



**PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
for Design-Build Ready Alternative Bid Package**

for

WIDENING OF BAPTISTE CREEK BRIDGES

Site Nos. 13X-0188/B1 & B2

Highway 401 – Station 22+065

Township of Tilbury, Chatham-Kent, Ontario

GWP 3034-19-00, WP 3232-19-01 & 3233-19-01

Assignment No. 3017-E-0006/0007

Work Item No. 07

Latitude and Longitude: 42.278908, -82.381596

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PML Ref.: 19KF030A
Index No.: 023FIR and 024FDR
GEOCRES No.: 40J8-71
December 16, 2019



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**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
for Design-Build Ready Alternative Bid Package**

Widening of Baptiste Creek Bridges
Site No. 13X-0188/B1 & B2
Highway 401 –Station 22+065
Township of Tilbury, Chatham-Kent, Ontario
G.W.P. 3034-19-00, Assignment No. 3017-E-0006/0007, Work Item No. 07

PART A – PRELIMINARY FOUNDATION INVESTIGATION PORTION OF THE REPORT

1. INTRODUCTION

The Ministry of Transportation Ontario (MTO) has retained WSP as the Prime Consultant, to provide services for the widening of EBL and WBL structures at three sites on Highway 401 under the request for proposal (RFP) for MTO Assignment No. 3017-E-0006/0007, Work Item No. 07. WSP retained Peto MacCallum Ltd. (PML) on behalf of MTO to provide foundation engineering services for this assignment. The Terms of Reference and Scope of Work for the Foundation Engineering services are outlined in the RFP for MTO Assignment No. 3017-E-0006/0007, Work Item No. 07.

This report is a Preliminary Foundation Investigation and Design Report for Design-Build Ready Alternative Bid Package for Baptiste Creek EBL and WBL Bridges located along Highway 401 at the crossing of Baptiste Creek in the Township of Tilbury, Chatham-Kent, Ontario. The subsurface investigation data was limited to previously available boreholes from a previously investigation supplemented by two (2) additional boreholes specifically drilled for the current assignment. Accordingly, further investigation will be required during detail design to establish or confirm / reassess the recommendations provided in this report.

2. SITE DESCRIPTION

Highway 401 in the area of the bridge site is elevated slightly above the natural topography, and accommodates two (2) lanes of vehicular traffic in each direction. The site is generally a flat area, with the exception of the highway embankments. The study area is surrounded by agricultural developments, and is located approximately 4.5 km east of the residential community of Tilbury.



3. FIELD INVESTIGATION PROCEDURES

The field work for the current foundation investigation involved drilling of two (2) boreholes to supplement the subsurface information from a previous investigation. The new boreholes are identified as BEB and BWB, located within the Highway 401 median, on the east and west side of Baptiste Creek, respectively. The boreholes were drilled to depths of 30 m below the existing ground surface. The locations, ground elevations and depths of drilling are summarized in Table 3.

Table 3: Borehole Location and Termination Depth

BOREHOLE NO.	LOCATION				DEPTH (m)	GROUND ELEVATION (m)
	NORTHING	EASTING	LATITUDE	LONGITUDE		
BEB	4 682 155.4	314 566.0	42.278908	-82.381596	30.0	179.4
BWB	4 682 146.7	314 527.7	42.278830	-82.382060	30.0	179.4

PML staff visited the site on August 17, 2019 to mark out the borehole locations. The appropriate utility companies cleared the underground services at the borehole locations. Public and private utility authorities were informed and all of the utility clearance documents were obtained prior to commencement of the drilling work.

PML staff used a portable GPS device to establish the borehole locations in the field. Subsequently, the locations and elevations of the drilled boreholes were surveyed by PML with a Sokkia SHC5000 Differential GPS unit equipped with a GCX3 (Network RTK rover) GNSS Receiver. The vertical and horizontal limits of accuracy of the Differential GPS unit are within 0.1 m and 0.5 m, respectively. All elevations (EL.) reported in this report are referred to in MTM NAD 83 Northing and Easting (MTM Zone - ON11) Geodetic datum and expressed in metres.

The equipment used for drilling was owned and operated by London Soil Test Inc. (London Soil), of London, Ontario. London Soil is a specialist drilling contractor and worked under the full-time supervision of a PML field supervisor. Boreholes BEB and BWB were drilled between



September 25 and 26, 2019. The boreholes were advanced using a D50-Turbo Track-mounted drilling rig equipped with 200 mm diameter hollow stem augers.

Refer to Drawings BC-1 and BC-2 in Appendix A for borehole location details.

Representative soil samples were recovered from the boreholes at 0.75 m intervals to a depth of 6.0 m and at 1.5 m intervals to a depth of 20 m, and at 3.0 m interval to the termination depth, using a conventional 51 mm OD split spoon sampler in accordance with the Standard Penetration Test (SPT) procedure. Standard penetration tests were conducted simultaneously with the sampling operation to assess the strength characteristics of the substrata. In addition, attempt was made to measure in-situ vane shear strength of clayey soil at depths where SPT values were below about 8 blows/300 mm, using a N-size (MTO) vane.

The groundwater conditions at the borehole locations were observed during the drilling operations by visual examination of the soil samples, sampler and drill rods as the samples were retrieved. In addition, water level measurements were taken in the open boreholes upon completion of drilling. A monitoring well, consisting of 50 mm outside diameter rigid PVC pipe, was installed adjacent to each borehole for groundwater level measurement. Water levels were measured using a Solinst flat tape water level reader.

Boreholes and monitoring wells were constructed and abandoned/decommissioned in conformance with the requirements of MTO guidelines and Ontario Regulation 903, amended by Ontario Regulation 372.

The Baptiste Creek water level was observed approximately at EL. 175.4 during the fieldwork carried out on September 26, 2019.

The recovered soil samples were delivered to the PML laboratory to conduct detailed visual examination, and index tests.



4. LABORATORY TEST PROCEDURES

Laboratory tests were conducted on representative SPT soil samples recovered during the fieldwork investigation work. Testing was conducted at PML's laboratory facility located in Toronto, Ontario. The laboratory testing program included the following:

- Natural moisture content determinations (42)
- Grain size distribution analyses (12)
- Atterberg limit tests (12)
- Consolidation test (1)

All the laboratory tests to determine the soil index properties were performed in accordance with the MTO test procedures, which follow the American Society for Testing Materials (ASTM) standards, with the exception of hydrometer tests (LS-702). The results of the grain size distribution analyses are presented in Figures GS-BC-1 to GS-BC-2. The results of the Atterberg Limit tests are presented in Figures PC-BC-1 to PC-BC-2. One-dimensional consolidation (ASTM D-2435) testing was conducted on one Shelby tube sample from borehole BEB and the results are presented in Figure B-1. All of the test results are summarized in the attached Record of Borehole Logs provided in Appendix A.

