSIVITY

Selected soil samples were submitted for analytical testing of corrosivity parameters including sulphate content. The corrosivity testing was carried out by SGS Canada Inc. of Lakefield, Ontario, a CALA accredited analytical laboratory. The results of the analytical tests are shown in the table below and the laboratory certificates of analysis are presented in Appendix E.

Table 5-1: Analytical Corrosivity Test Results

Sample ID	Depth (m)	Soil Sample	Sulphide (%)	Chloride (µg/g)	Sulphate (µg/g)	pH	Resistivity (ohm.cm)	Redox Potential (mV)	Electrical Conductivity (µS/cm)
BH25-01 RUN#1	8.6	Shale	< 0.01	230	20	7.98	2950	191	339
BH25-02	8.4-9.0	Silty Clay	0.02	1500	390	7.81	433	53	2310

6. MISCELLANEOUS

The as-drilled borehole locations and ground surface elevations were measured by Thurber following completion of the field program. Elements GEO Corp. of Hamilton, Ontario supplied and operated the drill rigs used to drill, test, and sample the boreholes and install the monitoring wells.

The field investigation was supervised on a full-time basis by Mr. Bryan Ho. Overall supervision of the field investigation program was provided by Ms. Madisan Chiarotto, P.Eng. and Mr. Mehdi Mostakhdemi, P.Eng.

Routine geotechnical laboratory testing was completed by Thurber's laboratory in Oakville. Advanced soil and rock laboratory testing was carried out by SGS Canada Inc. and Geomechanica Inc., respectively.

Interpretation of the field data and preparation of the report was performed by Ms. Madisan Chiarotto, P.Eng. The report was reviewed by Mr. Mehdi Mostakhdemi, a MTO Designated Principal Contact.

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PART B. FOUNDATION DESIGN REPORT

7. GENERAL

This section of the report presents interpretation of the factual data obtained during the field and laboratory investigation and presents preliminary foundation recommendations to support the preliminary structural design for the future interchange reconstruction and bridge replacement of the Casablanca Boulevard Bridge (Structure ID 18X-0196/B0) in Grimsby, Ontario. The General Arrangement of the proposed structure as of February 2026 is shown in Appendix H.

The comments and recommendations presented in this report are based on the subsurface soil and groundwater conditions encountered during the preliminary investigation. Additional investigation will be required at the detailed design stage. The soil conditions may vary between and beyond the borehole locations, and accordingly geotechnical inspection during construction is important to assess any variation of subsurface conditions and to provide additional recommendations if necessitated by such variations.

The interpretation and recommendations are intended for the use of the design consultant and MTO and shall not be relied upon by any other parties including the construction or design-build contractor or used for any purposes other than development of the preliminary project design. Comments on construction methodology and equipment, where presented, are provided only to highlight those aspects that could affect the design of the project. Contractors must make their own assessment of the factual information presented in previous sections (Part 1) of the report, and the implications on equipment selection, construction methodology, and scheduling and the like.

The report references the Canadian Highway Bridge Design Code (CHBDC 2025). In accordance with Tables 6.1 and 6.2 of the CHBDC 2025, a consequence classification of “typical consequence” and a degree of site and prediction model understanding of “typical understanding” have been assumed. It is understood that the structure is designated as a “Major Route” importance category.

8. BRIDGE FOUNDATION DESIGN

The existing structure consists of a four-span underpass, with the existing abutments and piers supported on spread footings. Based on the *General Arrangement* (GA) drawings for the existing structures, dated September 1967, the existing footing details are summarized below:

Table 8-1: Summary of Existing Bridge Foundations

Foundation Element	Relevant Borehole ID	Foundation Type	Footing Dimensions (m)	Founding Elevation (m)	Founding Stratum
South Abutment	25-01, 4* and 5*	Strip Footing	3.0	84.8	Hard Silty Clay over Shale Bedrock
South Pier (Pier C)	6*	Square Footings	2.4	82.9	Shale Bedrock
Center Pier (Pier B)	N/A	Square Footings	2.4	82.9	Shale Bedrock**
North Pier (Pier A)	3*	Square Footings	2.4	82.9	Weathered Shale
North Abutment	25-02, 1* and 2*	Strip Footing	3.0	85.3	Perched on Compacted Gravel Pad over Shale Bedrock

* Boreholes from 1966 investigation
** Inferred from surrounding boreholes

The bridge abutments were rehabilitated (around 2013) to convert the abutments of the structure to semi-integral abutments.

The proposed replacement structure is a two span steel girder structure with a width of about 38.6 m and span lengths of 41 m, and a skew angle of about 22 degrees, as shown in the preliminary General Arrangement (GA) drawings presented in Appendix H.

8.1 Foundation Options

The subsurface conditions encountered at the abutments consisted of embankment fill overlying a deposit of firm to hard silty clay which is underlain by shale bedrock. Groundwater levels measured in monitoring wells installed in the boreholes varied from Elev. 84.5 m to 85.6 m.

Based on the subsurface conditions, both shallow and deep foundation options have been considered for the replacement of the underpass structure. A summary of the advantages and disadvantages associated with each option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages and risks is provided in Table 1 following the text of this report.

- **Spread Footings:** Shallow foundations (strip or spread footings) are feasible to support probable abutment widening, and construction of new piers provided that they are founded on undisturbed hard native silty clay deposit, shale bedrock or a compacted (perched) granular pad (similar to the existing foundations). Foundation drilling was not carried out at the piers, however it is estimated that it would require excavation to a depth of approximately 2.5 m to 3.5 m relative to the present ground surface at the pier locations. Shallow foundations would allow for the construction of semi-integral abutments, if the existing structure can be modified to be compatible with the widening. If widening of the existing structure is planned, there is potential for up to approximately 15 mm of differential settlement between the existing structure and the new widened portions if shallow foundations are adopted, although the majority of this settlement will occur during or immediately following construction.

- **Driven Steel H-Piles or Pipe Piles:** Driving steel piles to bedrock at the piers is not ideal for this structure since the pile length may be inadequate due to the locally shallow depth to rock. At the location of the piers, the depth to bedrock is relatively shallow (as little as 2.5 m to 3 m below the QEW grades, based on the results from the 1966 investigation). Therefore, driven steel piles are not considered to be a practical option for support of the piers of a widened or new underpass. In view of this concern, the use of driven steel H-piles is not recommended to support the piers, and this option has not been developed further.

  However, driven steel piles are feasible and suitable for support of the abutment widening, or for the abutments in a full structure replacement, particularly if the abutment pile caps can be perched within the (new) approach embankments. In the case of widening, differential settlement between the existing structures (which are founded on spread footings), and the widening would be negligible.
- **Caissons Socketed into Bedrock:** Drilled shafts (caissons) socketed into the bedrock are feasible for this site but would require the use of temporary (or permanent) liners while advanced through the overburden. Use of caissons for construction of new piers may be preferred over shallow foundations as they require less excavation (and roadway protection) than the shallow footings would require reaching the founding stratum.

Based on the above considerations, the preferred option from a geotechnical/foundations perspective to support the piers on shallow footings or caissons depending on the roadway protection and construction staging requirements. Use of either shallow foundations (founded on hard silty clay or shale bedrock) with a semi-integral abutment configuration or steel piles, with the pile caps perched on approach embankments (to provide sufficient pile length), driven to bedrock with integral abutments maybe considered for the design. The skew of the structure and abutment heights need to be reviewed if utilizing an integral abutment.

8.2 Shallow Foundations

For support of widened or new abutments, associated wing walls, retaining walls and the piers, strip or spread footings should be founded on the hard native silty clay deposit or shale bedrock. The existing fills and soft to stiff native layers are not considered suitable for support of the structure. If spread footings are used for widening, it is likely that they will be constructed to match the existing foundation elevation (as Provided in Table 8.1) and structurally connected, as such the potential for differential settlements should be considered. In the case of a full structure replacement, it may be feasible to found the footings slightly higher than the existing foundations, consistent with the elevations provided in the Table 8.1, further assessment of the impact of the existing foundations and their removal will be required at detail design, once the geometry and span of the widening and/or replacement structure is established.

Alternatively, subexcavation can be carried out to the elevations identified in Table 8.1, then backfilled with an engineered granular pad consisting of compacted Ontario Provincial Standard Specification (OPSS) 1010 Granular A fill prior to construction of the footings at a higher elevation. In this case, the founding elevation for the new footings should be a minimum of 1.2 m below the lowest surrounding grade to provide adequate protection against frost penetration in accordance with Provincial Standards. The compacted granular pad should extend at least 1 m beyond all sides of the new abutment footings, then outward and downward at 1H:1V, or flatter. The granular fill should be placed in accordance with Provincial Standards.

For preliminary design, strip or spread footings placed on the properly prepared, hard silty clay (or on compacted granular fill following subexcavation of the surficial soils), at or below the design elevations given in the preceding section, should be designed based on a factored geotechnical resistance at Ultimate Limit States (ULS) of 600 kPa, and a factored geotechnical reaction at Serviceability Limit States (SLS, for 25 mm of settlement) of 400 kPa. These values assume a footing width of between 3 m and 4 m.

The geotechnical resistances should be reviewed if the selected footing width or founding elevation differs from those given above. In addition, these preliminary geotechnical resistances are provided for loads applied perpendicular to the surface of the footings; where applicable, inclination of the load should be taken into account in accordance with Section 6.10.2 of the *Canadian Highway Bridge Design Code (CHBDC 2025)* and its *Commentary*.

The sliding resistance of a cast-in-place footing on bedrock or engineered fill may be computed using unfactored friction coefficient of 0.7 or 0.4, respectively.

The preliminary geotechnical resistance values provided above will have to be re-evaluated and modified as necessary during detail design, based on future additional subsurface investigation at the proposed abutment and pier widening or replacement locations.

8.3 Driven Steel Piles

Perched abutments may be supported on steel H-piles or steel pipe (tube) piles driven to shale bedrock. It is expected that driven piles will penetrate into the weathered shale and the estimated pile tip elevations for the driven steel piles are provided in the following table:

Table 8-2: Summary of Pile Tip Elevations

Foundation Element	Relevant Borehole ID	Estimated Pile Tip Elevation (m)
South Abutment	25-01, 4* and 5*	83.8 to 83.1
North Abutment	25-02, 1* and 2*	81.5 to 82.5

The pile caps should be constructed at a minimum depth of 1.2 m for frost protection purposes per Provincial Standards.

For HP 310x110 piles driven with their tip on shale bedrock, at elevations as provided in the table above, a factored axial geotechnical resistance at ULS of 2,000 kN and a factored axial geotechnical reaction at SLS (for approximately 10 mm of settlement) of 1,400 kN may be used for preliminary design.

Similar axial resistances may be used in the design of closed-end, concrete-filled, 324 mm (12 ¾ in.) diameter steel pipe piles having a minimum wall thickness of 9.5 mm (3/8 in.).

Pile installation should be in accordance with Provincial Standard OPSS.PROV 903, as amended by Special Provision 109F57. The preliminary geotechnical resistances provided above will have to be re-evaluated and modified as necessary during detail design in consideration of the additional subsurface investigation that will be carried out at the widened foundation elements.

8.4 Drilled Shafts (Caissons)

Caissons socketed into the bedrock may be considered to support the piers of a widening or replacement structure. In general, the upper (up to) 1.5 m of the shale bedrock at the site is highly weathered and shall not be relied on to provide an end bearing resistance or shaft (skin) friction. Caissons designed for end bearing only must be extended to slightly weathered to fresh (sound) bedrock to provide adequate support. The embedment depth of the caisson should be measured relative to the lowest elevation of sound bedrock where sloping bedrock conditions are encountered. Caissons founded in bedrock may be designed using a factored end bearing geotechnical resistance at ULS of about 5 MPa.

The above factored end bearing geotechnical resistance at ULS are based on the assumption that the base of the caissons will be fully cleaned of mud, soil and/or shattered rock fragments, and the design is to rely on the end bearing resistance of the caissons only. The penetration into the bedrock may have to be increased to satisfy lateral resistance requirements. The bottom of caissons shall be inspected using a waterproof downhole color camera (e.g., Drilled Shaft

Inspection Device or SID), or a quantitative measurement of base sediment (e.g., Shaft Quantitative Inspection Device or SQUID), or an approved alternate to verify base cleanliness. The base of all shafts shall be clear of any base sediment at the time of concreting to ensure direct contact between the concrete and the founding sound bedrock. Reinspection of the base may be required if concreting is not initiated within 6 hours of the base inspection.

For caissons extended to bedrock, the axial resistance of the caissons in compression at SLS will be governed by the elastic compression of the caisson’s structural element.

Caisson socketed into the bedrock to rely on side-wall (shaft) frictional resistance only may be designed using a factored geotechnical resistance at ULS of 300 kPa below the depth of sound bedrock. Rock socket drilling should be conducted after the casings (liners) are properly sealed and seated into the bedrock. In this case, following the completion of the excavation for each caisson (including excavation of the rock socket), each rock socket shall be roughened, cleaned and inspected to ensure that the length of socket into sound bedrock is not reduced and to ensure that the quality of the reinforced concrete is not impacted due to presence of sediments (i.e., sediments being mixed with the freshly poured concrete). As such it is recommended that the thickness of the sediment at the base at the time of concreting be less than 75 mm, if the end bearing resistance of the caissons is not relied on. In addition, the upper 1.5 m of the bedrock should not be relied on, to account for potential bedrock surface sloping and the fractured portion of the bedrock. Consideration should be given to extending the rock sockets deeper than the theoretical (design) lengths to account for potential impact of the socket cleaning on the quality of the concrete.

8.5 Frost Cover

The depth of frost penetration at this site is 1.2 m, as per OPSD 3090.101. The base of footings or pile caps must be provided with a minimum of 1.2m of earth cover or provided with an equivalent thickness of thermal insulation as protection against frost action. The earth cover should be measured perpendicular to the ground surface. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 600 mm of soil cover. Insulation must extend to a minimum of 1.2 m laterally from the edge of the footings and be placed above the groundwater table.

8.6 Downdrag

The subsurface stratigraphy below the existing fills at the site includes a thin layer of firm to hard silty clay over shale bedrock. Hence, downdrag forces on piles (or other deep foundations) are negligible.

9. SEISMIC CONSIDERATIONS

In accordance with the CHBDC, and in the absence of measurements shear wave velocities at the site, a site seismic class C may be used for preliminary design purposes. Once more details regarding the proposed works are available, consideration should be given to conducting measurements of the shear wave velocity such as Multichannel Analysis of Surface Wave (MASW) methods to support seismic design of the structure.

In accordance with the CHBDC, the selection of the seismic site class is based on the soil conditions encountered in the upper 30 m of the stratigraphic profile. In general, the subsurface conditions encountered at the site consist of embankment fill overlying a deposit of firm to hard silty clay which is underlain by shale bedrock.

As per Table 4.1, Clause 4.4.3.2 of the CHBDC (2019), the site may be classified as Seismic Site Class C.

Based on the National Building Code of Canada (NBCC 2020), the peak horizontal ground acceleration (PGA), corresponding to a design earthquake having a 2 percent probability of being exceeded in 50 years (i.e. 2,475-year return period) is 0.237 g at the site.

10. BACKFILL AND LATERAL EARTH PRESSURES

The backfill should be in accordance with OPSS.PROV 902 and placed to the extents shown in OPSD 3101.150 or OPSD 3121.150 where applicable. All backfill to the walls should consist of Granular A or Granular B Type II material meeting the requirements of OPSS.PROV 1010.

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

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

where: p_h = horizontal pressure on the wall at depth h (kPa)
K = earth pressure coefficient
γ = unit weight of retained soil
h = depth below top of fill where pressure is computed (m)
q = value of any surcharge (kPa).

The earth pressure coefficients are dependent on the material used as backfill. Table 10.1 lists the unfactored parameters recommended for design for vertical walls, assuming an essentially level ground surface behind and in front of the walls. The at-rest coefficients should be employed for restrained walls. Active pressures should be used for any wingwalls or unrestrained walls.

Table 10-1: Static Earth Pressure Parameters

Loading Condition	Earth Pressure Coefficient (K)	
	OPSS Granular A or Granular B Type II $\varphi = 35°$, $\gamma = 22.8$ kN/m^3	OPSS Granular B Type I $\varphi = 32°$, $\gamma = 21.2$ kN/m^3
Active, K_a	0.27	0.31
At-rest, K_o	0.43	0.47
Passive, K_p	3.7	3.3

The design of the abutments and retaining walls must incorporate measures such as subdrains (OPSD 3101.150) and/or weep holes to permit drainage of the backfill and avoid the potential build-up of hydrostatic pressures behind the walls.

Compaction should be carried out in accordance with OPSS.PROV 501. Heavy compaction equipment should not be used adjacent to the abutment and retaining walls.

10.1 Seismic Lateral Earth Pressures

In accordance with Clause 6.14-7 of the CHBDC, structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading. The coefficients of horizontal earth pressures for seismic loading are presented in the following table and may be used:

Table 10-2: Seismic Coefficient of Lateral Earth Pressures for Non-Yielding Retaining Walls

Loading Condition	Earth Pressure Coefficient (K)	
	OPSS Granular A or Granular B Type II $\varphi = 35°$, $\gamma = 22.8$ kN/m^3	OPSS Granular B Type I $\varphi = 32°$, $\gamma = 21.2$ kN/m^3
Active, K_{AE}	0.33	0.36
Passive, K_{PE}	4.51	4.38

11. APPROACH EMBANKMENTS

For preliminary design, approach embankments about 8 m high constructed with side slopes at 2H:1V or flatter using clean earth fill, Select Subgrade Material (SSM), Granular B, or Granular A are anticipated to be stable for global stability. Slope erosion protection measures will be required, and such measures should be confirmed during detailed design.

It is recommended that all topsoil/organic material and existing surficial fill materials be stripped from the footprint of the new or widened approach embankments. The depth and extent of stripping should be assessed during detail design when additional subsurface information will be available for the widened approach embankment areas.

If a widening is proposed, the embankment fill widening should be placed and compacted in accordance with MTO's Special Provisions. Benching of the existing embankment side slopes should be carried out to "key in" the new fill materials for the widening, in accordance with OPSD 208.010 *(Benching of Earth Slopes).*

Additional fill for construction of the embankment widening could consist of clean earth fill or granular fill. From a geotechnical/foundations perspective, both earth and granular fill will provide good compatibility with the existing embankment fill materials – both those fill materials remaining in-place in the existing embankment side slope, and any existing embankment fill that is re-used for the widening after being cut from the benches.

In accordance with MTO's standard practice, a minimum 2 m wide bench should be provided where the embankment side slopes are equal to or greater than 8 m in height, such that the uninterrupted height of slope does not exceed 8 m. To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod is recommended as soon as practicable after construction of the embankments.

Slopes must be provided with erosion protection in accordance with OPSS.PROV 803 and OPSS.PROV 804. A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion. Surface water should be directed away from the slopes and conveyed down the slope in appropriately designed drainage channels or storm sewers.

11.1 Approach Embankment Stability

Slope stability analyses have been performed for embankments using the commercially available program Slope/W, to check that a minimum factors of safety of 1.3 and 1.5 are achieved for the proposed embankment heights and geometries under short-term and long-term static conditions, respectively. This minimum factor of safety is considered appropriate for the proposed northward and southward embankment widenings on this project, considering the design requirements and the available field and laboratory testing data.

The analysis results indicate that an approximately 8 m high embankment with side slopes oriented no steeper than 2H:1V will have a factor of safety of greater than 1.5 against global instability, assuming appropriate subgrade preparation and proper placement and compaction of the embankment fill materials.

Results of the slope stability analyses are presented in figures in Appendix I and summarized in the following table.

Table 11-1: Summary of Stability Assessments

Slope Inclination	Condition	Estimated Factor of Safety	Figure
2H:1V	Undrained	1.7	I1
	Drained	1.6	I2
	Seismic	1.3	I3

This preliminary assessment of the stability of approach embankments should be reviewed and confirmed based on the subsoil conditions encountered within the proposed approach embankment footprints during detail design.

11.2 Approach Embankment Settlement

Based on our preliminary assessment, the settlement of the foundation soils under the embankments with height of up to 7 m is estimated to be approximately 25 mm. This settlement is expected to occur relatively quickly during and immediately following construction of the widened approach embankments based on the nature of the soils at the site. This estimated magnitude of settlement should be reassessed based on the soil and groundwater conditions under the new approach embankments as determined during the detail design, with particular emphasis on the thickness and properties of any surficial soil deposits within the embankment widening footprint.

The above preliminary estimates do not include compression of the fill itself, which would occur during and after the construction of the embankment depending on the type of materials used. The magnitude of fill compression may range from 0.5 to 1 per cent of the newly placed height of the embankment, assuming approximately 98 per cent compaction of the embankment fill is achieved, relative to the material's standard Proctor maximum dry density. In the case where granular fill is used for embankment construction, settlement of the fill itself is expected to occur essentially during embankment construction, whereas non-granular earth fill materials are expected to exhibit some additional settlement over time.

Where construction is to be carried out in it stages, settlement of the existing nearest lane may occur and maintenance may be required during construction.

12. EXCAVATION AND DEWATERING

Excavations for footings, pile caps, and embankments are anticipated to extend through the surficial fill, and native silty clay deposit.

All temporary excavations must be carried out in accordance with the current Occupational Health and Safety Act (OHSA) of Ontario and local regulations. Provided adequate groundwater control is achieved, the soils above the water level are classified as Type 3 soils under OHSA. Soft clayey soils and loose, wet cohesionless soils are classified as Type 4 soils. Where an unsupported excavation penetrates more than one soil type, the entire excavation must be completed in accordance with the more stringent requirements as per the requirements of the regulation. Slopes of temporarily unsupported cuts should conform with the requirements of OHSA but should not be steeper than 1H:1V. Flatter slopes may be required at locations where water seepage or sloughing occurs during excavation. The excavation slopes should be continuously monitored for evidence of instability. Unsupported excavations below the water table are not recommended.

Where space restrictions preclude excavation of inclined slopes, excavation may be carried out using a shored excavation. Soil should not be stockpiled within a horizontal distance from the excavation wall equal to the depth of the excavation.

Use of a hydraulic excavator should be suitable for excavation in the fill and native overburden soils. The selection of the method of excavation is the responsibility of the contractor and must be based on their equipment, experience, and interpretation of the site conditions. The fill and native overburden deposits may contain cobbles and boulders and other obstructions, and the contractor must be prepared to handle these obstructions.

Dewatering of all excavations, if required, shall be carried out in accordance with OPSS.PROV 517 as modified by SP 517F01. The dewatering system is the responsibility of the Contractor, the dewatering system will be required to remain operational and effective until the excavations are backfilled.

13. EXCAVATIONS AND TEMPORARY SHORING

Excavations for footings, pile caps, and embankments are anticipated to extend through the surficial fill, and native silty clay deposit.

Where excavation space is limited, consideration may be given to using temporary shoring for advancement of excavations. The design of temporary shoring should be the responsibility of the contractor.

The design of the shoring system should be carried out by an experienced Professional Engineer in accordance with the latest version of the Canadian Foundation Engineering Manual (CFEM). The soil in the table below may apply for the design of the temporary shoring system.

Table 13-1 Soil Parameters for Temporary Shoring System Design

Soil Parameter	Existing Fill	Silty Clay
Angle of Internal Friction (degrees)	28	30
Total Unit Weight (kN/m^3)	21	20
K_a	0.36	0.33
K_0	0.53	0.50

K_a can be used for the design of a flexible retaining system where minor movement can be tolerated. K_o should be used in the design of a rigid shoring system where movement must be minimal.

As excavation will be carried out near existing structures and utilities care must be taken not to undermine these structures.

14. CORROSION AND SULPHATE ATTACK POTENTIAL

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the subsurface environment. The test results may be used to aid in the selection of coatings and corrosion protection systems. The corrosive effects of road deicing salts/brine should also be considered. Based on the test results, the following statements can be made:

- The potential for sulphate attack on concrete from the surrounding native soils is considered to be negligible due to the low concentration of sulphate and slightly alkaline pH values.
- The overall potential for corrosion on metal is considered high for the native soils (Silty Clay and Shale) samples taken at depths ranging from 8.4 m to 9.0 m.

15. RECOMMENDATIONS FOR ADDITIONAL WORK

The preliminary foundation recommendations provided in this report are based on the limited available information on the proposed structures themselves and the subsurface information obtained from boreholes advanced in the general vicinity of the proposed structure.

Additional subsurface investigation should be carried out to confirm the soil and groundwater conditions at the location of the bridge foundation elements, approach embankments, deep cuts and high fills, and/or retaining walls, as applicable.

Additional boreholes must be advanced within the footprint of the bridge abutment and pier locations and temporary bridge location(s) (if any) and in the area of the approach embankments / RSS Walls with the minimum number of boreholes and laboratory testing requirements as per the latest version of the MTO Guideline for Foundation Engineering Services. Boreholes should be advanced to a depth sufficiently below the anticipated footing design elevations as per MTO requirements to confirm the occurrence of appropriate foundation material.

Due to potentially variable depth and quality of the shale bedrock, boreholes should extend into sound bedrock.

The global stability of the approach embankments, retaining walls (if any), and magnitude of settlement of the foundation soils will need to be reassessed. All bearing resistances provided must be reassessed.

16. CLOSURE

Engineering analysis and preparation of this report was carried out by Ms. Madisan Chiarotto. The report was reviewed by Mr. Mehdi Mostakhdemi, a MTO Designated Principal Contact.

Thurber Engineering Ltd.
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Table 1: Summary of the advantages and disadvantages of foundation options

Foundation Option	Advantages	Disadvantages	Recommendation
Spread footings	• Conventional construction requiring relatively shallow excavations at abutment locations • Compatible with the foundations of the existing structure	• Relatively large area of excavations, and temporary protection may be required, specifically at the pier locations • May require dewatering • May preclude use of integral abutments	PREFERRED OPTION 1
Driven Steel Piles	• Higher bearing resistances than spread footings • Lesser amounts of differential settlement between existing and the widened structure if widening is planned • Reduced excess soil and/or slurry spoils	• Will require the abutments be perched on granular fill due to shallow depth to bedrock • Not feasible at the pier locations due to shallow depth to bedrock • May not have adequate penetration to satisfy lateral load demands	TECHNICALLY FEASIBLE – NOT PREFFERED OPTION
Drilled shafts (caissons) socketed into bedrock	• Lesser amounts of differential settlement between existing and the widened structure if widening is planned • Reduced excess soil and/or slurry spoils • Would likely eliminate construction of pile caps at the pier locations	• May preclude use of integral abutments	PREFFERRED OPTION 2

STATEMENT FOR USE AND INTERPRETATION OF REPORT

1. STANDARD OF CARE

This Report has been prepared in a manner consistent with that degree of care and skill ordinarily exercised by members of the same profession currently practicing under similar circumstances at the same time and in the same or similar locality and in compliance with all applicable laws.

2. COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment, including this Statement For Use and Interpretation of Report, are a part of the Report, which is of a summary nature and is not intended to stand alone without reference to the instructions given to Thurber by the Client, communications between Thurber and the Client, and any other reports, proposals or documents prepared by Thurber for the Client relative to the specific site described herein, all of which together constitute the Report.

IN ORDER TO PROPERLY UNDERSTAND THE SUGGESTIONS, RECOMMENDATIONS AND OPINIONS EXPRESSED HEREIN, REFERENCE MUST BE MADE TO THE WHOLE OF THE REPORT, AS DESCRIBED ABOVE. THURBER IS NOT RESPONSIBLE FOR USE BY ANY PARTY OF PORTIONS OF THE REPORT WITHOUT REFERENCE TO THE WHOLE OF THE REPORT.

3. BASIS OF REPORT

The Report has been prepared for the specific site, development, design objectives, and purposes that were described to Thurber by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the Report, subject to the limitations provided herein, are only valid to the extent that the Report expressly addresses proposed development, design objectives and purposes, and then only to the extent that there has been no material alteration to or variation from any of the said descriptions provided to Thurber, unless Thurber is specifically requested by the Client to review and revise the Report in light of such alteration or variation.

4. USE OF THE REPORT

The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client for the development, design objectives, and/or purposes described to Thurber by the Client. **NO OTHER PARTY MAY USE OR RELY ON THE REPORT OR ANY PORTION THEREOF FOR OTHER THAN THE CLIENT'S BENEFIT IN CONNECTION WITH THE PURPOSES DESCRIBED IN THE REPORT**. Any use which a third party makes of the Report is the sole responsibility of such third party and is always subject to this Statement for Use and Interpretation of Report. Thurber accepts no liability or responsibility for damages suffered by any third party resulting from use of the Report for purposes outside the reasonable contemplation of Thurber at the time it was prepared or in any manner unintended by Thurber.

5. INTERPRETATION OF THE REPORT

a) **Nature and Exactness of Soil and Contaminant Description:** Classification and identification of soils, rocks, geological units, contaminant materials and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors is inherently judgement-based. Comprehensive sampling and testing programs implemented with the appropriate equipment by experienced personnel may fail to locate some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and the Client and all other parties making use of such documents or records with or without our express written consent need to be aware of this risk and the Report is delivered subject to the express condition that such risk is accepted by the Client and such other parties. Some conditions are subject to change over time and those making use of the Report need to be aware of this possibility and understand that the Report only presents the interpreted conditions at the sampled points at the time of sampling. If special concerns exist, or the Client has special considerations or requirements, the Client must disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.

b) **Reliance on Provided Information:** The evaluation and conclusions contained in the Report have been prepared based on conditions in evidence at the time of site inspections and based on information provided to Thurber. Thurber has relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, Thurber does not accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report resulting from misstatements, omissions, misrepresentations, or fraudulent acts of the Client or other parties providing information relied on by Thurber. Thurber is entitled to rely on such representations, information and instructions and is not required to carry out investigations to determine the truth or accuracy of such representations, information and instructions.

c) **Design Services:** The Report may form part of design and construction documents for information purposes even though it may have been issued prior to final design being completed. Thurber is recommended to be retained to review final design, project plans and related documents prior to construction to confirm that they are consistent with the intent of the Report. Any differences that may exist between the Report's recommendations and the final design need to be reported to Thurber immediately so that Thurber can address potential conflicts.

d) **Construction Services:** During construction Thurber should be retained to provide field reviews. Field reviews consist of performing sufficient and timely observations of encountered conditions to confirm and document that the site conditions do not materially differ from those conditions considered in the preparation of the report. Adequate field reviews are necessary for Thurber to provide letters of assurance, in accordance with the requirements of many regulatory authorities.

6. INDEPENDENT JUDGEMENTS OF CLIENT

The information, interpretations and conclusions in the Report are based on Thurber's interpretation of conditions revealed through limited investigation conducted within a defined scope of services. Thurber does not accept responsibility for independent conclusions, interpretations, interpolations and/or decisions of the Client, or other parties who may come into possession of the Report, or any part thereof, which may be based on information contained in the Report. This restriction of liability includes, but is not limited to, decisions made to develop, purchase, or sell land, unless such decisions expressly form part of the stated purpose of the Report as described in Paragraph 3.

This document replaces Thurber's 2014 Statement of Limitations and Conditions

APPENDIX A

Site Photographs

Photograph 1: Casablanca Boulevard Bridge, looking North

Photograph 2: Casablanca Boulevard Bridge overlooking QEW, looking West

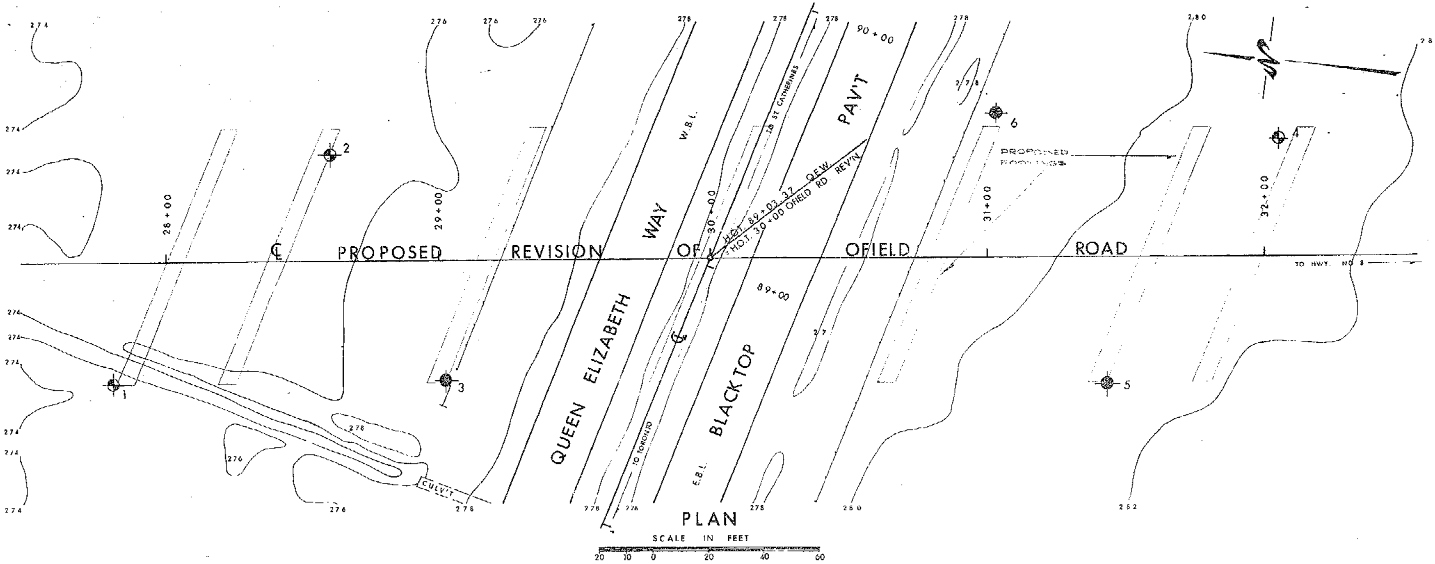
Photograph 3: Borehole 25-01 during investigation, looking North

Photograph 4: Borehole 25-01 after competition, looking East

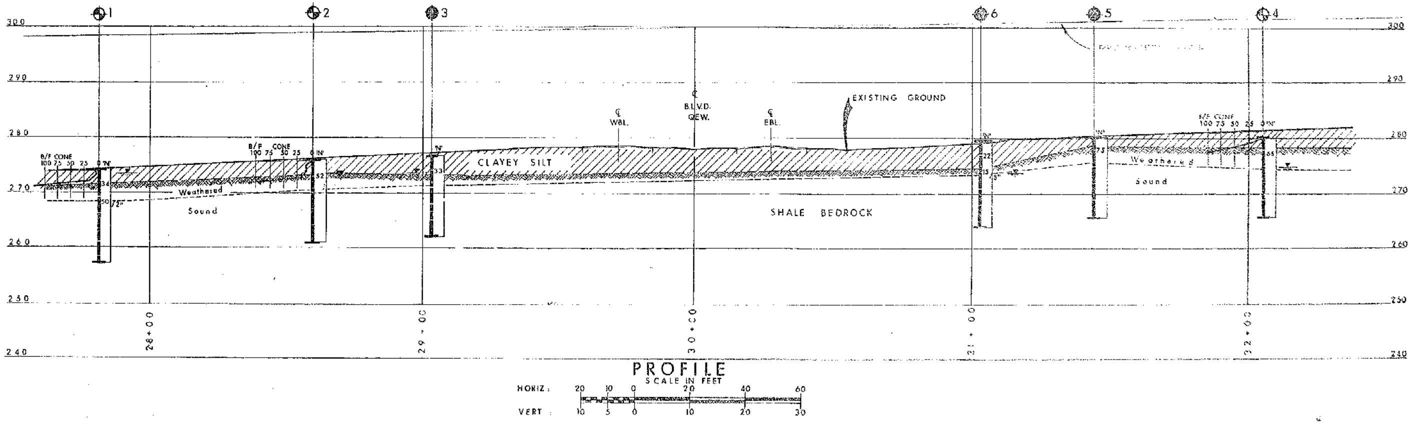
Photograph 5: Borehole 25-02 after completion, looking west

APPENDIX B

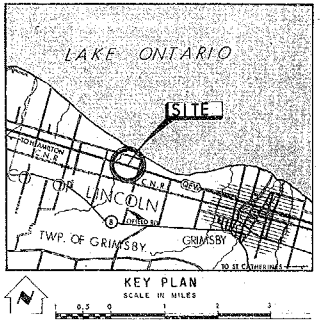
Background Data



PLAN



PROFILE



KEY PLAN

LEGEND

●	Bore Hole
⊕	Cone Penetration Hole
◐	Bore & Cone Penetration Hole
▽	Water Levels established at time of field investigation. JUNE, 1966.

NO.	ELEVATION	STATION	OFFSET
1	274·8	27+80	45'RT
2	275 4	28+60	37'LT
3	276 6	29+02	44'RT
4	280·4	32+ 05	42'LT
5	280·9	31+43	47'RT
6	279·5	31+01	46'LT

-- NOTE --

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

REVISIONS DATE	BY	DESCRIPTION

PRINT RECORD		
NO.	FOR	DATE

DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & TESTING DIVISION – FOUNDATION SECTION

OFIELD ROAD REVISION

KING'S HIGHWAY NO. Q.E.W DIST. NO. 4
CO. LINCOLN
TWP. NORTH GRIMSBY LOT 17 CON. B.F.C & 1

BORE HOLE LOCATIONS & SOIL STRATA

SUBM'D. L.P.	CHECKED	W.P. NO. 224 - 63	M.&T. DRAWING NO. 66-F-54A
DRAWN J.N.	CHECKED	JOB NO. 66 - F - 54	
DATE 20, JULY ,1966		SITE NO.	BRIDGE DRAWING NO.
APPROVED PRINCIPAL FOUNDATION ENGINEER		CONT. NO.	

REF. NO.: E 4729-1

ABBREVIATIONS USED IN THIS REPORT

PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N' :- THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 12 INCHES INTO THE SUBSOIL, DRIVEN BY MEANS OF A 140 POUND HAMMER FALLING FREELY A DISTANCE OF 30 INCHES.

DYNAMIC PENETRATION RESISTANCE :- THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 2 INCH, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 12 INCHES INTO THE SUBSOIL, THE DRIVING ENERGY BEING 350 FOOT POUNDS PER BLOW.

DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

CONSISTENCY	'N' BLOWS / FT.	c LB./ SQ. FT.	DENSENESS	'N' BLOWS / FT.
VERY SOFT	0 - 2	0 - 250	VERY LOOSE	0 - 4
SOFT	2 - 4	250 - 500	LOOSE	4 - 10
FIRM	4 - 8	500 - 1000	COMPACT	10 - 30
STIFF	8 - 15	1000 - 2000	DENSE	30 - 50
VERY STIFF	15 - 30	2000 - 4000	VERY DENSE	> 50
HARD	> 30	> 4000		

TYPE OF SAMPLE

S.S.	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.B.	SCRAPER BUCKET SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE
S.T.	SLOTTED TUBE SAMPLE		

P.H.	SAMPLE ADVANCED HYDRAULICALLY
P.M.	SAMPLE ADVANCED MANUALLY

SOIL TESTS

Qu	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V.	FIELD VANE
Qcu	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	DRAINED TRIAXIAL	S	SENSITIVITY

ABBREVIATIONS USED IN THIS REPORT

SOIL PROPERTIES

γ	UNIT WEIGHT OF SOIL (BULK DENSITY)	
γ_s	UNIT WEIGHT OF SOLID PARTICLES	
γ_w	UNIT WEIGHT OF WATER	
γ_d	UNIT DRY WEIGHT OF SOIL (DRY DENSITY)	
γ'	UNIT WEIGHT OF SUBMERGED SOIL	
G	SPECIFIC GRAVITY OF SOLID PARTICLES $G = \frac{\gamma_s}{\gamma_w}$	
e	VOID RATIO	
n	POROSITY	
w	WATER CONTENT	
S_r	DEGREE OF SATURATION	
w_L	LIQUID LIMIT	
w_p	PLASTIC LIMIT	
I_p	PLASTICITY INDEX	
s	SHRINKAGE LIMIT	
I_L	LIQUIDITY INDEX $= \frac{w - w_p}{I_p}$	
I_c	CONSISTENCY INDEX $= \frac{w_L - w}{I_p}$	
e_{max}	VOID RATIO IN LOOSEST STATE	
e_{min}	VOID RATIO IN DENSEST STATE	
I_D	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$	
	RELATIVE DENSITY D_r IS ALSO USED	
h	HYDRAULIC HEAD OR POTENTIAL	
q	RATE OF DISCHARGE	
v	VELOCITY OF FLOW	
i	HYDRAULIC GRADIENT	
k	COEFFICIENT OF PERMEABILITY	
j	SEEPAGE FORCE PER UNIT VOLUME	
m_v	COEFFICIENT OF VOLUME CHANGE $= \frac{-\Delta e}{(1+e)\Delta\sigma}$	
c_v	COEFFICIENT OF CONSOLIDATION	
C_c	COMPRESSION INDEX $= \frac{\Delta e}{\Delta \log_{10} \sigma}$	
T_v	TIME FACTOR $= \frac{c_v t}{d^2}$ (d, DRAINAGE PATH)	
U	DEGREE OF CONSOLIDATION	
τ_f	SHEAR STRENGTH	
c'	EFFECTIVE COHESION INTERCEPT	IN TERMS OF EFFECTIVE STRESS $\tau_f = c' + \sigma' \tan \phi'$
ϕ'	EFFECTIVE ANGLE OF SHEARING RESISTANCE, OR FRICTION	
c_u	APPARENT COHESION	IN TERMS OF TOTAL STRESS $\tau_f = c_u + \sigma \tan \phi$
ϕ_u	APPARENT ANGLE OF SHEARING RESISTANCE, OR FRICTION	
μ	COEFFICIENT OF FRICTION	
S_t	SENSITIVITY	

GENERAL

π	$= 3{\cdot}1416$
e	BASE OF NATURAL LOGARITHMS 2·7183
$\log_e a$ OR $\ln a$	NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$	LOGARITHM OF a TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

STRESS AND STRAIN

u	PORE PRESSURE
σ	NORMAL STRESS
σ'	NORMAL EFFECTIVE STRESS ($\bar{\sigma}$ IS ALSO USED)
τ	SHEAR STRESS
ε	LINEAR STRAIN
γ	SHEAR STRAIN
ν	POISSON'S RATIO (μ IS ALSO USED)
E	MODULUS OF LINEAR DEFORMATION (YOUNGS MODULUS)
G	MODULUS OF SHEAR DEFORMATION
K	MODULUS OF COMPRESSIBILITY
η	COEFFICIENT OF VISCOSITY

EARTH PRESSURE

d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_o	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY. DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
k_s	MODULUS OF SUBGRADE REACTION

SLOPES

H	VERTICAL HEIGHT OF SLOPE
D	DEPTH BELOW TOE OF SLOPE TO HARD STRATUM
β	ANGLE OF SLOPE TO HORIZONTAL

FORM DB-MT-126 (FORMERLY DB-ML-126)
65-1831

OFFICE REPORT ON SOIL EXPLORATION

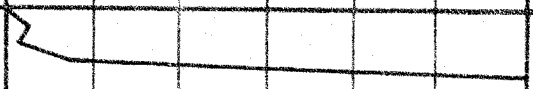
DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

JOB 66-F-54
W. P. 224-63
DATUM Geodetic
LOCATION QEW & Ofield Rd., Sta. 27/80; 45' Rt.
BORING DATE June 1, 1966.
BOREHOLE TYPE Cone: Washboring BX; BXL
ORIGINATED BY L.P.
COMPILED BY W.T.E.
CHECKED BY M.D.

SOIL PROFILE			SAMPLES				DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100 SHEAR STRENGTH P.S.F.	LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w WATER CONTENT %		
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE			BULK DENSITY γ P.C.F.	REMARKS
274.8	Groundlevel									
0.0	Clayey silt.	▨								▼ WL
271.6 3.2			1	SS	34	270				
268.5 6.3	(Weathered)		2	SS	50/2"					El. 273.5
	Shale Bedrock (Sound)		3	RC BXL	-					Rec. 87%
			4	RC BXL	-	260				Rec. 87%
258.0 16.8	End of borehole.									



FORM OB-MT-126 (FORMERLY OB-ML-126)
65-1831

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

JOB 66-F-54
W.P. 224-63
DATUM Geodetic

LOCATION QEW & Ofield Rd., Sta. 28+60, 37' Rt.
BORING DATE June 2, 1966.
BOREHOLE TYPE Cone: Washboring BX; BXL

ORIGINATED BY L.P.
COMPILED BY W.T.E.
CHECKED BY M.D.

SOIL PROFILE			SAMPLES				DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100 SHEAR STRENGTH P.S.F.	LIQUID LIMIT —— w_L PLASTIC LIMIT —— w_P WATER CONTENT —— w WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS
ELEV / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV SCALE				
275.4	Groundlevel									
273.2	Clayey Silt									▼ WL
2.2 270.6	(Weathered)		1	SS	52	270	25/7"			El. 273
4.8	Shale Bedrock (Sound)		2	RC (BXL)	-					Rec. 94%
			3	RC (BXL)	-					Rec. 78%
260.4 15.0	End of Borehole.					260				

FORM DB-MT-126 (FORMERLY DB-ML-126)
65-1831

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 3

FOUNDATION SECTION

JOB 66-F-54
W.P. 224-63
DATUM Geodetic

LOCATION QEW & Ofield Rd., Sta. 29/02, 44' Rt.
BORING DATE June 2, 1966.
BOREHOLE TYPE Washboring BX; BXL

ORIGINATED BY L.P.
COMPILED BY W.T.E.
CHECKED BY M.D.

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT SHEAR STRENGTH P.S.F.	LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS
ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT					
276.6	Groundlevel									
0.0	Clayey Silt	▨								
273.2		▨	1	SS	33					▼ WL El. 273.5
271.5	(Weathered)	▨								
5.1	Shale Bedrock (Sound)		2	RC (BXL)	-	270				Rec. 83%
			3	RC (BXL)	-					Rec. 100%
262.0										
14.6	End of borehole.					260				

FORM OB-MT-126 (FORMERLY OB-ML-126)
65-1831

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 4

FOUNDATION SECTION

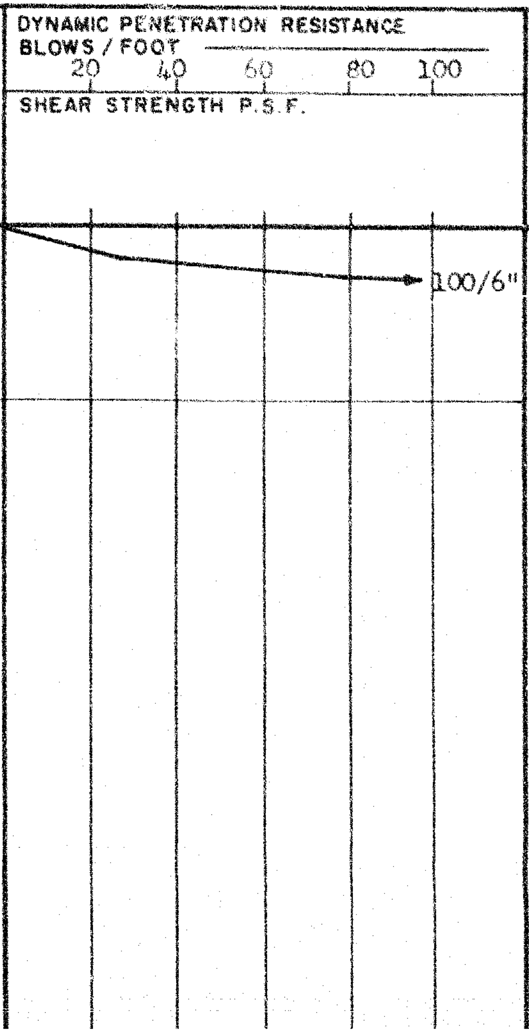
JOB 66-F-54
W.P. 224-63
DATUM Geodetic

LOCATION ℄Em & Ofield Rd., Sta. 32+05, 42' Lt.
BORING DATE June 3, 1966.
BOREHOLE TYPE Cone, Washboring BX; BXL

ORIGINATED BY L.P.
COMPILED BY W.T.E.
CHECKED BY M.D.

SOIL PROFILE			SAMPLES				
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	REMARKS
280.4	Groundlevel						
278.4 2.0	Clayey-silt (Weathered)		1	SS	68		WL Elev. 274.9
				BXL			
274.5 5.9	Shale Bedrock (Sound)		2	SS	76/1"		
			3	RC (BAL)	-	270	Rec. 92%
265.1			4	RC (BXL)	-		Rec. 95%
15.3	End of borehole.						

DYNAMIC PENETRATION RESISTANCE
BLOWS / FOOT
SHEAR STRENGTH P.S.F.



LIQUID LIMIT —— w_L
PLASTIC LIMIT —— w_P
WATER CONTENT —— w

WATER CONTENT %

BULK DENSITY γ P.C.F.

FORM DB-MT-126 (FORMERLY DB-ML-124)
65-1831

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 5

FOUNDATION SECTION

JOB 66-F-54
W.P. 224-63
DATUM Geodetic
LOCATION QEW & Ofield Rd., Sta. 31+43, 47' Rt.
BORING DATE June 3, 1966.
BOREHOLE TYPE Washboring BX, BXL
ORIGINATED BY L.P.
COMPILED BY W.T.E.
CHECKED BY M.D.

SOIL PROFILE			SAMPLES				DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT SHEAR STRENGTH P.S.F.	LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w w_P — w — w_L WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS
ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE				
280.9	Groundlevel									
278.9	Clayey silt.	▨				280				
2.0		▨	1	SS	75					
275.9	(Weathered)	▨								
5.0	Shale Bedrock (Sound)		2	RC (BXL)	-					Rec. 90%
			3	RC (BXL)	-	270				Rec. 100%
265.4										
15.5	End of borehole.									

FORM OB-MT-126 (FORMERLY OB-ML-126)
65-1831

OFFICE REPORT ON SOIL EXPLORATION

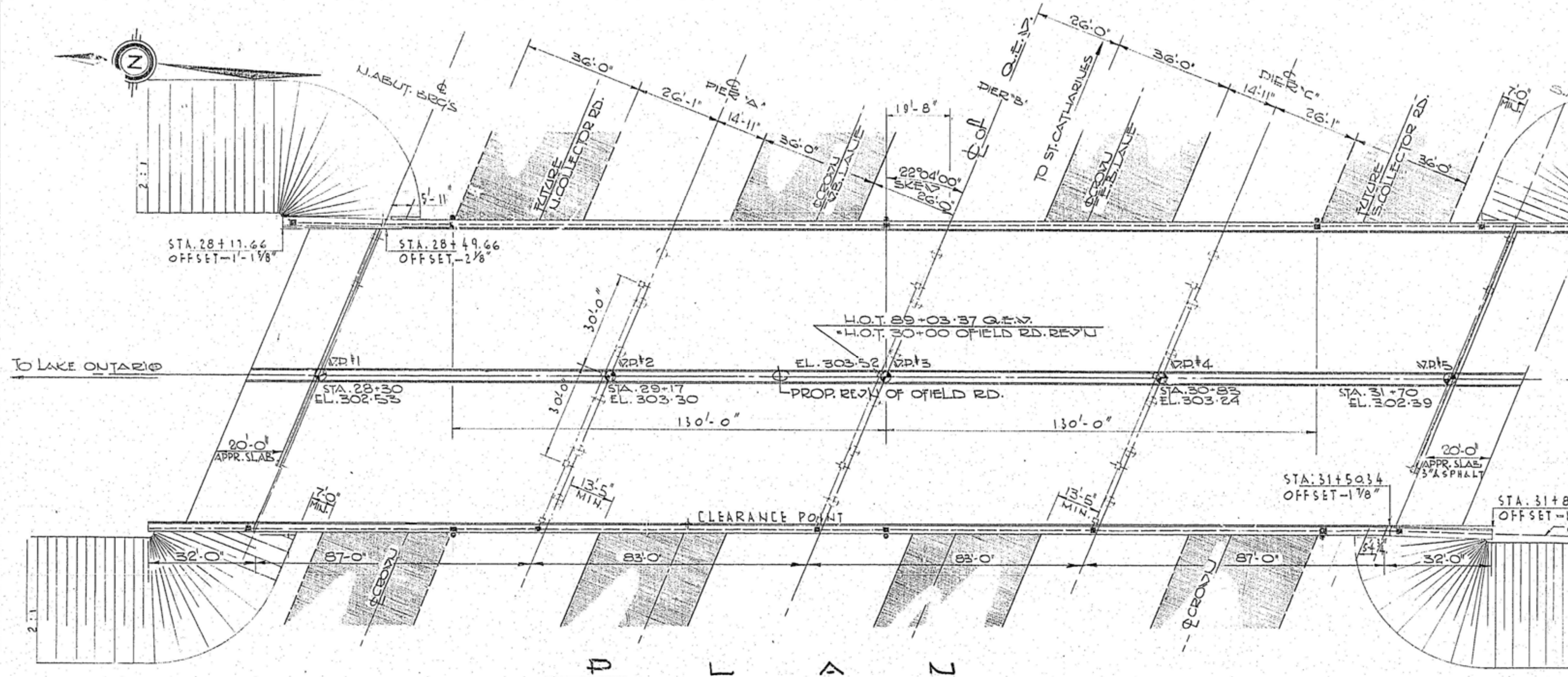
DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 6

FOUNDATION SECTION

JOB 66-F-54
W.P. 224-63
DATUM Geodetic
LOCATION Ofield Rd. & QEW, Sta. 31/01, 46' Lt.
BORING DATE June 6, 1966.
BOREHOLE TYPE Washboring BX; BXL
ORIGINATED BY L.P.
COMPILED BY W.T.E.
CHECKED BY M.D.

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT SHEAR STRENGTH P.S.F.	LIQUID LIMIT — w_L PLASTIC LIMIT — w_P WATER CONTENT — w w_P w w_L WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS
ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT					
279.5	Groundlevel									
0.0	Clayey silt	▨								
		▨	1	SS	22					WL Elev.
274.5		▨								
273.5	(Weathered)	▨	2	SS	15/5"					▼ 273.5
	Shale Bedrock		3	RC (BXL)	-	270				Rec. 94%
	(Sound)		4	RC (BXL)	-					Rec. 100%
264.2										
15.3	End of borehole.									



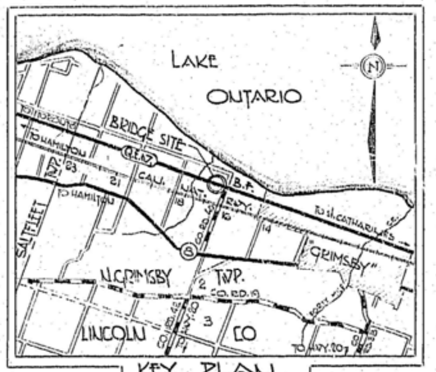
PLAN

SCALE: 1" = 20'-0"

ELECTRICAL DETAIL LEGEND.
- ------ 3" DUCT
- -·-·-·- 2" DUCT
- ▪ 8"X8"X8" JUNCTION. BOX
- ☐ 8"X8"X8" JUNCTION BOX
- ● LIGHING POLE LOCATION

SEE DRWG. D-6029-13

SKEW 22°04'00"
SIN. = 0·3756852
COS. = 0·9267474
TAN. = 0.4053804
SEC. = 1.0790427



KEY PLAN
SCALE - 1 IN. = 1 MI.

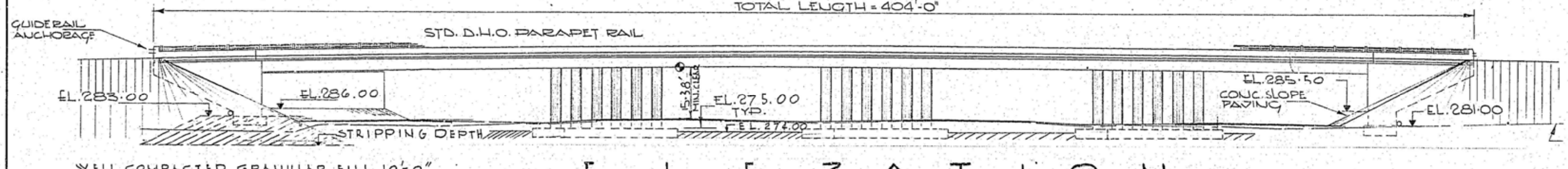
GENERAL NOTES

CLASS OF CONCRETE
- PIER CAP, DECK, CURBS & CONC. ABOVE CURBS — 4000 P.S.I.
- PRESTRESSED BEAMS: CENTRE SPAN — 5000 P.S.I.
  END SPAN — 6000 P.S.I.
- REMAINDER — 3000 P.S.I.

CLEAR COVER ON REINFORCING STEEL
- FOOTINGS 3", ABUT'S 3", PIERS 2", DIAPHRAGMS 2", DECK TOP 1½", BOTT. 1"
  CURBS 2", APPR. SLAB 2", END POSTS & PARAPET 1½".

CONSTRUCTION NOTES
- THE CONTRACTOR IS RESPONSIBLE FOR FINISHING THE BEARINGS SEATS DEAD LEVEL TO THE SPECIFIED ELEVATIONS WITH A TOLERANCE OF ± 1/8"
- NO CONCRETE SHALL BE PLACED ABOVE THE ABUTMENT SEATS UNTIL THE CONCRETE IN THE DECK HAS BEEN PLACED
- PAYLINE FOR ROCK IS AT EL. 274.00
- COMPACTED FILL TO BE PLACED WELL IN ADVANCE TO ELIMINATE FUTURE SETTLEMENT.