5. SITE GEOLOGY AND SUBSURFACE CONDITIONS

5.1 Site Geology

In general, the project area is located within the St. Clair Clay Plains physiographic region. The Quaternary Geology map published by the Ontario Ministry of Northern Development and Mines (MNDM), indicates that the surface conditions in the area of the bridge site consist of Tavistock Till deposits; silty clay matrix. Based on the Bedrock Geology map (MRD126-REV1, 2011) published by the MNDM, the project area consists of Middle Devonian limestone, dolostone and shale of the Hamilton Group rock formation.



5.2 Previous Investigation

The field investigation for the existing bridges was carried out between January 6 and 15, 1959, and consisted of four (4) boreholes drilled to depths ranging from 15.5 m (EL. 162.4) to 30.8 m (EL. 147.0) below the ground surface that existed at the time of investigation. Based on the foundation investigation and design report (FIDR, Geocres No. 40J08-018, dated February 19, 1959), representative soil samples were recovered from the boreholes at frequent intervals to the termination depth of the boreholes, using a conventional 51 mm OD split spoon sampler, while simultaneously conducting SPT to assess the strength characteristics of the substrata. In addition, 76 mm diameter thin wall tube (Shelby) undisturbed samples were also recovered to conduct complex laboratory tests. The laboratory tests consisted of index tests such as moisture content, Atterberg limits and grain size distribution. One-Dimensional consolidation test was conducted on one of the undisturbed samples retrieved.

Based on the previous investigation, the subsoil conditions in the area of the proposed structure is expected to consist of 300 mm to 500 mm of topsoil underlain by silty clay to the maximum borehole termination depth of 30.8 m (EL. 147.0). The upper most part of this clay layer to a depth of 1.2 m to 1.8 m appears to be desiccated. The SPT 'N'-values reported to a depth of 15.2 m (EL. 162.7) to 16.7 m (EL. 161.0) ranged from 10 blows to as high as 32 blows, indicating stiff to hard consistency. Below these depths, the SPT 'N'-values reported ranged from 6 blows to 13 blows, indicating firm to stiff consistency. However, the silty clay deposit in one of the boreholes extended below 24.8 m (EL. 152.9) to 30.8 m (EL. 147.0) was observed to contain proportions of sand and gravel varying from 14% to 16%.

Groundwater was not encountered during drilling of any of the boreholes. The stabilized groundwater level was assumed to be limited by the creek water level of EL. 175.1 that was observed during the investigation.

5.3 Current Investigation

The subsurface conditions encountered during the current investigation along with the field and laboratory test results are shown on the attached Record of Borehole Sheets. The borehole locations and stratigraphic profile sections are shown on Drawings BC-1 and BC-2. The boundaries between soil strata have been established at the borehole locations only.



The boundaries of soil strata between and beyond the boreholes are assumed and may vary from location to location.

In general, the subsoil conditions immediately below the ground surface on the proposed structures consist of approximately 3.0 m thick fill composed of silty sand to sandy silt and clayey silt to silty clay, with varying proportion of sand and gravel, which is underlain by a 27.0 m thick deposit of hard to firm silty clay to clayey silt till deposit. Boreholes BEB and BWB were terminated in the firm clayey silt till at a maximum depth of 30.0 m below the existing ground surface. For classification purposes, the soils encountered at this site can be divided into two (2) distinct zones:

- a) Fill
 - i) Clayey Silt, some sand, some gravel (Fill)
 - ii) Silty Sand to Sandy Silt, Some Gravel (Fill)
 - iii) Silty Clay, some sand, trace gravel
- b) Silty Clay to Clayey Silt, Some Sand, Trace Gravel (Till)

5.3.1 Fill

- i) Clayey Silt, some sand, some gravel (Fill)

A layer of clayey silt fill was encountered just below the existing ground surface. The layer extended to a depth of 0.6 m (EL.178.8) below the existing ground surface in borehole BWB. This fill is underlain by silty sand to sandy silt, some gravel fill. A thin layer of clayey silt fill was also encountered in borehole BEB at a depth of 2.6 m (EL.176.8) and extends to a depth of 3.0 m (EL. 176.4) below the existing ground surface.

One SPT 'N'-value recorded in this fill was 21 blows, indicating very stiff consistency. The moisture contents of samples tested from this fill were 7.5% and 8.2%.

- ii) Silty Sand to Sandy Silt, some gravel (Fill)

Silty sand to sandy silt, some gravel fill was encountered just below the existing ground surface in borehole BEB and underneath the clayey silt, some sand, some gravel fill in borehole BWB. This fill starts to a depth of 0.6 m (EL. 178.8) in borehole BWB and extends to a depth of 3.0 m (EL. 176.4) in both boreholes.



The SPT 'N'-values in the silty sand to sandy silt fill were between 9 and 19 blows, indicating loose to compact conditions. The moisture contents of the samples tested from this fill were between 5.9% and 10.3%.

iii) Silty Clay, some sand, trace gravel

A thin layer of silty clay fill was encountered underneath the silty sand to sandy silt in borehole BWB. This fill starts at a depth of 2.6 m (EL. 176.8) and extends to a depth of 3.0 m (EL. 176.4) below the existing ground surface.

One SPT 'N'-value recorded in this fill was 10 blows, indicating stiff consistency. The moisture content of a sample tested from this fill was 10.3%.

5.3.2 Silty Clay to Clayey Silt, Some Sand, Trace Gravel (Till)

The fill in boreholes BEB and BWB is underlain by this silty clay to clayey silt till deposit with varying proportions of sand and gravel. This till deposit starts at depths ranging from 3.0 m (EL. 176.4) to the (borehole) termination depth of 30.0 m (EL. 149.4) below the existing ground surface. Small natural gas pockets were encountered at depth near termination depth. The SPT 'N'-values in this deposit vary from 10 to 31 blows, indicating stiff to hard consistency from EL. 176.4 to EL. 165.0. Below EL. 165.0, the SPT 'N'-values typically range from 6 to 12, indicating firm to stiff consistency, with the exception of two SPT 'N'-values of 16 and 26 at about EL. 159.5 in both boreholes, indicating very stiff consistency. The moisture contents of the samples tested from this till deposit were between 13.7% and 26.2%.

In-situ vane shear tests were attempted at depths where low 'N'-values were observed. The tests were performed at seven (7) locations between EL. 164.0 and EL. 149.0 within this till deposit and the uncorrected vane shear strengths (C_u) measured ranged between 80 kPa and 117 kPa, with a sensitivity ratio value between 1 and 2, indicating stiff to very stiff consistency, compared to firm to very stiff based on SPT 'N'-values.