LIST OF DRAWINGS
1. GENERAL PLAN
2. BORE HOLE LOC. & SOIL DATA
3. FOOTING LAYOUT
4. N & S ABUTMENT & WINGWALLS
5. PIERS
6. PRESTRESSED GIRDERS
7. DECK & CURBS
8. APPROACH SLAB
9. DETAILS OF CONC. SLOPE PAVING
10. STD. STEEL PARAPET RAIL
11. PARAPET WALL DETAILS
12. STANDARD DETAILS
13. BRIDGE ELECTRICAL DETAILS.
14. BRIDGE ELECTRICAL DETAILS

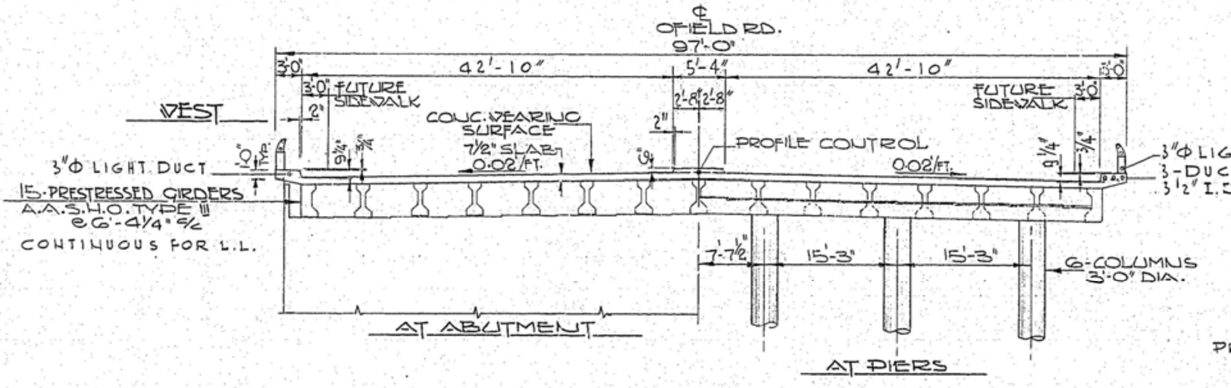


WELL COMPACTED GRANULAR FILL 10'-0" ALL AROUND FOOTING SEE D-6029-12

ELEVATION

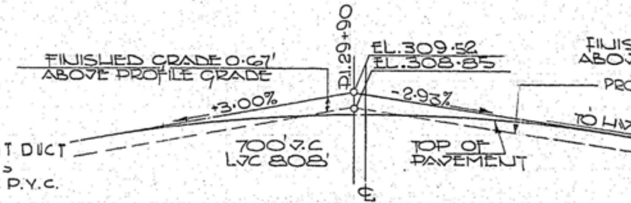
SCALE: 1" = 20'-0"

B.M. 275.40
GEODETIC DATUM
CUT "X" ON N.W. COR. OF CONC. CULV.
80'-0 RT. OF STA. 28+90 REV'N



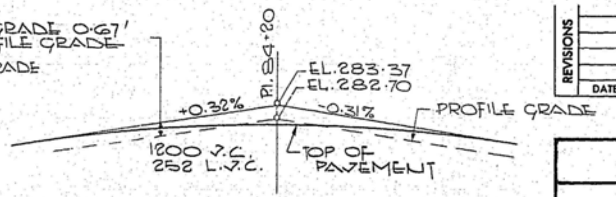
TYP. DECK SECTIONS

SCALE: 1" = 10'-0"



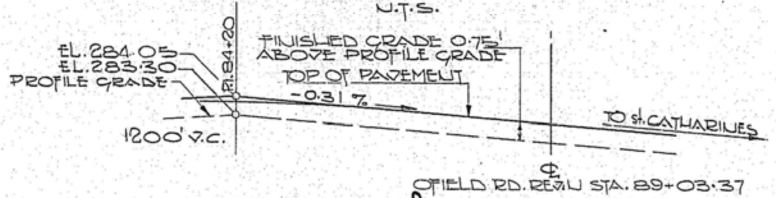
PROFILE of OFIELD RD. REV'N

N.T.S.



PROFILE OF FUTURE N & S. COLLECTOR RD'S

(CHAINAGES TAKEN SAME AS Q.E.W.)
N.T.S.



PROFILE of Q.E.W.

N.T.S.

PRINT RECORD		
No.	FOR	DATE
4	OS, P3	20/7/67

REVISIONS		
DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS ONTARIO
BRIDGE DIVISION

OFIELD ROAD UNDERPASS

16.3 MI. W. OF ST. CATHARINES W. LIMITS
KING'S HIGHWAY No. Q.E.W. DIST. No. 4
CO. LINCOLN
TWP. N. GRIMSBY LOT 17 CON. B.F. & 1

GENERAL PLAN

F. GORMEK

APPROVED		BRIDGE ENGINEER		SITE No. 18-196	W.P. No. 224-63
DESIGN	FG	CHECK	LN	CONTRACT Nos.	68-136
DRAWING	E.A.	CHECK	FG	DRAWING No.	D-6029-1
DATE	JUNE 67	LOADING	HS20-44		

TWP 136-196-1-A

N.W.C.L. TRANSLAR 2M-2-66

THURBER

APPENDIX C

Borehole Locations Drawing

MINISTRY OF TRANSPORTATION, ONTARIO

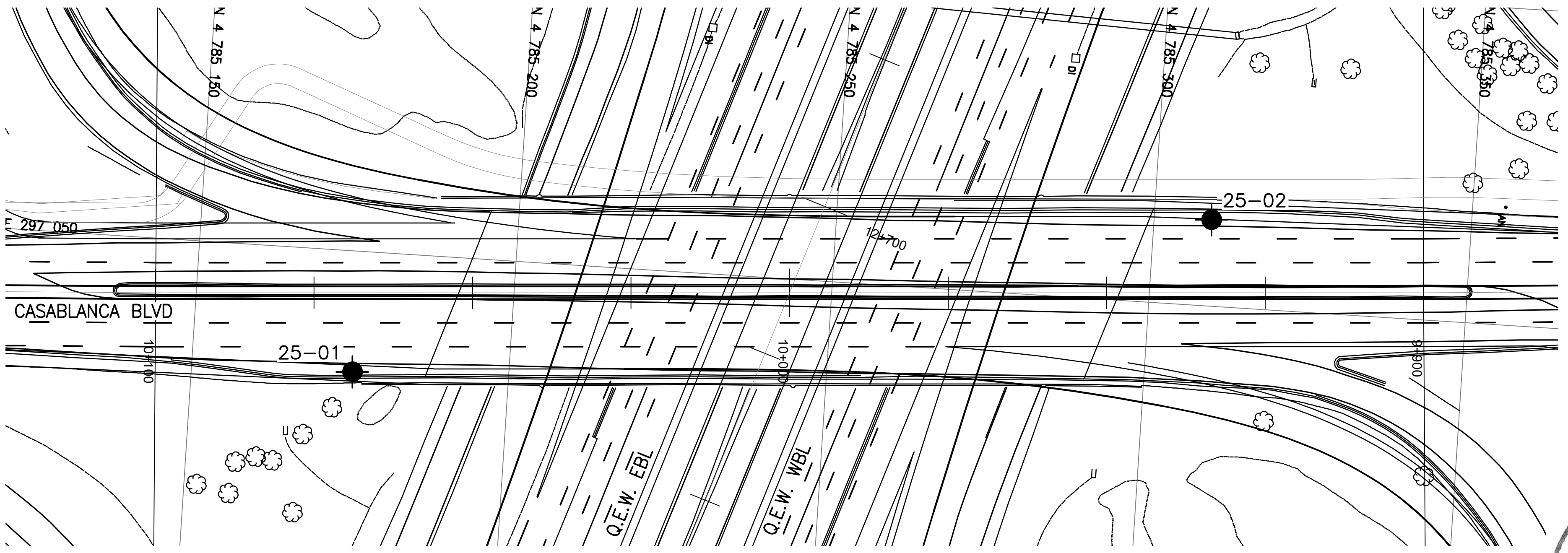
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
WP No

SHEET

Q.E.W.
CASABLANCA BLVD
INTERCHANGE
BOREHOLE LOCATIONS AND SOIL STRATA

THURBER



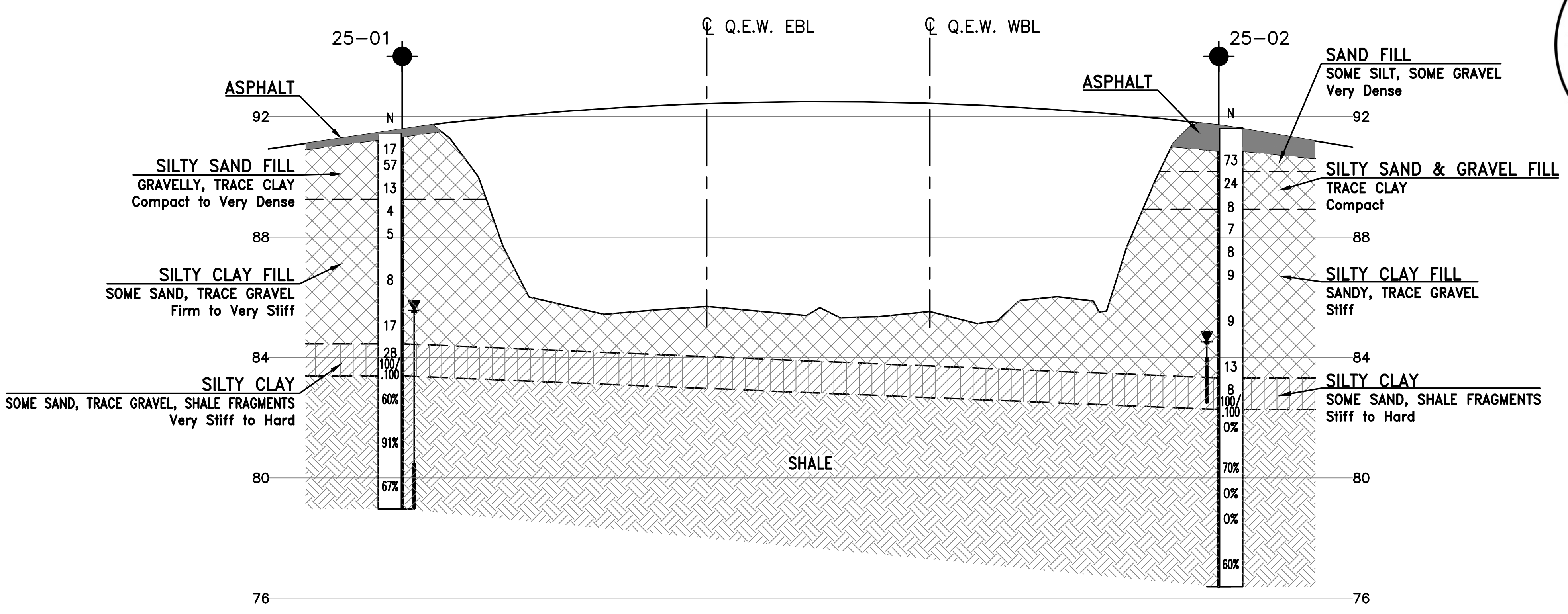
PLAN

SCALE 1:1000

LICENSED PROFESSIONAL ENGINEER
M. MOSTAKHDEMI
100185447
2026/05/04
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
M. P. CHIAROTTO
100554017
2026/05/04
PROVINCE OF ONTARIO

KEYPLAN



PROFILE ALONG ℄ CASABLANCA BLVD

H 1:1000
V 1:200

L E G E N D

●	Borehole
⊕	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
⊻	Water Level Upon Completion of Drilling
⊻	Water Level in Monitoring Well/Piezometer
┃	Monitoring Well/Piezometer Screen
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
25–01	91.5	4 785 175.4	297 069.1
25–02	91.6	4 785 309.1	297 036.4

-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 10.

GEOCRES No. 30M04-129

REVISIONS	DATE	BY	DESCRIPTION

DESIGN MC	CHK MM	CODE	LOAD	DATE MAY 2026
DRAWN MFA	CHK MC	SITE	STRUCT	DWG 1

FILENAME: H:\Drafting\73000\73956\TED–73956–BHPP.dwg
PLOTDATE: 5/4/2026 10:56 AM

APPENDIX D

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

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

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE

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

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

▼ Water Level
C_{pen} Shear Strength Determination by Pocket Penetrometer

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

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

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

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

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**		**Field Estimation of Hardness***
	(MPa)	**(psi)**	
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

Ministry of Transportation
Ontario

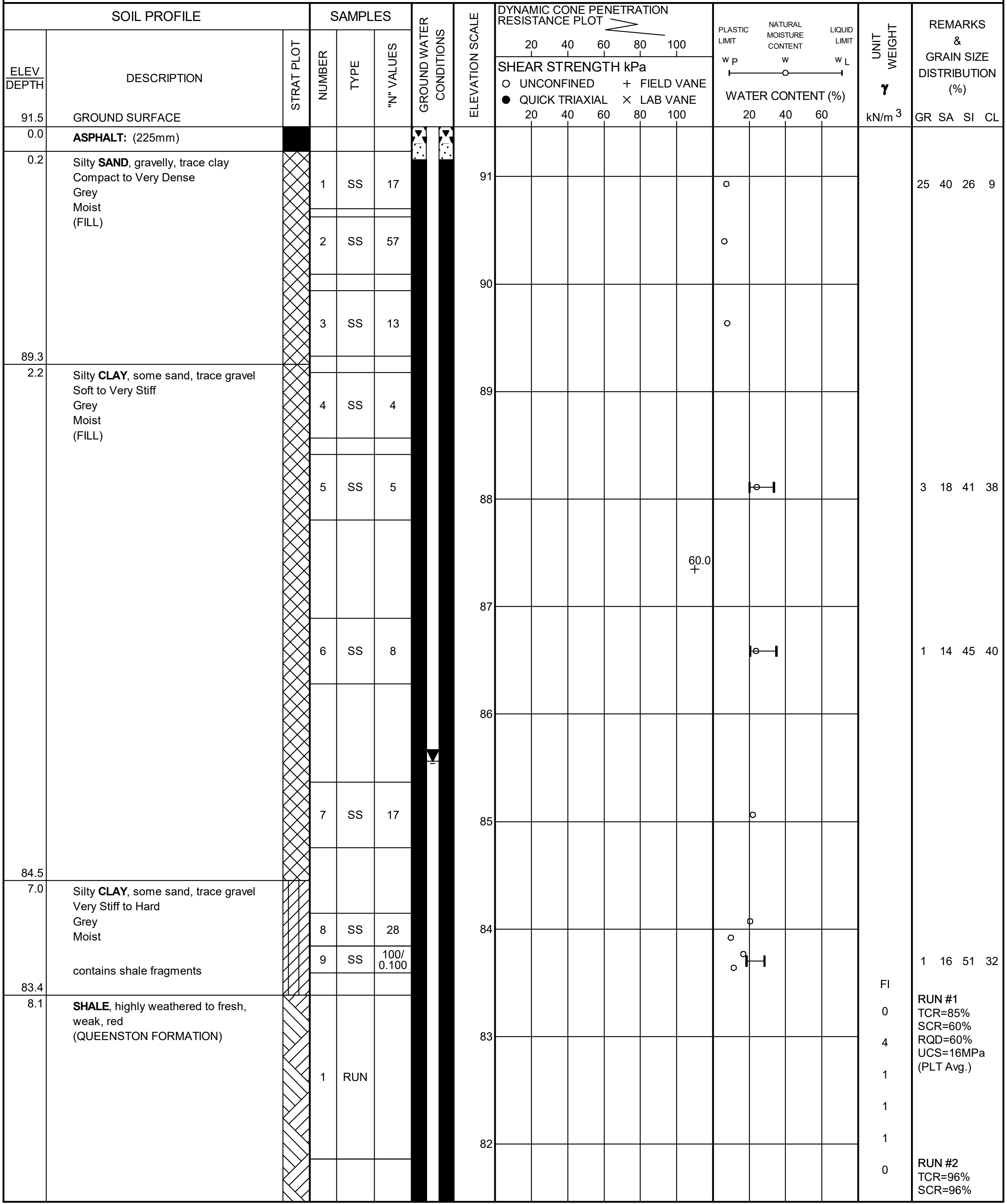
THURBER

RECORD OF BOREHOLE No 25-01

1 OF 2

METRIC

W.P. ______
LOCATION Casablanca Blvd QEW Interchange, Grimsby, ON: N 4 785 175.4 E 297 069.1
ORIGINATED BY BH
DIST Central HWY ______
BOREHOLE TYPE Hollow Stem Auger to 8.1m, HQ Coring from 8.1m to 12.5m
COMPILED BY MA
DATUM Geodetic
DATE 2025.10.21 - 2025.10.21
LATITUDE 43.206397
LONGITUDE -79.595139
CHECKED BY MC



ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
91.5	GROUND SURFACE					
0.0	**ASPHALT:** (225mm)					
0.2	Silty **SAND**, gravelly, trace clay Compact to Very Dense Grey Moist (FILL)	1	SS	17		25 40 26 9
		2	SS	57		
89.3		3	SS	13		
2.2	Silty **CLAY**, some sand, trace gravel Soft to Very Stiff Grey Moist (FILL)	4	SS	4		
		5	SS	5		3 18 41 38
		6	SS	8		1 14 45 40
84.5		7	SS	17		
7.0	Silty **CLAY**, some sand, trace gravel Very Stiff to Hard Grey Moist	8	SS	28		
83.4	contains shale fragments	9	SS	100/ 0.100		1 16 51 32
8.1	**SHALE**, highly weathered to fresh, weak, red (QUEENSTON FORMATION)	1	RUN		FI 0 4 1 1 1	RUN #1 TCR=85% SCR=60% RQD=60% UCS=16MPa (PLT Avg.)
					0	RUN #2 TCR=96% SCR=96%

Continued Next Page

+3, ×3: Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No 25-01

2 OF 2

METRIC

W.P. ______ LOCATION Casablanca Blvd QEW Interchange, Grimsby, ON: N 4 785 175.4 E 297 069.1 ORIGINATED BY BH
DIST Central HWY ______ BOREHOLE TYPE Hollow Stem Auger to 8.1m, HQ Coring from 8.1m to 12.5m COMPILED BY MA
DATUM Geodetic DATE 2025.10.21 - 2025.10.21 LATITUDE 43.206397 LONGITUDE -79.595139 CHECKED BY MC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	Continued From Previous Page										
			2	RUN			81			0 2 1 1	RQD=91% UCS=36.3MPa UCS=20MPa (PLT Avg.)
79.0			3	RUN			80 79			1 3 >10 3 1	**RUN #3** TCR=100% SCR=87% RQD=67% UCS=18.9MPa (PLT Avg.)
12.5	END OF BOREHOLE AT 12.5m Monitoring well installation consist of 50mm diameter schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS DATE DEPTH(m) ELEV.(m) 2025.11.24 5.9 85.6										

+ ³, × ³: Numbers refer to Sensitivity

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

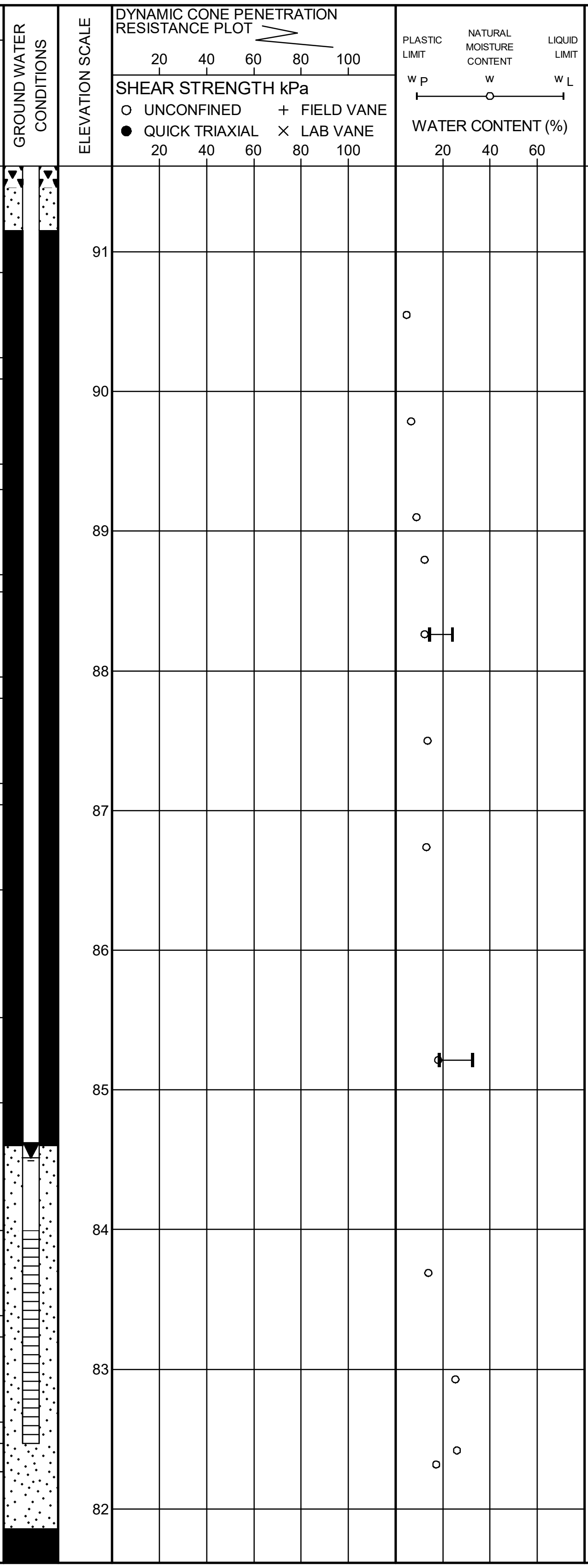
RECORD OF BOREHOLE No 25-02

1 OF 2

METRIC

W.P. ______
LOCATION Casablanca Blvd QEW Interchange, Grimsby, ON: N 4 785 309.1 E 297 036.4
ORIGINATED BY BH
DIST Central HWY ______
BOREHOLE TYPE Hollow Stem Auger to 9.3m, HQ Coring from 9.3m to 15.2m
COMPILED BY MA
DATUM Geodetic
DATE 2025.10.19 - 2025.10.20
LATITUDE 43.207600
LONGITUDE -79.595543
CHECKED BY MC

ELEV	DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
91.6		GROUND SURFACE								
	0.0	**ASPHALT:** (760mm)								
90.9	0.8	**SAND**, some silt, some gravel Very Dense Brown Moist (FILL)	1	SS	73					
90.2	1.4	Silty **SAND** and **GRAVEL**, trace clay Compact Brown Moist (FILL)	2	SS	24		42	36	22 (SI+CL)	
			3	SS	8					
88.9	2.7	Silty **CLAY**, sandy, trace gravel Stiff Brown Moist (FILL)	4	SS	7		5	24	40	31
			5	SS	8					
			6	SS	9					
			7	SS	9		3	20	39	38
			8	SS	13					
83.3	8.3	Silty **CLAY**, some sand Stiff to Hard Grey Moist shale fragments	9	SS	8		0	14	56	30
			10	SS	100/ 0.100	FI				
82.3	9.3	**SHALE**, highly weathered to fresh, weak, red (QUEENSTON FORMATION)	1	RUN		>10 >10	RUN #1 TCR=93% SCR=40% RQD=0% UCS=5.1MPa			



Continued Next Page

+³, ×³: Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No 25-02

2 OF 2

METRIC

W.P. ____
LOCATION Casablanca Blvd QEW Interchange, Grimsby, ON: N 4 785 309.1 E 297 036.4
ORIGINATED BY BH
DIST Central HWY ____
BOREHOLE TYPE Hollow Stem Auger to 9.3m, HQ Coring from 9.3m to 15.2m
COMPILED BY MA
DATUM Geodetic
DATE 2025.10.19 - 2025.10.20
LATITUDE 43.207600
LONGITUDE -79.595543
CHECKED BY MC

SOIL PROFILE			SAMPLES							
ELEV DEPTH	DESCRIPTION Continued From Previous Page	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
										7	(PLT Avg.)
										2	
							81			4	RUN #2 TCR=100% SCR=93% RQD=70% UCS=6.7MPa UCS=11MPa (PLT Avg.)
										4	
			2	RUN						2	
							80			1	
										1	
			3	RUN						1	RUN #3 TCR=86% SCR=43% RQD=0%
										0	
							79			0	RUN #4 TCR=0% SCR=0% RQD=0%
			4	RUN						0	
										0	
							78			0	RUN #5 TCR=100% SCR=60% RQD=60% UCS=13.4MPa (PLT Avg.)
										>10	
										2	
			5	RUN						2	
							77			0	
76.4										1	
15.2	END OF BOREHOLE AT 15.2m Monitoring well installation consist of 50mm diameter schedule 40 PVC pipe with a 1.52m slotted screen.										

WATER LEVEL READINGS

DATE	DEPTH(m)	ELEV.(m)
2025.11.24	7.1	84.5

ONTMT4S2 2020LIBRARY(MTO).GLB MTO-73956.GPJ 4/24/26

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

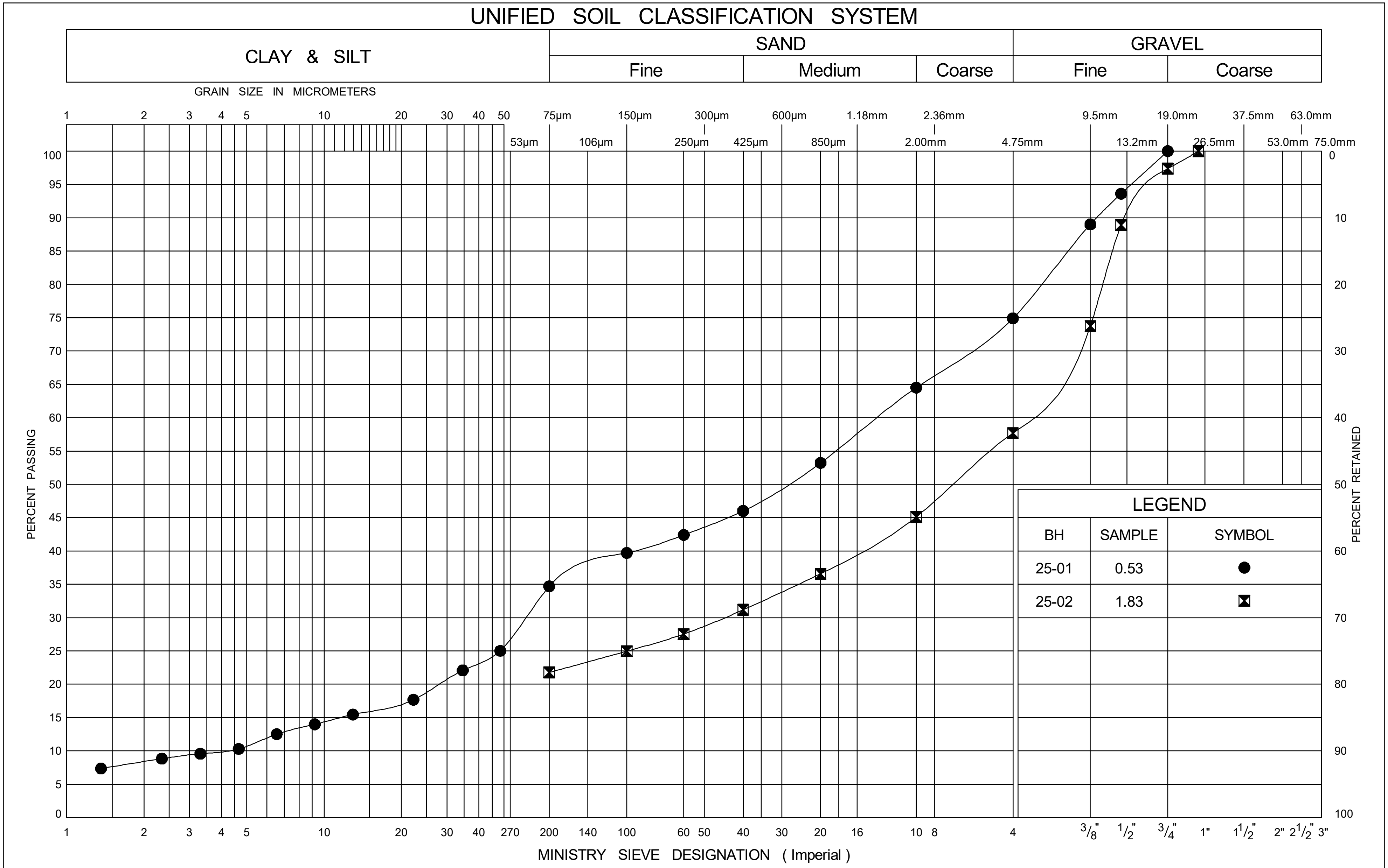
20 / 15 / 5 / 10 (%) STRAIN AT FAILURE

THURBER

APPENDIX E

Soil Laboratory Test Results

78 12 M



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GRAIN SIZE DISTRIBUTION

Sand Fill

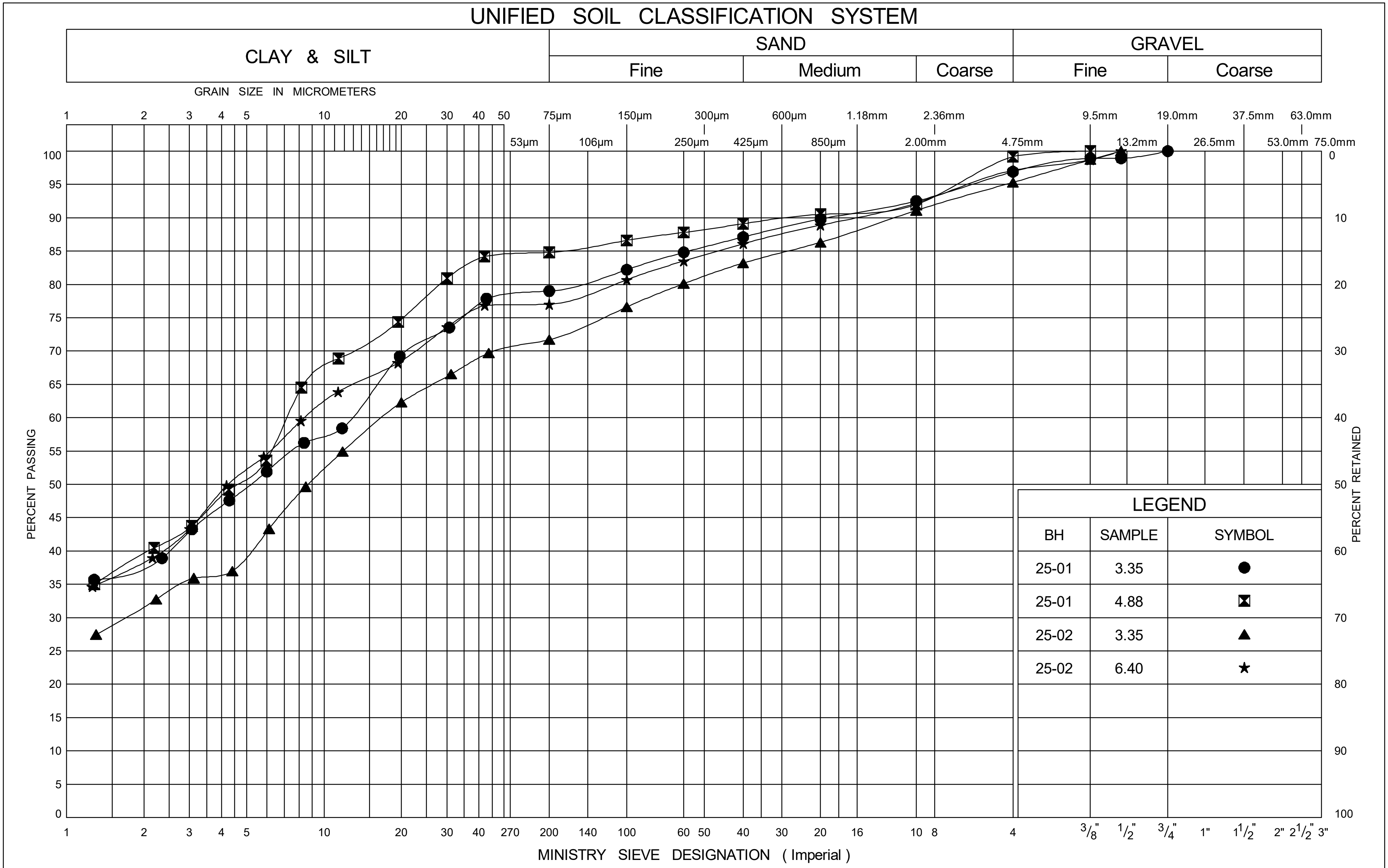
FIG No 1

W.P.

Casablanca Blvd/QEW Interchange

ONTARIO MOT GRAIN SIZE MTO-73956.GPJ ONTARIO MOT.GDT 12/9/25

78 12 M



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GRAIN SIZE DISTRIBUTION

Silty Clay Fill

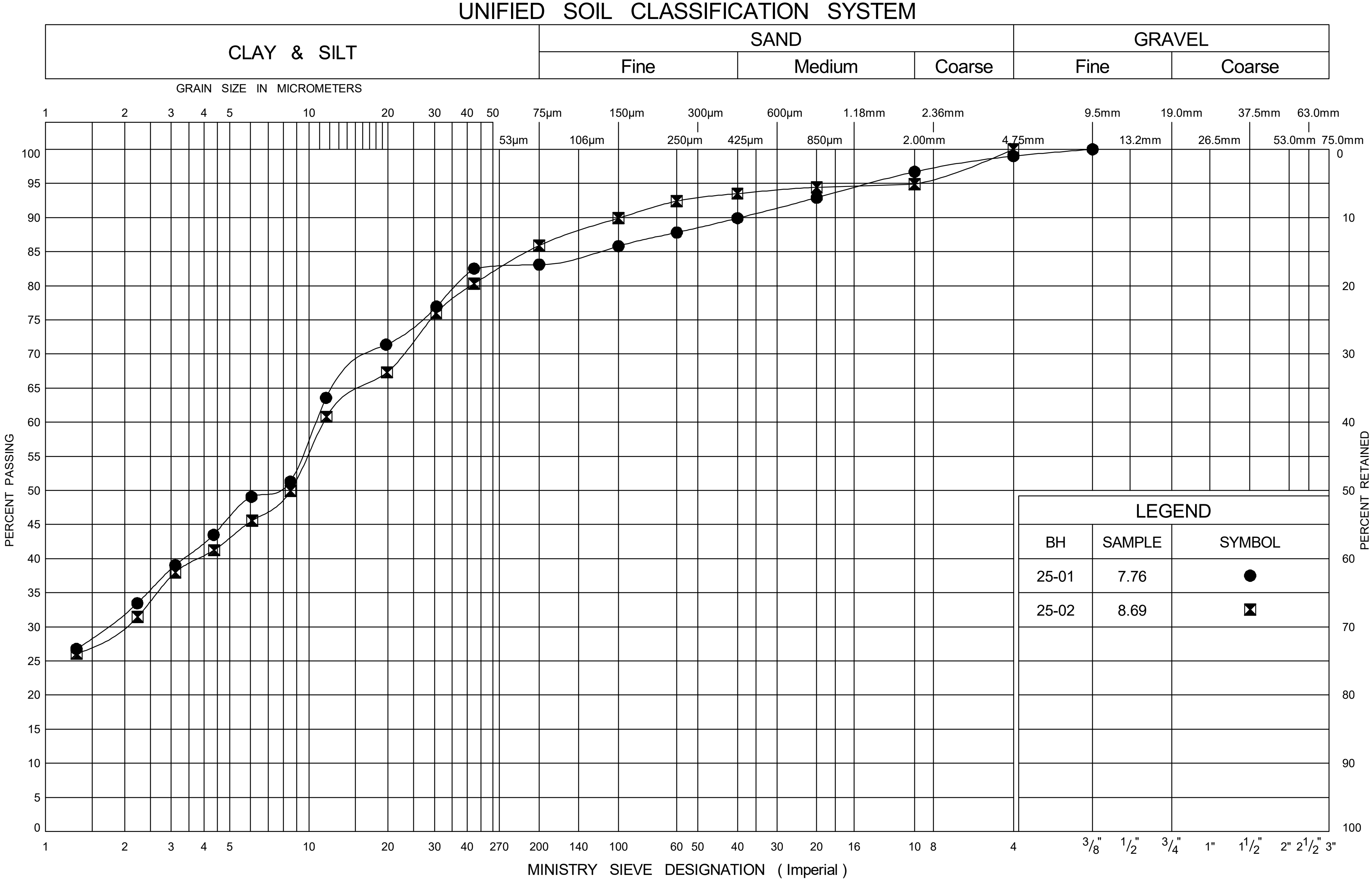
FIG No 2

W.P.