The grain size distribution results of selected silty clay to clayey silt samples from this till deposit are provided in Figures GS-BC-1 and GS-BC-2, and the results of Atterberg limits for the same samples are provided in Figures PC-BC-1 and PC-BC-2 in Appendix A.



Sieve analysis tests were performed on twelve (12) representative samples and the test results indicate that this deposit consists of 1 to 5% gravel, 13% to 19% sand, 36% to 63% silt, and 19% to 42% clay. Atterberg limit tests were performed on eleven samples (11) representative samples and the test results indicate that liquid limit values range from 21 to 43, plastic limit values range from 15 to 21 and corresponding plasticity index values range from 6 to 23. Based on the test results, the clayey soil may be classified as clay of low to medium plasticity (CL/CI) in the Unified Soil Classification System (USCS), i.e., clayey silt/silty clay and classified as clayey silt to Silty clay based on the MTO Soil Classification system.

One-dimensional consolidation testing was conducted on one Shelby tube sample obtained from borehole BEB that was considered to be representative of the site conditions. As part of the one-dimensional consolidation and particle size analysis of soils (LS-702), specific gravity test was performed on one sample from the clayey silt till deposit. The specific gravity of the clayey silt sample determined was 2.713. Bulk unit weight of sample determined was 19.7 kN/m^3 and corresponding dry unit weight of the samples were 15.9 kN/m^3 .

The test results are provided in Appendix A. The following table summarizes the consolidation characteristics of the sample.

BOREHOLE NO./ DEPTH (m)	EFFECTIVE OVERBURDEN PRESSURE (kPa)	PRE- CONSOLIDATION PRESSURE (kPa)	OVER CONSOLIDATION RATIO (OCR)	INITIAL VOID RATIO (e_o)	COMPRESSION INDEX (C_c)
BEB/18.6	206	288	1.4	0.674	0.190

5.3.3 Groundwater

Groundwater was not encountered during and upon completion of drilling in boreholes BWB and BEB. The water level in the creek, which may ultimately control the groundwater level, was observed approximately at EL. 175.4 during the fieldwork.

A monitoring well consisting of 50 mm diameter PVC pipe was installed adjacent to boreholes BEB and BWB. Water level readings from the monitoring wells are summarized in Table 5.3.3.



Table 5.3.3: Water Level Readings in Monitoring Wells

MONITORING WELL (MW)	GROUND SURFACE ELEVATION (m)	TOP OF CASING ELEVATION (m)	MID-SCREEN DEPTH (m) (ELEVATION, m)	WATER LEVEL MEASURED IN MONITORING WELL, DEPTH (m) (ELEVATION, m)		
				2019/10/1	2019/10/11	2019/10/24
BEB	179.4	180.2	6.9 (EL. 172.5)	5.7 (EL. 173.7)	2.5 (EL. 176.9)	2.5 (EL. 176.9)
BWB	179.4	180.2	6.9 (EL. 172.5)	6.6 (EL. 172.8)	2.1 (EL. 177.3)	2.1 (EL. 177.3)

Groundwater levels may fluctuate due to the influence of precipitation and seasonal change. The groundwater measurements were observed and measured prior to backfilling the boreholes. Groundwater levels are shown on the Borehole Logs in Appendix A.

5.3.4 Soil Corrosivity

Four (4) representative soil samples were sent to SGS Canada Inc. located in Toronto, Ontario, which is accredited by Canadian Analytical Laboratory Association (CALA). The corrosivity test results provided by SGS are presented in Appendix A. A summary of the test results is presented in the Table 5.3.4.



Table 5.3.4: Summary of Corrosivity Results

BOREHOLE ID	SAMPLE NO.	CORROSIVITY INDEX	SULPHIDE (%)	SOIL REDOX POTENTIAL (mV)	pH	RESISTIVITY (Ohm-cm)	CONDUCTIVITY (uS/cm)	SULPHATE (µg/g)	CHLORIDE (µg/g)
BEB	3	4	<0.02	209	8.78	7260	138	8.5	17
BEB	6	11	<0.02	254	7.71	855	1170	900	99
BWB	4	5.5	0.05	401	7.97	2860	350	320	53
BWB	7	14.5	0.11	284	7.74	483	2070	2800	42



The Foundation Investigation portion of the report was prepared by Mr. K. Amatya, P.Eng. and Mr. N. Rahman P. Eng., Project Engineers and reviewed by Mr. G. Uwimana, MEng, P.Eng., Senior Engineer. Mr. R. Ng, MBA, PhD, P.Eng. Principal Consultant conducted an independent review of the report.

Yours very truly

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PART B – PRELIMINARY FOUNDATION DESIGN PORTION OF THE REPORT

6. PROJECT DESCRIPTION

6.1 General

The Ministry of Transportation Ontario has proposed to widen the existing Highway 401 from four lanes to six lanes (widening 1 lane in each direction), from east of Essex Road 42 easterly to west of Merlin Road. The proposed widening will be along the existing median between Highway 401 EBL and WBL with Ontario Tall Wall concrete Median barrier. The Ministry requires a Design-Build ready alternative package for the delivery of this project.

This report provides recommendations for foundation design based on interpretation of the previous report and on the geotechnical data presented in the factual portion for the new investigation presented in this report (Part A) and the details provided on the General Arrangement (GA) drawings for the proposed eastbound and westbound bridges on Highway 401 at the crossing of Baptiste Creek in the Town of Tilbury, Ontario. Based on the GA drawings, it is proposed to widen the existing bridges to the median with a single-span structure similar to the existing structures.

The discussions and recommendations presented in this report are based on the information provided by WSP and the factual data obtained during the geotechnical investigation carried out by PML.

This preliminary foundation investigation and design report with the interpretation and recommendations are intended for use by WSP on behalf of MTO. Any other parties including the Project CO Team or design-build contractor may use the information presented in this report at their own risks. Where comments are made on construction, they are provided only to highlight those aspects, which could affect the design of the project. Contractors must make their own interpretation of the factual information provided in Part A of the report, as it may affect equipment selection, proposed construction methods and scheduling.

6.2 Existing Structures

Based on the General Layout Drawings D 4311-1, 3 and 5 dated May 1959, the existing bridges on EBL and WBL are rigid frame structures with a clear span of 17.6 m, and supported on approximately 2.1 m wide strip footings founded at about EL. 173.4. The footings were designed



for an allowable bearing pressure of approximately 168 kPa. Both bridges consist of 5.6 m high wing walls on both sides of the inlet and outlet of the creek to confine the approach embankment fill. The length of the wing walls range from 4.5 m to 8.7 m and the approach slabs at both locations are about 6.1 m long. The footings of the abutments and the wing walls are protected from scour by sheet pile walls driven to a depth of 3.8 m (EL. 170.8) below the top of the footings at EL. 174.1.