Casablanca Blvd/QEW Interchange

ONTARIO MOT GRAIN SIZE MTO-73956.GPJ ONTARIO MOT.GDT 12/9/25

78 12 M



LEGEND

BH	SAMPLE	SYMBOL
25-01	7.76	●
25-02	8.69	☒

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GRAIN SIZE DISTRIBUTION

Silty Clay

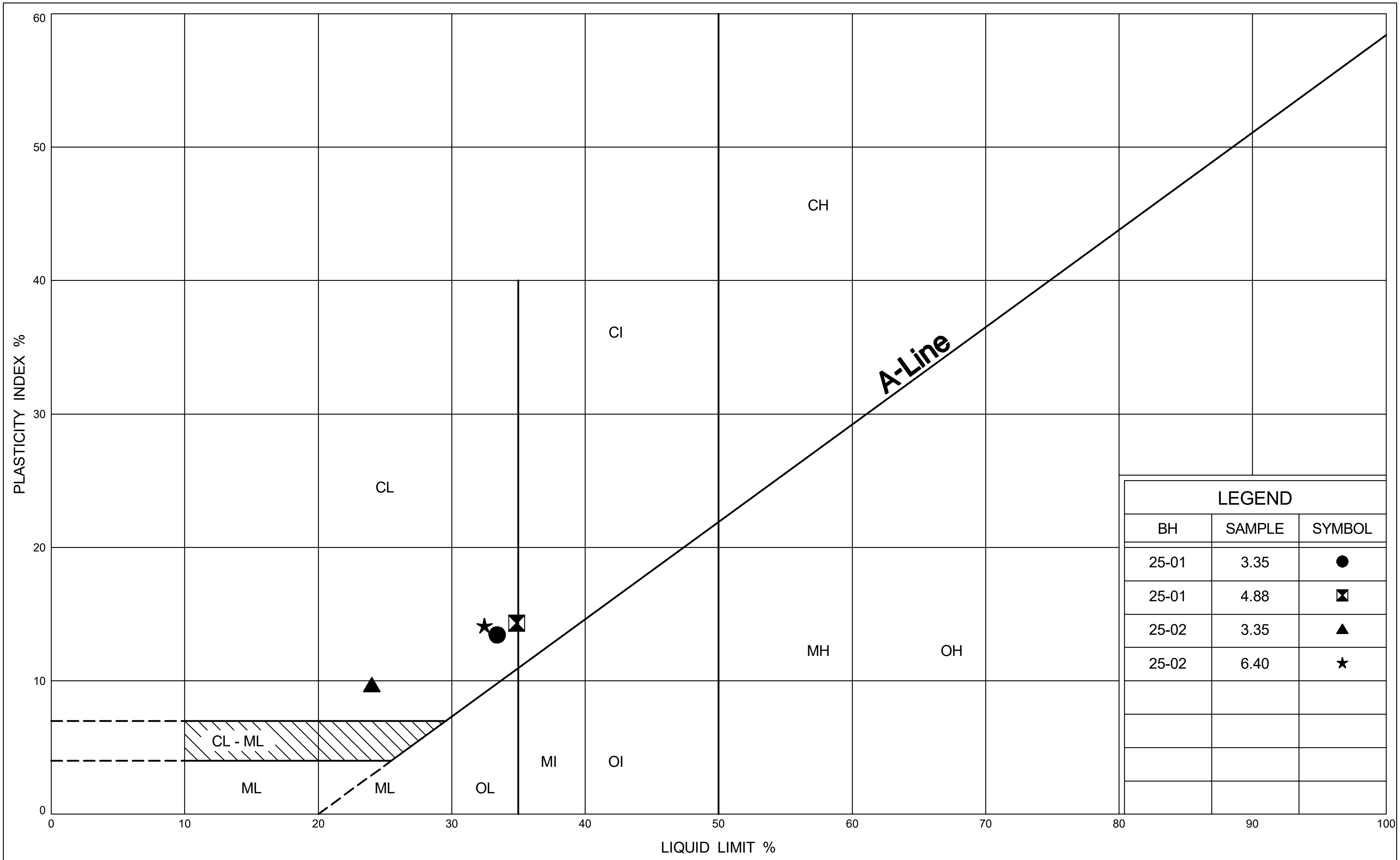
FIG No 3

W.P.

Casablanca Blvd/QEW Interchange

ONTARIO MOT GRAIN SIZE MTO-73956.GPJ ONTARIO MOT.GDT 12/9/25

Oct 75, FF - S - 21



LEGEND

BH	SAMPLE	SYMBOL
25-01	3.35	●
25-01	4.88	⧗
25-02	3.35	▲
25-02	6.40	★

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Ontario

PLASTICITY CHART

Silty Clay Fill

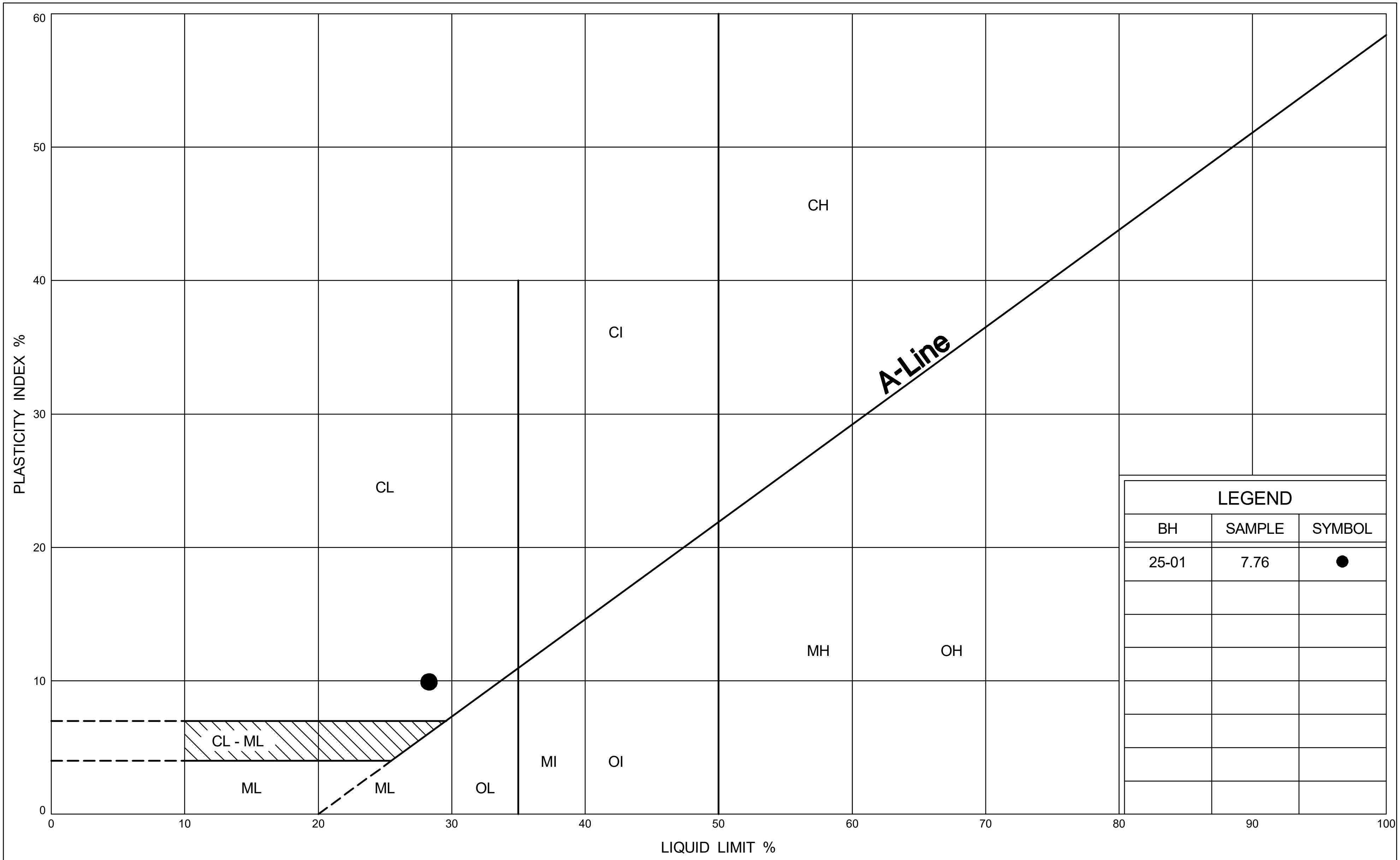
FIG No 4

W.P.

Casablanca Blvd/QEW Interchange

ONTARIO MOT PLASTICITY CHART MTO-73956.GPJ ONTARIO MOT.GDT 12/9/25

Oct 75, FF - S - 21



BH	SAMPLE	SYMBOL
25-01	7.76	●

ONTARIO MOT PLASTICITY CHART MTO-73956.GPJ ONTARIO MOT.GDT 12/9/25

Ministry of
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Ontario

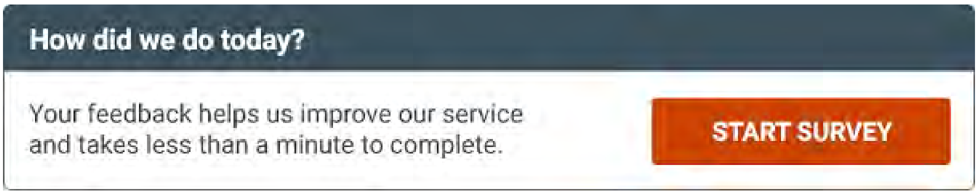
PLASTICITY CHART

Silty Clay

FIG No 5

W.P.

Casablanca Blvd/QEW Interchange


How did we do today?

Your feedback helps us improve our service and takes less than a minute to complete.

FINAL REPORT

CA40259-NOV25 R1

73956, C.asablanc.a Blvd, Grimsby, ON

Prepared for

Thurber Engineering Ltd.

FINAL REPORT

CA40259-NOV25 R1

First Page

CLIENT DETAILS

Client	Thurber Engineering Ltd.
Address	235 Martindale Road - Unit 10 St. Catharines, ON L2W 1A5. Canada
Contact	Madisan Chiarotto
Telephone	905-829-8666
Facsimile	
Email	mchiarotto@thurber.ca
Works #	
Project	73956, C.asablanc.a Blvd, Grimsby, ON
Reference	
Batch	
Samples	SOIL (2)

LABORATORY DETAILS

Project Specialist	Brad Moore Hon. B.Sc
Laboratory	SGS Canada Inc.
Address	185 Concession St., Lakefield ON, K0L 2H0
Telephone	705-652-2143
Facsimile	705-652-6365
Email	brad.moore@sgs.com
SGS Reference	CA40259-NOV25
Received	2025-11-25
Approved	12/01/2025
Report Number	CA40259-NOV25 R1
Date Reported	12/01/2025

COMMENTS

Temperature of Sample upon Receipt: 9 degrees C

Cooling Agent Present:Yes

Custody Seal Present:Yes

Chain of Custody Number:N/A

Corrosivity Index is based on the American Water Works Corrosivity Scale according to AWWA C-105.   An index greater than 10 indicates the soil matrix may be corrosive to cast iron alloys.

SIGNATORIES

Brad Moore Hon. B.Sc

TABLE OF CONTENTS

FINAL REPORT

CA40259-NOV25 R1

Client: Thurber Engineering Ltd.
Project: 73956, C.asablanc.a Blvd, Grimsby, ON
Project Manager: Madisan Chiarotto
Samplers: Bryan Ho

MATRIX: SOIL

			Sample Number	5	6
			Sample Name	BH25-01 Run#1 28'4"	BH25-02 27'6"-29'6"
			Sample Matrix	Soil	Soil
			Sample Date	2025-10-22 00:00	2025-10-21 00:00
Parameter	**Units**	**RL**		**Result**	**Result**
Corrosivity Index					
Corrosivity Index	none	1		2	18
pH	pH Units	0.05		7.98	7.81
Soil Redox Potential	mV	no		191	53
Sulphide (Na2CO3)	%	0.01		< 0.01	0.02
Resistivity (calculated)	ohms.cm	-9999		2950	433
General Chemistry					
Conductivity	uS/cm	2		339	2310
Metals and Inorganics					
Sulphate	µg/g	0.4		20	390
Other (ORP)					
Chloride	µg/g	0.4		230	1500

QC SUMMARY

Anions by IC

Method: EPA300/MA300-Ions1.3 | Internal ref.: ME-CA-[ENV]IC-LAK-AN-001

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Chloride	DIO0670-NOV25	µg/g	0.4	<0.4	3	35	102	80	120	106	75	125
Sulphate	DIO0670-NOV25	µg/g	0.4	<0.4	4	35	97	80	120	99	75	125

Carbon/Sulphur

Method: ASTM E1915-07A | Internal ref.: ME-CA-[ENV]ARD-LAK-AN-020

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Sulphide (Na2CO3)	ECS0123-NOV25	%	0.01	< 0.01								

Conductivity

Method: SM 2510 | Internal ref.: ME-CA-[ENV]EWL-LAK-AN-006

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Conductivity	EWL0567-NOV25	uS/cm	2	< 2	0	20	99	90	110	NA		

QC SUMMARY

pH

Method: SM 4500 | Internal ref.: ME-CA-[ENV]EWL-LAK-AN-001

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
pH	EWL0567-NOV25	pH Units	0.05	NA	0		100			NA		

Method Blank: a blank matrix that is carried through the entire analytical procedure. Used to assess laboratory contamination.

Duplicate: Paired analysis of a separate portion of the same sample that is carried through the entire analytical procedure. Used to evaluate measurement precision.

LCS/Spike Blank: Laboratory control sample or spike blank refer to a blank matrix to which a known amount of analyte has been added. Used to evaluate analyte recovery and laboratory accuracy without sample matrix effects.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate laboratory accuracy with sample matrix effects.

Reference Material: a material or substance matrix matched to the samples that contains a known amount of the analyte of interest. A reference material may be used in place of a matrix spike.

RL: Reporting limit

RPD: Relative percent difference

AC: Acceptance criteria

Multielement Scan Qualifier: as the number of analytes in a scan increases, so does the chance of a limit exceedance by random chance as opposed to a real method problem. Thus, in multielement scans, for the LCS and matrix spike, up to 10% of the analytes may exceed the quoted limits by up to 10% absolute and the spike is considered acceptable.

Duplicate Qualifier: for duplicates as the measured result approaches the RL, the uncertainty associated with the value increases dramatically, thus duplicate acceptance limits apply only where the average of the two duplicates is greater than five times the RL.

Matrix Spike Qualifier: for matrix spikes, as the concentration of the native analyte increases, the uncertainty of the matrix spike recovery increases. Thus, the matrix spike acceptance limits apply only when the concentration of the matrix spike is greater than or equal to the concentration of the native analyte.

LEGEND

FOOTNOTES

NSS	Insufficient sample for analysis.
RL	Reporting Limit.
↑	Reporting limit raised.
↓	Reporting limit lowered.
NA	The sample was not analysed for this analyte
ND	Non Detect

Results relate only to the sample tested.
Data reported represent the sample as submitted to SGS.

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Please refer to SGS General Conditions of Services located at http://www.sgs.com/terms_and_conditions.htm (Printed copies are available upon request.)

Test method information available upon request.
"Temperature Upon Receipt" is representative of the whole shipment and may not reflect the temperature of individual samples.

SGS Canada Inc. statement of conformity decision rule does not consider uncertainty when analytical results are compared to a specified standard or regulation.

-- End of Analytical Report --

SGS Environment, Health & Safety - Lakefield: 185 Concession St., Lakefield, ON K0L 2H0 Phone: 705-652-2000 Fax: 705-652-6365 Web: www.sgs.com/environment
- London: 657 Consortium Court, London, ON, N6E 2S8 Phone: 519-672-4500 Toll Free: 877-848-8060 Fax: 519-672-0361

Request for Laboratory Services and CHAIN OF CUSTODY

No:
Page _1_ of _1_

Laboratory Information Section - Lab use only

Received By: Ahmeer
Received Date: 11 / 25 / 25 (mm/dd/yy)
Received Time: 12 : 30 (hr : min)

Received By (signature): AA
Custody Seal Present: Yes ☑ No ☐
Custody Seal Intact: Yes ☑ No ☐

Cooling Agent Present: Yes ✓ ☐ No ☐ Type: ICEPACK
Temperature Upon Receipt (°C) 9 x 2

LAB LIMS #: CA40259Nov25

REPORT INFORMATION

Company: Thurber Engineering Ltd
Contact: Madisan Chiarotto
Address: 235 Martindale Rd, St. Catharines
Phone: 905-829-8666
Fax:
Email: mchiarotto@thurber.ca

INVOICE INFORMATION

☑ (same as Report Information)
Company:
Contact:
Address:
Phone:
Email: accountingON@thurber.ca

Quotation #:
P.O. #: 73956
Site Location/ID: Casablanca Blvd, Grimsby ON

TURNAROUND TIME (TAT) REQUIRED

☑ Regular TAT (5-7days)

TAT's are quoted in business days (exclude statutory holidays & weekends). Samples received after 6pm or on weekends: TAT begins next business day

RUSH TAT (Additional Charges May Apply): ☐ 1 Day ☐ 2 Days ☐ 3 Days ☐ 4 Days

PLEASE CONFIRM RUSH FEASIBILITY WITH SGS REPRESENTATIVE PRIOR TO SUBMISSION

Specify Due Date:

***NOTE:** DRINKING (POTABLE) WATER SAMPLES FOR HUMAN CONSUMPTION MUST BE SUBMITTED WITH SGS DRINKING WATER CHAIN OF CUSTODY

REGULATIONS

☑ **O.Reg 153/04** ☑ **O.Reg 406/19**
☑ Table 1 ☑es/Park Soil Texture:
☐ Table 2 ☐d/Com ☑parse
☐ Table 3 ☐gri/Other ☐edium/Fine
☐ Table ____
Soil Volume ☐350m3 ☑ >350m3

Other Regulations:
☐Reg 347/558 (3 Day min TAT)
☐PWQO ☐MER
☐CCME ☐ther:
☐MISA
☐PDWS Not Reportable *See note

Sewer By-Law:
☐anitary
☐orm
Municipality:

RECORD OF SITE CONDITION (RSC) ☐YES ☑NO

ANALYSIS REQUESTED

						M & I				SVOC		PCB	PHC		VOC		Pest	Other (please specify)							TCLP	
	SAMPLE IDENTIFICATION	DATE SAMPLED	TIME SAMPLED	# OF BOTTLES	MATRIX	Field Filtered (Y/N)	Metals & Inorganics incl CrVI, CN,Hg pH,(B(HWS),EC,SAR-soil) (Cl, Na-water)	Full Metals Suite ICP metals plus B(HWS-soil only) Hg, CrVI	ICP Metals only Sb,As,Ba,Be,B,Cd,Cr,Co,Cu,Pb,Mo,Ni,	PAHs only	SVOCs all incl PAHs, ABNs, CPs	PC☑s Total ☐ Aroclor	F1-F4 + BTEX	F1-F4 only no BTEX	VOCs all incl BTEX	BTEX only	☐ Pesticides Organochlorine or specify other	Corrosivity Package	OCPs	D&F		Appendix 2: 406/19 Leachate Screening Levels Table :	Sewer Use: Specify pkg:	Water Characterization Pkg General Extended ☐	Specify TCLP tests ☐M&I ☐VOC ☐PCB ☐B(a)P ☐ABN ☐Ignit.	COMMENTS:
1	BH25-01 RUN#1 28'4"	22-Oct-25	AM	1	Weak Shale													x								
2	BH25-02 27'6"-29'6"	21-Oct-25	AM	1	Soil													x								
3																										
4																										
5																										
6																										
7																										
8																										
9																										
10																										
11																										
12																										

Observations/Comments/Special Instructions

Sampled By (NAME): Bryan Ho
Signature:
Date: ___/___/___ (mm/dd/yy)
Pink Copy - Client

Relinquished by (NAME):
Signature:
Date: ___/___/___ (mm/dd/yy)
Yellow & White Copy - SGS

Revision #: 1.4
Date of Issue: 22 May. 2020

Note: Submission of samples to SGS is acknowledgement that you have been provided direction on sample collection/handling and transportation of samples. {2} Submission of samples to SGS is considered authorization for completion of work. Signatures may appear on this form or be retained on file in the contract, or in an alternative format (e.g. shipping documents). {3} Results may be sent by email to an unlimited number of addresses for no additional cost. Fax is available upon request. This document is issued by the Company under its General Conditions of Service accessible at http://www.sgs.com/terms_and_conditions.htm. (Printed copies are available upon request.) Attention is drawn to the limitation of liability, indemnification and jurisdiction issues defined therein.

APPENDIX F

Rock Laboratory Test Results

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No: 73956
Client: NA
Project Name: Casablanca Blvd/QEW
Core Size: HQ **BH No :** BH25-01
Date Drilled: 21-Oct-25
Date Tested: 23-Oct-25
Tester: GA
Reviewed by: GL

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	8.9	A	1.9	63.0	67.8	0.4	9.6	Shale w Siltstone	Weak
2	1	9.4	A	4.6	62.9	69.4	0.9	22.4	Shale	Weak
3	2	10.0	A	5.0	63.0	68.6	1.0	24.7	Shale	Weak
4	2	10.3	A	2.9	62.9	69.0	0.6	14.4	Shale	Weak
5	2	10.6	A	4.2	62.9	68.2	0.9	20.9	Shale	Weak
6	3	11.1	A	5.0	62.8	66.3	1.1	25.5	Shale	Medium Strong
7	3	11.5	A	3.4	62.8	63.4	0.7	17.6	Shale w Siltstone	Weak
8	3	11.8	A	2.9	62.9	74.0	0.6	13.7	Shale	Weak
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35										

* 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.

THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET

ASTM D5731-08

Job No:	73956	**Date Drilled:**	19-Oct-25
Client:	NA	**Date Tested:**	23-Oct-25
Project Name:	Casablanca Blvd/QEW	**Tester:**	GA
Core Size:	HQ **BH No :** BH25-02	**Reviewed by:**	GL

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	1.0	62.9	63.0	0.2	5.1	Shale	Weak
2	2	10.5	A	2.0	62.9	65.3	0.4	10.5	Shale	Weak
3	2	10.9	A	1.9	62.9	64.3	0.4	10.1	Shale	Weak
4	2	12.0	A	2.6	62.9	69.3	0.5	12.5	Shale	Weak
5	Could not test any sample from run 3, No sample obtained in run 4									
6	5	14.1	A	1.5	60.3	67.2	0.3	7.8	Shale	Weak
7	5	14.7	A	2.9	60.2	59.0	0.7	16.7	Shale	Weak
8	5	15.0	A	2.8	60.3	59.9	0.6	15.6	Shale	Weak
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35										

* 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.

GEOMECHANICA

Geomechanica Inc.
#14-1240 Speers Rd.
Oakville ON
L6L 2X4 Canada

November 11, 2025

Madisan Chiarotto
Thurber Engineering Ltd.
202 – 1908 Ironoak Way
Oakville, Ontario
Canada, L6H 0N1

Re: UCS Testing
(Thurber Engineering Ltd. Project No. 73956)

Dear Madisan Chiarotto:

On October 23, 2025 a series of 2 core samples (HQ-sized) were received by Geomechanica Inc. via drop-off by Thirber personnel. These samples were identified as being from Thurber Engineering Ltd. Project No. 73956. From these samples, 2 Uniaxial Compressive Strength (UCS) tests were completed.

Details regarding the steps of specimen preparation and testing along with the test results are presented in the accompanying laboratory report and summary spreadsheet.

Sincerely,

Bryan Tatone, PhD, PEng

Geomechanica Inc.
Tel: +1-647-478-9767
lab@geomechanica.com

Rock Laboratory Testing Results

A report submitted to:

Madisan Chiarotto
Thurber Engineering Ltd.
202 – 1908 Ironoak Way
Oakville, Ontario
Canada, L6H 0N1

Prepared by:

Bryan Tatone, PhD, PEng

Omid Mahabadi, PhD, PEng
Geomechanica Inc.
#14-1240 Speers Rd.
Oakville ON
L6L 2X4 Canada
Tel: +1-647-478-9767
lab@geomechanica.com

November 11, 2025
Project number: 73956

Abstract

This document summarizes the results of 2 Uniaxial Compressive Strength (UCS) tests. The UCS values along with photographs of samples before and after testing are presented herein.

In this document:

1 Uniaxial Compressive Strength Tests

1.1 Overview

This section summarizes the results of uniaxial compressive strength (UCS) testing. The testing was performed in Geomechanica Inc.'s rock testing laboratory using a 150 ton (1.3 MN) Forney loading frame equipped with pressure-compensated control valve to maintain an axial strain rate of approximately 0.15 mm/min (Figure 1). The preparation and testing procedure for each specimen included the following:

1. Unwrapping the core sample, inspecting it for damage, and re-wrapping it in electrical tape to minimize exposure to moisture during subsequent specimen preparation.
2. Diamond cutting the core sample to obtain a cylindrical specimen with an appropriate length (length:diameter = 2:1) and nearly parallel end faces.
3. Diamond grinding the specimen to obtain flat (within ±0.025 mm) and parallel end faces (within 0.25°).
4. Placing the specimen into the loading frame, applying a 1 kN axial load, and removing the electrical tape.
5. Axially loading the specimens to rupture while continuously recording axial force to determine the peak strength (UCS).

Figure 1: Forney loading frame setup for UCS testing.

Using a precision V-block mounted on the magnetic chuck of the surface grinder, test specimens met the end flatness, end parallelism, and perpendicularity criteria set out in ASTM D4543-19. The side straightness criteria, as checked with a feeler gauge, and the minimum length:diameter criteria were met for all specimens unless noted otherwise in Table 1. Testing of the specimens followed ASTM D7012-14 Method C.

1.2 Results

The results of UCS testing are summarized in Table 1. Additional specimen and test details are provided on the summary spreadsheet that accompanies this report.

Table 1: Summary of Uniaxial Compression test results.

Sample	Depth (m)	Bulk density ρ (g/cm^3)	UCS (MPa)	Lithology	Failure description
BH25-01, Run 2	9.60 - 9.91	2.585	36.3	Red Shale	1
BH25-02, Run 2	11.35 - 11.63	2.611	6.7	Red Shale	2, 3

[1] Inclined shear failure
[2] Axial splitting failure
[3] Specimen emitted pore water upon loading

1.3 Specimen photographs

Photographs of the specimens before and after testing are presented in the Appendix of this report.

Appendices

Specimen sheets

- BH25-01, Run 2
- BH25-02, Run 2

Uniaxial Compression Test

Client	Thurber Engineering Ltd.	**Project**	73956
Sample	BH25-01, Run 2	**Depth**	9.60 - 9.91

Specimen parameters

Diameter (mm) [a]	62.45
Length (mm) [a]	127.01
L/D ratio	2.03
Bulk density ρ (g/cm^3)	2.585
UCS (MPa)	36.3
Lithology	Red Shale
Failure description [b]	1

[a] Additional specimen measurement/details provided in accompanying summary spreadsheet.
[b] Failure description: [1] Inclined shear failure;

Prior to testing

After testing

Remarks:

Performed by	JNO	**Date**	2025-11-10

Uniaxial Compression Test

Client	Thurber Engineering Ltd.	**Project**	73956
Sample	BH25-02, Run 2	**Depth**	11.35 - 11.63

Specimen parameters

Diameter (mm) [a]	62.59
Length (mm) [a]	126.63
L/D ratio	2.02
Bulk density ρ (g/cm^3)	2.611
UCS (MPa)	6.7
Lithology	Red Shale
Failure description [b]	2, 3

[a] Additional specimen measurement/details provided in accompanying summary spreadsheet.

[b] Failure description: [2] Axial splitting failure; [3] Specimen emitted pore water upon loading;

Prior to testing

After testing

Performed by	JNO	**Date**	2025-11-10

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APPENDIX G

Rock Core Photograph

Photographs of Rock Core

Borehole 25-01

Borehole 25-01 – Run 1 – 8.08 m to 9.60 m

Borehole 25-01 – Run 2 – 9.60 to 10.97 m

Borehole 25-01 – Run 3 – 10.97 m to 12.50 m

Borehole 25-02

Borehole 25-02 – Run 1 – 9.35 m to 10.52 m

Borehole 25-02 – Run 2 – 10.52 m to 12.04 m

Borehole 25-02 – Run 3 – 12.04 m to 12.22 m

Borehole 25-02 – Run 5 – 13.74 m to 15.24 m

APPENDIX H

General Arrangement Drawing (February 2026)

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

CONT. No. 2013–2040
GWP No. 2191–10–00

Q.E.W.
CASABLANCA BLVD. UNDERPASS
REHABILITATION

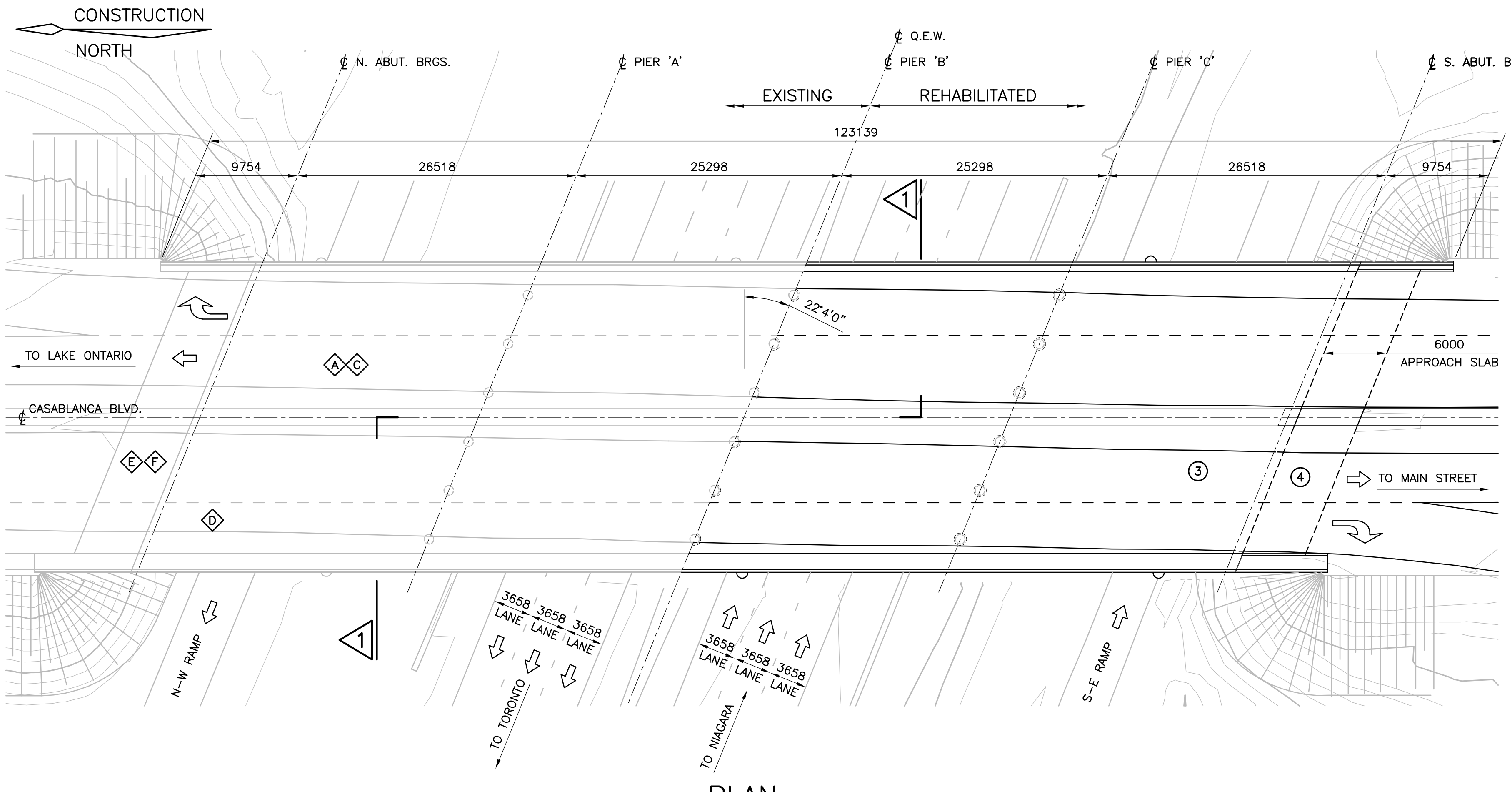
GENERAL ARRANGEMENT

SHEET
57

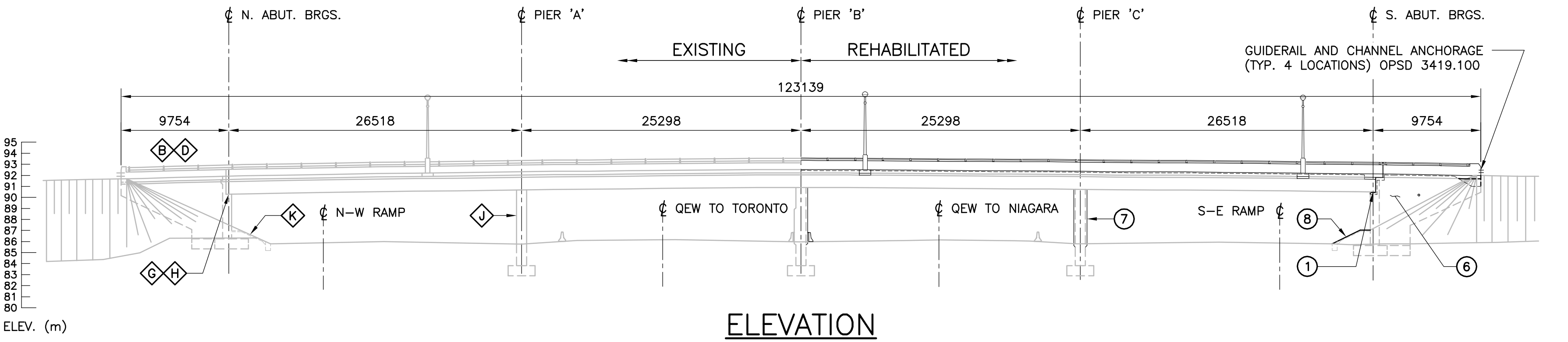
OCT 25/2013
Y.B. CHI
100088366
R. Chi

OCT 25/2013
T. THARMABALA
46042115
T. Tharmabala

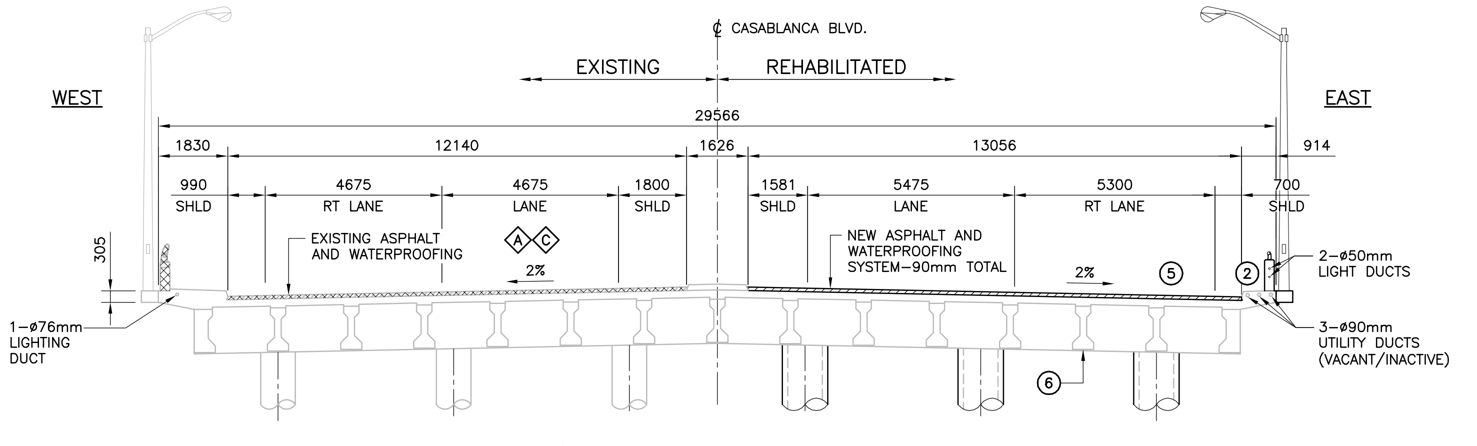
McINTOSH PERRY



PLAN
1 : 300



ELEVATION
1 : 300



1 DECK SECTION
1 : 100

LEGEND:

▩	ASPHALT AND CONCRETE REMOVAL
▨	NEW ASPHALT
░	NEW CONCRETE

SCOPE OF REHABILITATION WORK:

THE GENERAL SCOPE OF THE REHABILITATION WORK OUTLINED BELOW AND DESIGNATED ON THIS DRAWING SHALL BE CONSIDERED SIMILAR EACH SIDE OF THE CENTRELINE OF THE BRIDGE, UNLESS NOTED OTHERWISE.

REMOVALS:

A REMOVE EXISTING ASPHALT AND WATERPROOFING FROM DECK.
B REMOVE PORTION OF THE EXISTING CURB, CONCRETE PARAPET WALL, RAILING AND LIGHT POLE BASE AT BOTH SIDES OF THE BRIDGE.
C REMOVE DELAMINATED AND DETERIORATED CONCRETE FROM DECK SURFACE, SIDEWALK AND MEDIAN.
D REMOVE END PORTION OF THE DECK AT ABUTMENTS INCLUDING EXPANSION JOINTS.
E REMOVE APPROACH SLABS INCLUDING EXISTING PAVING. INSTALL PROTECTION SYSTEM AS REQUIRED.
F EXCAVATE BEHIND BALLAST WALLS AND REMOVE BALLAST WALLS INCLUDING EXPANSION JOINT ASSEMBLY.
G JACK UP THE BRIDGE AT ABUTMENTS AND REMOVE EXISTING BEARINGS AT ABUTMENTS.
H REMOVE CONCRETE AT BEARING SEATS AT ABUTMENTS.
I REMOVE DETERIORATED CONCRETE FROM DECK SOFFIT, DECK FASCIA, GIRDERS, ABUTMENTS AND WINGWALLS.
J REMOVE ALL EXISTING CONCRETE ENCASEMENT AND DELAMINATED CONCRETE ON PIER COLUMNS.
K REMOVE DAMAGED SLOPE PAVING AT ABUTMENTS.

- OTHER AS SHOWN AND/OR SPECIFIED IN THE CONTRACT.
- NOT INTENDED SEQUENCE OF REMOVALS.

NEW CONSTRUCTION:

① RECONSTRUCT BEARING SEATS AND INSTALL NEW BEARINGS AT ABUTMENTS.
② REPAIR CONCRETE DECK SURFACE, CONSTRUCT NEW PARAPET WALLS AND LIGHT POLE BASE AT BOTH SIDES OF THE BRIDGE.
③ CONSTRUCT THE NEW SEMI–INTEGRAL DECK EXTENSIONS AT THE ABUTMENTS.
④ CONSTRUCT NEW APPROACH SLABS.
⑤ WATERPROOF AND PAVE THE DECK AND APPROACH SLABS TO THE LIMITS SHOWN ON THE DRAWINGS.
⑥ REPAIR DETERIORATED AREAS AT THE SIDEWALK, MEDIAN, DECK SOFFIT, DECK FASCIA, GIRDERS, ABUTMENTS, AND WINGWALLS.
⑦ ENCASE ALL COCUMNS.
⑧ REPAIR SLOPE PAVING AT ABUTMENTS.