According to inspection reports prepared by others, the abutment walls appear to be in good condition structurally with no major cracks on the abutments or wing walls other than a few surficial cracks. The slopes on both sides of the abutments were heavily vegetated at the time of field investigation on September, 2019. The embankment slopes were observed to be in stable condition with no erosion or scour around the toe of the embankments.

6.3 Proposed Structure

Based on the GA drawings dated October 2019 that were provided by WSP, the proposed structure widening will have a clear span of 17.6 m, which is the same as for the existing bridges. It is proposed to support the widened portion of the new abutments on piles. The pile cap is shown to be founded EL. 173.4, the same as the existing strip footings. The design grade of the approaches at the east and west abutments will be approximately EL. 180.0.

7. FOUNDATION RECOMMENDATIONS

7.1 Subsoil Conditions

The summary of the subsoil and groundwater conditions are provided in Part A of this report.

7.2 Foundation Alternatives

The foundation alternatives discussed below are for preliminary design purposes to support the Design-Build Conceptual Design criterion. Additional foundation investigation and design services would be required to produce a detail design level Foundation Design Report.

For comparison purposes, the following Table 7.2 provides the advantages, disadvantages, risks and consequences of the foundation alternatives to support the proposed structure.



Table 7.2 – Comparison of Foundation Options

FOUNDATION TYPE	ADVANTAGES	DISADVANTAGES	RISKS / CONSEQUENCES	RELATIVE COST
Driven Steel Piles	Higher confidence level in settlement performance, which is the critical issue for this bridge widening May not require deep excavations for forming pile caps	Potential vibration induced during driving Potential requirement to design for corrosion protection Closed-end pipe pile is considered as displacement pile, which will impact the existing footings during installation. Hence, this type of pile is not considered feasible and practical at this site	Steel piles may require corrosion protection, in which case the corrosion protection would need to be designed by specialists.	Moderate
Caissons	Considering in-situ shear strengths measured during current investigation and existing groundwater conditions, caissons are not considered feasible at this site location due to the high cost and limited resistance available due to the subsoil conditions		Higher risk of undermining existing shallow foundations due to soil removal in caisson installations	Relatively high
Shallow Foundations	Same construction as existing, but now under different circumstances considering the depth that would be required for construction at the same founding level and the importance of acceptable differential settlement performance for widenings	Deep excavations and roadway protection/shoring in the order of 9 m would be required to construct spread footings for widenings at the same elevation as existing spread footings Longitudinal slip joints would be required to accommodate the differential settlements between the spread footing foundations for the existing structure and the new widenings	Higher risk of excessive settlement and resulting distresses to bridges due to differential settlements	Foundation cost relatively low, but more cost uncertainty due to deeper excavation and shoring requirements

Based on the evaluation summarized in Table 7.2, driven H-piles are the preferred foundation for the proposed widenings from a foundations engineering perspective.



7.2.1 Driven Steel Piles

The proposed structure may be supported on steel H-piles or open-ended pipe piles (if displacement concerns are adequately addressed by the design-build entity) driven to the tip elevations indicated on Table 7.2.1a, and terminated in the cohesive till deposit. The geotechnical resistances provided below are primarily derived from shaft friction. As a result, the spacing between piles should be at least three times the diameter or width of the piles. The removal of soil inside the open-ended steel tube pile may be practically limited to a depth that is structurally required for placing reinforcement and concrete to connect the pile to the pile cap. Further, consideration may be given to locating the pile caps supporting the abutments in pile bent configuration or at an elevation that is structurally feasible and below the frost penetration depth of 1.0 m, to avoid excessively deep excavations and shoring requirements adjacent to existing footings.

It is anticipated that driving of piles will commence from the proposed founding elevation of the pile cap, adjacent to the existing footings. In order to prevent damages to the existing structures, the piles should be lowered in approximately 400 mm to 500 mm diameter pre-augured holes extending to a depth of 3.0 m below the founding levels of the existing footings, i.e., to about El. 170.0, and driven to the tip elevations suggested in Table 7.2.1a.

The construction of the deep foundation should conform to OPSS.PROV 903, amended by SP 109F57.

Table 7.2.1a summarizes the geotechnical resistances for the preliminary design purpose of open-ended pipe and steel H piles.

Table 7.2.1a: Geotechnical Resistances for Preliminary Design

TYPE OF PILES	EMBEDMENT LENGTH BELOW EXISTING FOUNDATION DEPTH (m)	PILE TIP ELEVATION (m)	FACTORED AXIAL RESISTANCES (kN)	
			AT ULS	AT SLS
324 mm O.D, diameter, 6.3 mm thick Open-ended Pipe Pile	15	155.0	310	230
	18	152.0	360	280



Table 7.2.1a: Geotechnical Resistances for Preliminary Design

TYPE OF PILES	EMBEDMENT LENGTH BELOW EXISTING FOUNDATION DEPTH (m)	PILE TIP ELEVATION (m)	FACTORED AXIAL RESISTANCES (kN)	
			AT ULS	AT SLS
HP 310 x 110 Steel H-Piles	15	155.0	370	280
	18	152.0	460	350
HP 310 x 79 Steel H-Piles	15	155.0	340	250
	18	152.0	420	320

It is estimated that settlement of individual piles (SLS condition) may be less than or equal to 10 mm for the factored axial resistance at Serviceability Limit State (SLS) provided in Table 7.2.1a.

The lateral resistance of the piles may be computed using the equation provided below for cohesive soils, and the soil parameters recommended in Table 7.2.1b.

a) Cohesive Soils (Davison, 1970)

$$k_s = 67 \tau_u/d$$

where τ_u = Undrained shear strength

d = Pile diameter or width, m

Table 7.2.1b: Parameters for Calculation of Coefficient of Lateral Subgrade Reaction

SOIL BOUNDARY ELEVATION		SOIL TYPE	UNDRAINED SHEAR STRENGTH (kPa)	n_h Values (kN/m ³)
FROM	TO			
173.4	163.0	Very stiff silty clay to clayey silt	110	-
163.0	152.4	Stiff silty clay to clayey silt	80	-

The ultimate lateral resistance may have to be reduced, based on pile spacing for open-ended pipe piles. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in Table 7.2.1c:



Table 7.2.1c – Pile Spacing and Group Reduction Factor¹

CONDITION	PILE SPACING (CENTRE TO CENTRE)	GROUP REDUCTION FACTOR
Pile group oriented perpendicular to the direction of loading	$\geq 2.5D$	1.0
Pile group oriented parallel to direction of loading	8 D	1.0
	6 D	0.7
	4 D	0.4
	3 D	0.25

1. Terzaghi, K., Peck, R.B. & Mesri, G. (1996). *Soil Mechanics in Engineering Practice* (3rd ed.). New York, NY: John Wiley & Sons, Inc.

For steel H-piles, battered piles could be considered to resist lateral loads on the abutments.

Boulders and cobbles were not encountered during the current investigation. The piles may not require pile tip reinforcement during driving through the existing soil overburden. However, if pile reinforcement is considered during construction, oversized driving shoes similar to Ontario Provincial Standard Design (OPSD) 3000.100 (Foundation Piles Steel H-Pile Driving Shoe) or Titus H bearing pile point are not suggested. These types of pile tip reinforcement may reduce the shaft friction and may lead to overrun, especially when the pile capacity is derived from shaft friction.

7.2.2 Shallow Foundation

The existing structures are supported on shallow footings placed at about EL. 173.4. The existing ground level in the area of the median appears to be same as that of the road (EL. 180.0) with a forward slope of 2H:1V towards the creek. The construction of footings for the proposed abutments may require about 6.0 m to 6.5 m excavation from the existing forward slope and related roadway protection/shoring. Settlement monitoring would be required to measure differential settlements between the existing bridge footings and the new footings for the widened portions of the bridges and to monitoring existing structures adjacent to foundation excavations during construction. The settlement monitoring program would be required for a specified period of time during the construction phase and upon completion of construction. Baseline readings would be taken prior to commencement of the construction work for the widening to monitor



movement of existing structures during the construction phase and immediately upon completion of construction to document differential settlements. Subsequent changes from the baseline readings would be monitored in conjunction with pre-established criteria for dealing with Cautionary, Review, and Alert level settlement/movement readings for safety purposes. Such a monitoring program could be developed, by MTO and WSP, if this foundation option is selected.

The proposed east and west abutments could be supported on footings placed approximately at El. 173.4. The geotechnical resistances provided on Table 7.2.2 for a 2.1 m wide footing may be considered for the design of the proposed widening bridges.

Table 7.2.2 – Founding Elevation and Geotechnical Resistance for Shallow Foundation

LOCATION	FOUNDING ELEVATION	FACTORED GEOTECHNICAL RESISTANCE AT ULS (kPa)	FACTORED GEOTECHNICAL RESISTANCE AT SLS (kPa)
East Abutment	173.4	300	200
West Abutment			

The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.10.4 of the CHBDC (2014). The previous foundation report from 1959 predicted settlements in the order of 165 mm of settlements under the projected loadings. Depending on the degree of consolidation of the founding soil under the highway embankment at the location of the proposed foundations for the widenings, the total settlement of new footings under the recommended SLS loads could be in the order of 70 to 85 mm.

A modulus of subgrade reaction for the soil at the founding level (EL. 173.4) of footing may be estimated at 15,000 kN/m³ for the design purposes.

The existing footings have been in place for almost sixty (60) years; long enough for cessation of time-related settlements. Accordingly, differential settlements are expected between the new and existing structures. In view of this, it is advisable to provide a “slip or Isolation” joint between the existing and the new structures to accommodate the differential settlements.



The sliding resistance of footings against lateral loads between the concrete footing and subgrade should be calculated in accordance with Section 6.10.5 of the CHBDC (2014). For cast-in-place concrete footings constructed on concrete working slabs and on top of very stiff clayey silt, the following friction factors ($\tan\delta$) are suggested:

- Cast-In-Place footing on concrete working slab: = 0.6
- Cast-In-Place concrete working slab on very stiff clayey silt: = 0.4

7.2.3 Rehabilitation of Existing Bridges

For the purpose of bridge rehabilitation (without addition of new median load) and evaluation of existing footings, the previous assessment in Section 6.2 of the previous PML report, based on previous codes, has been re-evaluated. Based on current practice and the requirements of the 2014 CHBDC, we suggest that the design values be updated to factored geotechnical resistances of 240 kPa at SLS and 450 kPa at ULS.

7.2.4 Impact on Existing Footings

The structural arrangement, orientation, size, and spacing of shallow foundations placed close/adjacent to each other should be designed to minimize the degree of overlap of the foundation soil pressure bulbs and/or ensure that the soil pressures within the pressure bulb overlap zones do not exceed the design/specified geotechnical resistance values.

The influence factors in the bearing capacity calculation/assessment include footing configurations, soil compressibility, and loading. Stuart (1962)², Mandel (1965)³, and West and Stuart (1965)⁴ considered that the influence of adjacent footings for soils with low angles of internal resistance, similar to the subgrade soils at this site, may not be significant considering that the interaction effects are reduced as the length (L) to width (B) ratio (L/B) exceeds one and the compressibility of soil may have a lessening effect on the interference. The risk of punching shear failure is considered negligible based on assessment of the foundation investigation and design data presented in this report. Although, in a qualitative sense, the interference effect may not be significant in the bearing capacity calculations, the design and construction methodology shall be devised and carried out to limit further impact on the existing footing(s).

2. Stuart, J.G. (1962). Interference between foundations with special reference to surface footings in sand, *Geotechnique* 12(1), 15-22.
3. Mandel, J. (1965). Interférence platique de semelles filantes; *Proceedings, Sixth International Conference on Soil Mechanics and Foundation Engineering*, Montreal, Vol II, 127-131.
4. West, J.M. & Stuart, J.G. (1965). Oblique loading resulting from interference between surface footings on sand, *Proceedings, Sixth International Conference on Soil Mechanics and Foundation Engineering*, Montreal, Vol II, 214-217.



The footings for the proposed bridge are expected to be placed along the longitudinal axis of the existing footings with the length of each new footing running parallel to the longitudinal axis of the existing footings. The new footings are to be founded on clayey soil. The width (B) of each new strip footing is assumed to be about 2.5 m with an approximate length (L) of 15.0 m. Assuming a footing length to width ratio (L/B) of 7.5, the effect of the new footings on the existing footings is considered to be minor. If the design is such that the influence will be limited, factored geotechnical resistances in the order of 240 kPa at SLS and 450 kPa at ULS may be assumed for preliminary checking the existing footings for the current load, and additional load from the abutment backfill and the new footing. The final configuration and design should be reviewed, and the conditions analysed and examined by a geotechnical engineer to ensure that the recommendations and adopted soil parameters are appropriately incorporated.

7.3 Approach Embankments

Based on the GA drawing, the existing ground elevation of the east and west approaches is at about EL. 180.0. The height of the fill required in the median approaches to match the existing road elevation, is about 0.5 m to 0.6 m. Based on the required fill, no instability and settlement problems are anticipated. The fill should consist of suitable fill material compacted in conformance with OPSS 501. Any spongy or soft area observed within the base of the excavation should be removed before placing the fill.