- OTHER AS SHOWN AND/OR SPECIFIED IN THE CONTRACT.
- NOT INTENDED SEQUENCE OF NEW CONSTRUCTION.

LIST OF DRAWINGS:

R1–1	GENERAL ARRANGEMENT
R1–2	CONSTRUCTION STAGING
R1–3	JACKING AND BEARING DETAILS
R1–4	ABUTMENT REMOVAL
R1–5	ABUTMENT RECONSTRUCTION – I
R1–6	ABUTMENT RECONSTRUCTION – II
R1–7	PIER ENCASEMENT DETAILS
R1–8	DECK END REMOVAL
R1–9	DECK END RECONSTRUCTION – I
R1–10	DECK END RECONSTRUCTION – II
R1–11	EAST LIGHT POLE BASE REMOVALS AND RECONSTRUCTION
R1–12	WEST LIGHT POLE BASE REMOVALS AND RECONSTRUCTION
R1–13	APPROACH SLAB
R1–14	PARAPET WALL WITH S/W AND RAILING PL2 – STAINLESS STEEL REBAR
R1–15	RAILING FOR PARAPET WALL
R1–16	STANDARD DETAILS – I
R1–17	STANDARD DETAILS – II

LIST OF ABBREVIATIONS:

T/D	DENOTES TOP OF CONCRETE DECK
EL.	DENOTES ELEVATION
BRGS.	DENOTES BEARINGS
ABUT.	DENOTES ABUTMENT
TYP.	DENOTES TYPICAL
WP.	DENOTES WORKING POINT
STA.	DENOTES STATION
NTS	DENOTES NOT TO SCALE
EXP	DENOTES EXPANSION
DWG	DENOTES DRAWING
EXT	DENOTES EXISTING
CA	DENOTES CONTRACT ADMINISTRATOR
C.J.	DENOTES CONSTRUCTION JOINT
F.F.	DENOTES FRONT FACE
B.F.	DENOTES BACK FACE

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

GENERAL NOTES:

CLASS OF CONCRETE :

- ALL CONCRETE 30MPa

CLEAR COVER TO REINFORCING STEEL

- DECK . . . . TOP 70 ± 20mm
  BOTTOM 50 ± 10mm
- REMAINDER 70 ± 20mm UNLESS OTHERWISE NOTED

REINFORCING STEEL:

- REINFORCING STEEL SHALL BE GRADE 400W UNLESS OTHERWISE SPECIFIED.
- BAR MARKED WITH PREFIX 'S' DENOTES STAINLESS STEEL BARS.
- STAINLESS REINFORCING STEEL BARS SHALL BE TYPE 316LN OR DUPLEX 2205 AND HAVE A MINIMUM YIELD STRENGTH OF 500 MPa.
- UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES SHALL BE CLASS B.
- BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS, WHILE STIRRUP AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWING SS12–1, UNLESS INDICATED OTHERWISE.

CONSTRUCTION NOTES:

- THE CONTRACTOR SHALL VERIFY ALL RELEVANT DIMENSIONS, ELEVATIONS, STATIONS AND DETAILS ON SITE AND REPORT ANY DISCREPANCIES TO THE CONTRACT ADMINISTRATOR PRIOR TO PROCEEDING WITH REHABILITATION WORK.
- THE CONTRACTOR SHALL ESTABLISH THE BEARING SEAT ELEVATIONS BY DEDUCTING THE ACTUAL BEARING THICKNESS FROM THE TOP OF BEARING ELEVATIONS. IF THE ACTUAL BEARING THICKNESS ARE DIFFERENT FROM THOSE GIVEN WITH THE BEARING DESIGN DATA, THE CONTRACTOR SHALL MAKE ADJUSTMENTS TO SUIT.
- TYPICAL AREAS OF REPAIRS ARE INDICATED ON THE DRAWINGS WHERE REPAIR LIMITS ARE NOT SHOWN, LIMITS SHALL BE INDICATED BY THE CONTRACT ADMINISTRATOR.
- SAWCUT IN CONCRETE, WHERE DESIGNATED SHALL BE 25mm DEEP OR TO THE FIRST LAYER OF REINFORCING STEEL, WHICHEVER IS LESS. SAWCUT SURFACES AGAINST WHICH NEW CONCRETE WILL BE PLACED SHALL BE ROUGHENED.
- PROTECTION SYSTEM SHALL MEET REQUIREMENTS FOR PERFORMANCE LEVEL 2 AS SPECIFIED IN THE CONTRACT DOCUMENTS.
- THE CONTRACTOR SHALL PROVIDE DEBRIS PLATFORM AND NECESSARY CONTAINMENT MEASURES TO COLLECT FALLING CONCRETE AND CONSTRUCTION DEBRIS SUCH THAT NO DEBRIS OR MATERIALS RESULTING FROM THE REMOVAL WORK FALL ON ANY ROADWAY AND OTHER AREAS BELOW THE BRIDGE.

APPLICABLE STANDARD DRAWINGS:

OPSD 912.430 –	GUIDE RAIL SYSTEM, STEEL BEAM STRUCTURE CONNECTION
OPSD 3370.100 –	DECK, WATERPROOFING HOT APPLIED ASPHALT MEMBRANE WITH PROTECTION BOARD
OPSD 3370.101 –	DECK, WATERPROOFING HOT APPLIED ASPHALT MEMBRANE AT ACTIVE CRACKS GREATER THAN 2mm WIDE AND CONSTRUCTION JOINTS
OPSD 3390.100 –	DECK DRIP CHANNEL
OPSD 3390.150 –	FALSEWORK CLEARANCE TO TRAFFIC LANES
OPSD 3419.100 –	BARRIERS AND RAILINGS – STEEL GUIDE RAIL AND CHANNEL ANCHORAGE
OPSD 3941.200 –	FIGURES IN CONCRETE – SITE NUMBER AND DATE LAYOUT
OPSD 3950.100 –	JOINTS – CONCRETE EXPANSION AND CONSTRUCTION ON STRUCTURE

REVISIONS
DESCRIPTION

DESIGN YC	CHK TT	CODE CHBDC–2006	LOAD CL–625–ONT	DATE OCT/13
DRAWN MC	CHK TT	SITE 18–196	R1	DWG R1–1

MINISTRY OF TRANSPORTATION, ONTARIO PR–D–707 88–05

CAD FILE LOCATION AND NAME: D:\WORK\KM–12–637 – MTO – CR – QEW 8 Bridge Rehab\04 CASABLANCA BLVD UNDERPASS\12–637–04–001GA.dwg
MODIFIED: 10/22/2013 11:53:04 AMBY: M.CAO
DATE PLOTTED: 10/25/2013 12:04:10 PMBY: MING CAO

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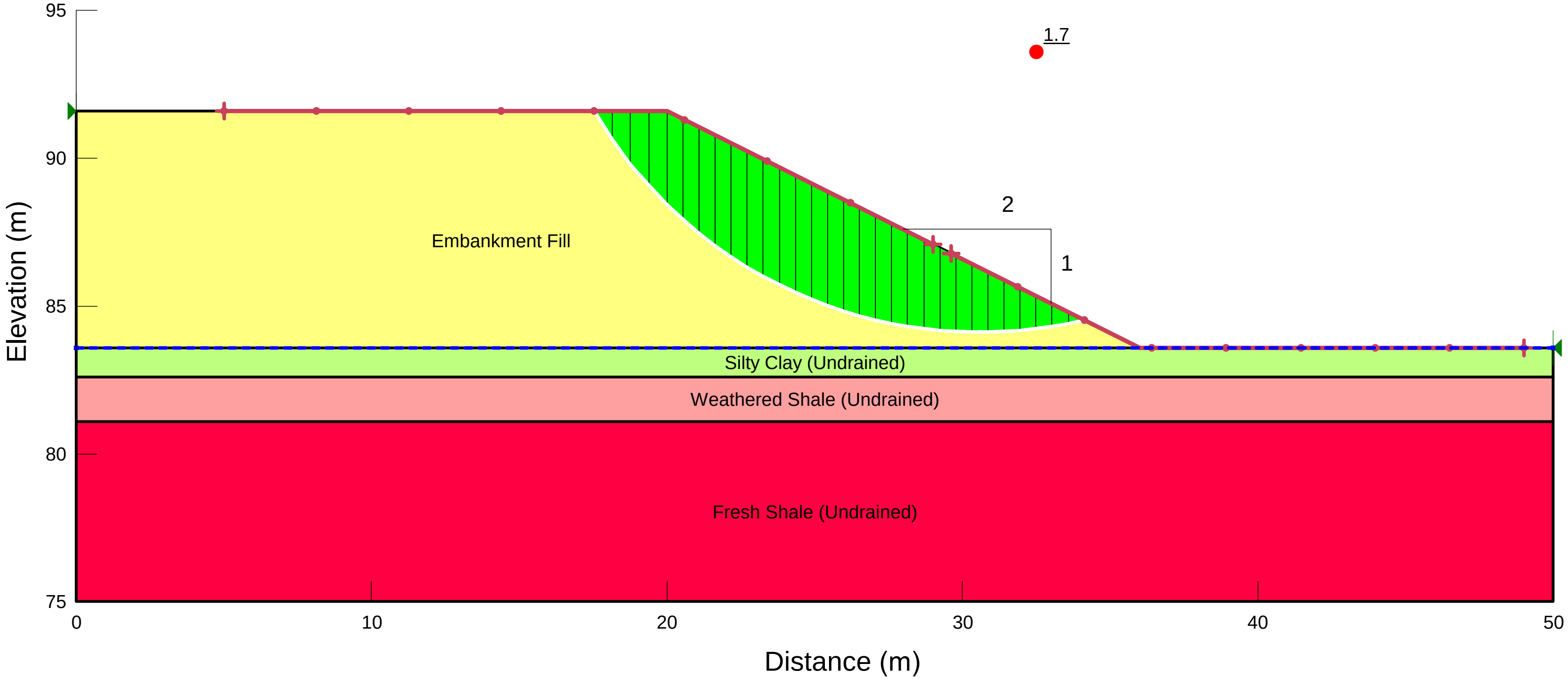
APPENDIX I

Slope Stability Figures

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
🟨	Embankment Fill	Mohr-Coulomb	21	0	32
🟥	Fresh Shale (Undrained)	Mohr-Coulomb	22	3,350	0
🟩	Silty Clay (Undrained)	Mohr-Coulomb	20	50	0
■	Weathered Shale (Undrained)	Mohr-Coulomb	21	100	0



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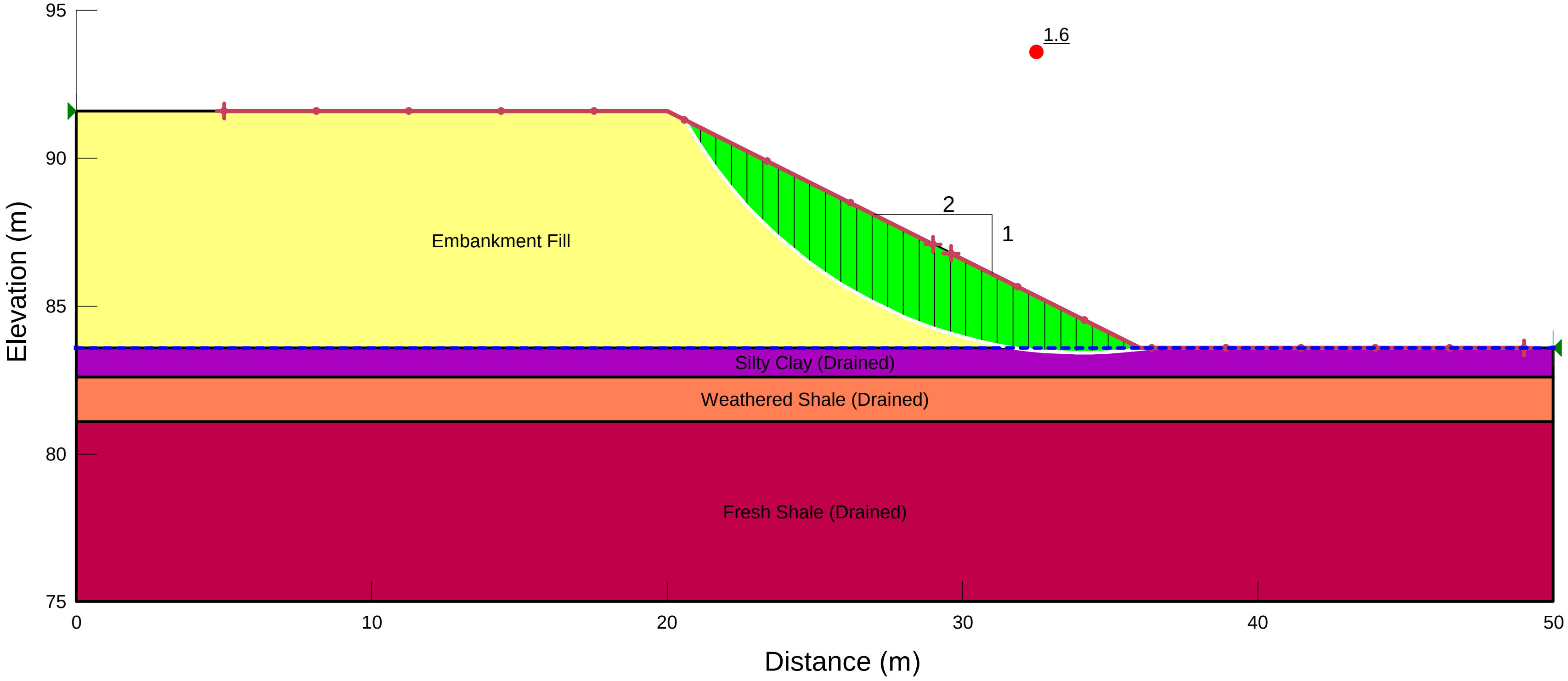
Project	73956 - Casablanca Blvd QEW Interchange		
Analysis	SLOPE/W Analysis - Undrained		Figure I1
Seismic Coefficient H: g, V: g	Last Run 05/04/2026, 11:40:28 AM	Scale 1:235	

Additional Details

Name: 2D Geometry
Comments:
Method: Morgenstern-Price, Half-Sine
Entry: (17.531153, 91.6) m, Exit: (34.120868, 84.539566) m
Center: (30.581519, 99.243674) m, Radius: 15.12408 m

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
🟨	Embankment Fill	Mohr-Coulomb	21	0	32
🟥	Fresh Shale (Drained)	Mohr-Coulomb	22	10	38
🟪	Silty Clay (Drained)	Mohr-Coulomb	20	5	30
🟧	Weathered Shale (Drained)	Mohr-Coulomb	21	5	30



Project 73956 - Casablanca Blvd QEW Interchange

Analysis SLOPE/W Analysis - Drained

Figure I2

Seismic Coefficient H: g, V: g

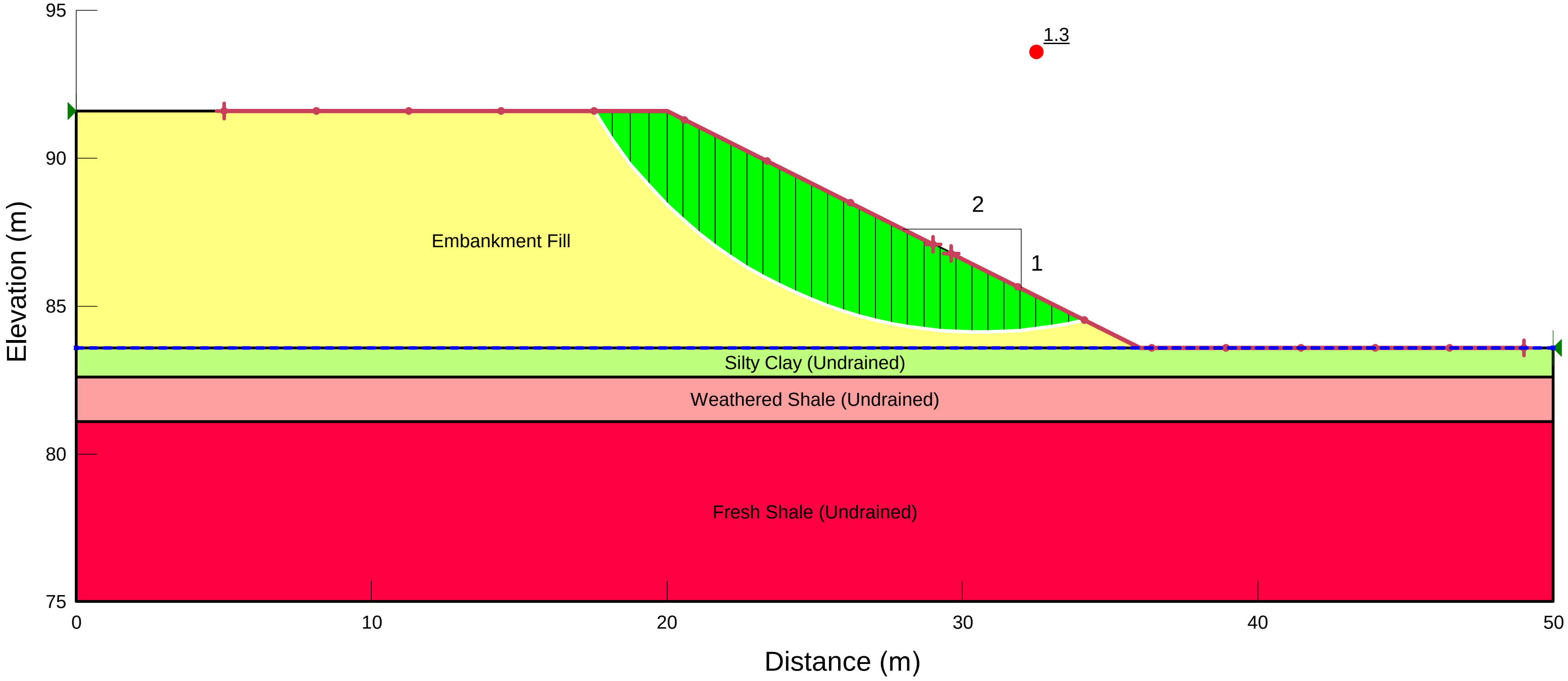
Last Run 05/04/2026, 11:40:28 AM

Scale 1:235

Additional Details

Name: 2D Geometry
Comments:
Method: Morgenstern-Price, Half-Sine
Entry: (20.593847, 91.303076) m, Exit: (36.415889, 83.6) m
Center: (34.061619, 98.865049) m, Radius: 15.445527 m

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
🟨	Embankment Fill	Mohr-Coulomb	21	0	32
🟥	Fresh Shale (Undrained)	Mohr-Coulomb	22	3,350	0
🟩	Silty Clay (Undrained)	Mohr-Coulomb	20	50	0
🟥	Weathered Shale (Undrained)	Mohr-Coulomb	21	100	0



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Project 73956 - Casablanca Blvd QEW Interchange

Analysis SLOPE/W Analysis - Seismic **Figure I3**

Seismic Coefficient H: 0.12g, V: g

Last Run 05/04/2026, 11:40:28 AM

Scale 1:235

Additional Details

Name: 2D Geometry
Comments:
Method: Morgenstern-Price, Half-Sine
Entry: (17.531153, 91.6) m, Exit: (34.120868, 84.539566) m
Center: (30.581519, 99.243674) m, Radius: 15.12408 m