7.4 Lateral Earth Pressures

Earth retaining walls or abutments should be designed to resist the horizontal earth pressure imposed by the backfill and any surcharge load. The earth pressure for concrete structures should be computed as per Clause 6.12.2 of Canadian Highway Bridge Design Code (CHBDC, 2014). The lateral earth pressure, p (kPa), may be computed by using the following equation, assuming a triangular pressure distribution:

$$p = K (\gamma h_1 + \gamma' h_2 + q) + \gamma_w h_2 + C_p + C_s$$

where K = Coefficient of lateral earth pressure (dimensionless)

$$\gamma = \text{Unit weight of backfill material above assumed water level (kN/m}^3\text{)}$$



- γ' = Unit weight of submerged backfill ($\gamma_{\text{sat}} - \gamma_w$) material below assumed water level (kN/m^3)
- γ_w = Unit weight of water (9.8 kN/m^3)
- h_1 = Depth below final grade above design water level (m)
- h_2 = Depth below design water level (m)
- q = Surcharge load (kPa)
- C_p = Compaction pressure (kPa) (Clause 6.12.3 of CHBDC, 2014)
- C_s = Earth pressure from seismic events, (kPa) (Clause 4.6.5 of CHBDC, 2014)

Ontario Provincial Standard Specifications (OPSS.PROV 1010) Granular 'A' or 'B Type II' should be used as backfill material behind the wall and compacted in accordance with the requirements specified in the OPSS 902 (Excavation and Backfilling of Structures), amended by SP 109S12. The backfill material should be placed in layers not exceeding 200 mm (8 in.) in thickness before compaction.

Heavy vibratory compaction equipment adjacent to retaining structures should be restricted to limit the compaction pressure described in Clause 6.12.3 of the CHBDC, 2014. Restrictions on compaction near the retaining wall shall be as specified in OPSS 902 amended by SP 109S12. The type of compaction equipment and the compaction procedure that can be used for this purpose should be in accordance with OPSS.PROV 501 (Construction Specification for Compacting). Table 7.4 provides the recommended earth pressure coefficients.

Table 7.4 – Earth Pressure Coefficients

PARAMETERS	OPSS GRANULAR 'A'	OPSS GRANULAR 'B' TYPE II	FILL	CLAYEY SILT TO SILT CLAY
Internal Friction Angle, (degrees)	35°	30°	Effective Stress Value 24°	Effective Stress Value 20°
Unit weight, γ (kN/m^3)	22.5 ± 0.3	21.5 ± 0.3	18.0 ± 0.5	19.5 ± 0.5
Coefficient of Active Earth Pressure, K_a	0.27	0.33	0.42	0.49



Table 7.4 – Earth Pressure Coefficients

PARAMETERS	OPSS GRANULAR 'A'	OPSS GRANULAR 'B' TYPE II	FILL	CLAYEY SILT TO SILT CLAY
Coefficient of Earth Pressure at Rest, K_o	0.43	0.5	0.59	0.65
Coefficient of Passive Earth Pressure, K_p	3.69	3	2.37	2.04

The coefficient of earth pressure “at rest” should be used for design of rigid and unyielding walls where sufficient movement of the structure wall is not permitted. For unrestrained structures, the active earth pressure coefficient should be employed.

A weeping tile system (OPSS 405 and OPSD 3190.100) and/or weep holes should be installed to minimize the build-up of hydrostatic pressure behind the wall. The weeping tiles should be surrounded by a properly designed granular filter or geotextile (FOS 125 μm to 250 μm) to prevent migration of fines into the system. The drainage pipe should be installed on a positive grade and lead to a frost-free outlet. The geotextile should conform to OPSS.PROV 1860.

Backfilling adjacent to retaining structures should be carried out in conformance with OPSS 902 amended by SP 109S12. The minimum requirement of backfill material should be in accordance with OPSD 3101.150 for abutment and for retaining walls, it should be in accordance with OPSD 3121.150.

7.5 Seismic Considerations

The Spectral ($S_a(T)$, where T is in seconds) and Peak Ground Acceleration (PGA) for the project site is 0.110 ($S_a(0.2)$) and 0.067 (2%/50 years), respectively, based on the longitude and latitude coordinates of the proposed structure (National Building Code of Canada, 2015). The PGA_{ref} for the site is 0.054 in accordance with Clause 4.4.3.3, CHBDC (2014). The soil below the founding level at this site for seismic design purposes is classified as Site Class D in accordance with Clause 4.4.3.2, CHBDC 2014.



In accordance with Clause 4.4.4, CHBDC (2014), a seismic performance category of 1 (major-route and other bridges) is considered for the site. No seismic related design considerations are anticipated for this site.

7.6 Scour Protection

Cognizant of the potential harmful effects of stream flow to foundation systems carrying bridges in flood plains and near flowing waterbodies such as river, creek, stream and channel, scour and erosion protection should be considered. Assessment and evaluation to determine if bridge scour protection will required for this site may be carried out during the detail design stage.

The assessment, analysis, design, nature and extent of the bridge scour protection that may be required at this site is the responsibility of a qualified hydraulic engineer experienced in this field. It is suggested that the bridge scour and stability analyses be carried out in accordance with the guidelines set out in the Hydraulic Engineering Circular (HEC) Nos. 18 (Evaluating Scour at Bridges), 20 (Stream Stability at Highway Structures) and 23 (Bridge Scour and Stream Instability Countermeasures), and CHBDC (2014).

It is anticipated that scour protection may be required on creek banks, adjacent to the abutments and foreslopes. Rock protection or riprap could be provided to a minimum height of 1.0 m above the high flood level expected in the creek to the toe of the slope and into the creek bed within the plan limits of the bridges. The construction for rock protection or rip-rap should be in accordance with OPSS.PROV 511.

7.7 Frost Protection

All pile caps or footings shall be provided with a minimum of 1.0 m of earth cover or equivalent thermal insulation as protection against frost action as per OPSD 3090.101 (Foundation Frost Penetration Depths for Southern Ontario).

8. ROADWAY PROTECTION

For the construction of the proposed structure, it may require a properly designed temporary roadway protection system. The earth pressure values presented in Table 7.4 may be used for



design. Temporary roadway protection shall be designed to meet at least a Performance Level of 2 and constructed in accordance with OPSS.PROV 539 (Construction Specification for Temporary Protection Systems), amended by SP 105S09. The Contractor shall be responsible for the selection, detailed design and performance of the roadway protection system. OPSS.PROV 539, amended by SP 105S09, also calls for monitoring of the roadway protection system by the Contractor to check the horizontal and vertical displacements of the roadway.

9. EXCAVATION

All excavations should be carried out in accordance with the Occupational Health and Safety Act (OHSA) and MTO Regulations for Construction Projects. According to Occupational Health and Safety Act (Ontario Regulation 213/91) criteria, the existing fill is classified as Type 3 soils. The very stiff cohesive soil is considered as Type 2 soil. The stiff cohesive soil is considered as Type 3 soil. Soils below groundwater table and soils showing persistent seepages are considered having the characteristics of a Type 4 soil. The open cut procedure will be governed by soils with the highest soil type number.

The protection system for excavations should be in accordance with OPSS.PROV 539, amended by SP 105S09. Construction Specifications for Excavating and Backfilling—Structures should be in accordance with OPSS 902 amended by SP 109S12. All excavated surfaces should be kept free of frost and water during the period of construction. Runoff shall be directed away from open excavations and should not be allowed to flow into the excavation. Excavated material shall not be stockpiled on top of the excavation.

Prior to excavation, the locations and depths of existing underground utilities should be verified. All underground utilities that might be exposed and become unsupported as a result of the excavation should be properly supported to avoid potential damage.

Based on GA drawing, the pile cap is to be founded at EL. 173.4, about 6.0 m to 6.5 m below the existing ground surface. The protection system for excavation will be required in accordance with OPSS.PROV 539, amended by SP 105S09. A shoring system consisting of sheet piles or of H-piles with timber lagging may be used for excavation.

Alternatively, the pile cap may be founded at as high an elevation as possible but below the frost depth.



The base of the pile cap excavation should be protected from disturbance by placing a minimum 100 mm thick lean concrete, following the removal of existing fill material.

10. CONSTRUCTION CONSIDERATIONS

The "red flag" issues outlined in the following subsections and the recommended methods of overcoming these issues are intended to alert and aid the designer and the contractor. These comments and recommendations are based on the conditions revealed during the investigations and no responsibility is assumed by the consultants or the Client for alerting the contractor to all critical issues for each foundation alternative. The requirements to deliver acceptable quality of construction remain the responsibility of the contractor.

10.1 Groundwater Control

It is anticipated that up to 6.5 m of excavations will be required to found the proposed pile caps at both abutments at approximately EL. 173.4, which will extend below the prevailing groundwater approximately at EL. 175.4. A temporary protection system (i.e. cofferdam) will be required for dewatering operations to permit construction in the dry. A cofferdam consisting of sheet piles or H-piles with timber lagging may be used for excavation and dewatering. Dewatering may be carried out from the sump pumps located along the periphery of the cofferdam

Alternatively, the pile cap could be founded at about 2.5 m below ground surface EL. 177.4. With this option, conventional sump pumping techniques are considered to be adequate to mitigate any surface runoff and seepage from localized soil fissures at the excavation depth

In any case, groundwater should be lowered a minimum of 0.5 m below excavations for construction in-the-dry. The Contractor is responsible for the selection, design and performance of the groundwater control measures.

The contractor shall be responsible for the selection, performance and detailed design of the shoring and dewatering system including cofferdam. The dewatering system should be designed to conform to the requirement of OPSS.PROV 517, SP 517F01 and NSSP FOUN0003.



In accordance with SP 517F01, the dewatering system should be designed by a designer with a minimum 5 years of experience in the field. A preconstruction survey is not required due to the relatively shallow depth of dewatering and the relatively large distances to critical private properties.

10.2 Soil Corrosivity

A total of four (4) samples from the fill and clayey silt to silty clay deposit were tested for soil corrosivity and potential exposure of concrete to sulphate attack. A summary of the results of chemical analyses are provided in Section 5.3.4 of Part A of this report. The sulphate concentration varied from 50 µg/g to as high as 950 µg/g (0.0050% to 0.095%). Compared to the values suggested in Canadian Standard A23.1-14, the effect of fill material on buried concrete is considered negligible. The chloride contents of the samples from the fill ranged from as low as 16 µg/g to 290 µg/g (0.0016% to 0.029%). Generally, the concentration value in excess of 250 ppm (0.025%) leads to corrosive environment for buried metals or reinforcing steel. The potential for corrosive environment of this fill is assessed to be low to moderate.

Electrical resistivity less than 2000 ohm-cm generally leads to highly corrosive environment for steel elements in contact with soil. The resistivity values of fill samples ranged from 1120 ohm-cm to 5090 ohm-cm. The test results suggest that a corrosive environment exists at this site for steel elements in contact with fill where the resistivity was less than 2000 ohm-cm. The pH values of fill samples ranged from 7.73 to 8.23.

Generally, the existing fill may lead to a corrosive environment for steel elements and sulphate attack when concrete is in contact. It may be advisable to use imported backfill material selected to provide a more benign chemical environment for the approach embankments. Otherwise, measures to mitigate the impact of the chemical environment could be considered.



11. CLOSURE

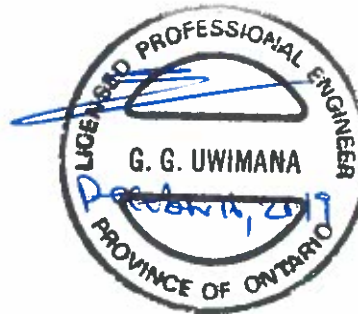
The Foundation Design portion of the report was prepared by Mr. N. Rahman, P.Eng. and reviewed by Mr. G. Uwimana, MEng, P.Eng., Senior Engineer. Mr. R. Ng, MBA, PhD, P.Eng. Principal Consultant conducted an independent review of the report.

Yours very truly

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NR/GU/RN/nr-nk



APPENDIX A

Borehole Locations Plan and Soil Strata Drawings BC-1 and BC-2

Explanation of Terms Used in Report

Record of Borehole Sheets

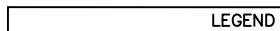
Results of Grain Size Distribution Analyses – Figures GS-BC-1 to GS-BC-2

Results of Atterberg Limit Tests – Figures PC-BC-1 to PC-BC-2

Consolidation Test Results – Figure B-1

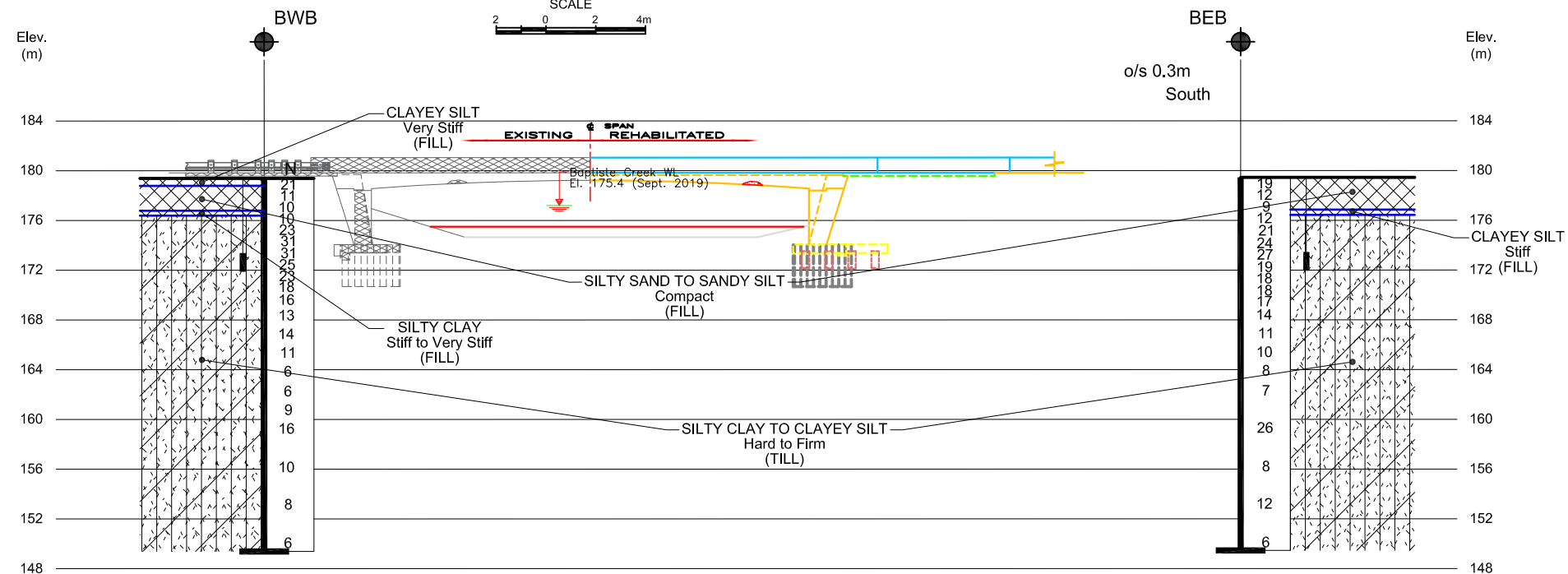
Results of Chemical Tests Provided by SGS Canada Inc.

SHEET

[illegible]

The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

REVISIONS					
	DATE	BY	DESCRIPTION		
Geocres No. 40J8-71					
HWY No	401				DIST WEST REGIO
SUBM'D	NL	CHECKED	KA	DATE DEC. 10, 2019	SITE 13-187/2
DRAWN	NL	CHECKED	NR	APPROVED RN	DWG BC-1



1. THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF REPORT AND RECORD OF BOREHOLE LOGS.
2. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
3. DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES AND METRES.

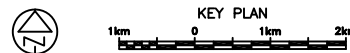
SCALE

HORIZONTAL 2 0 2 4m

VERTICAL 5 0 5 10m



Reference WSP Ltd. Drawing: 18M-02111-07-304-001GA.dwg,
dated October 2019.



LEGEND

- BEB
- Borehole Location
 - N Blows/0.3m (Std. Pen Test, 475 J/blow)
 - * Groundwater Level Could Not Be Established
 - Water Level During Drilling (September 2019)
 - Piezometer

BH No	ELEVATION	NORTHINGS	EASTINGS
BWB	179.4	4 682 146.7	314 527.7
BEB	179.4	4 682 155.4	314 566.0

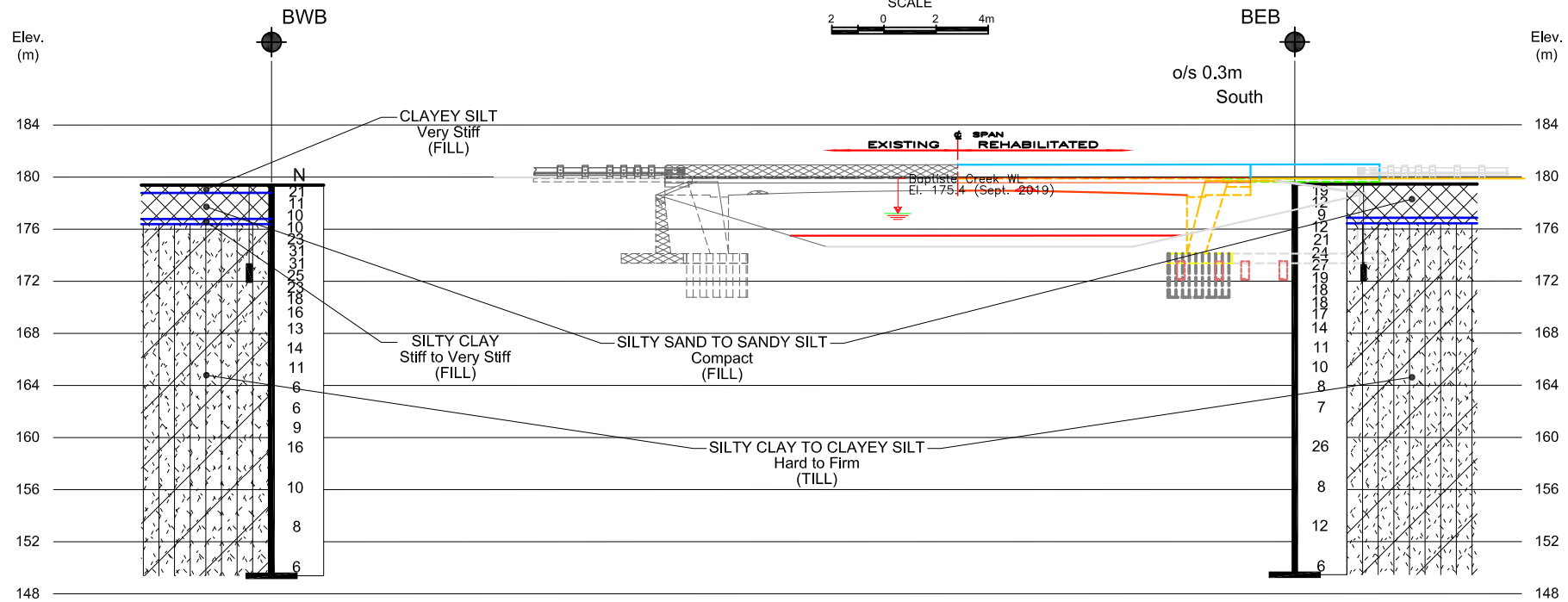
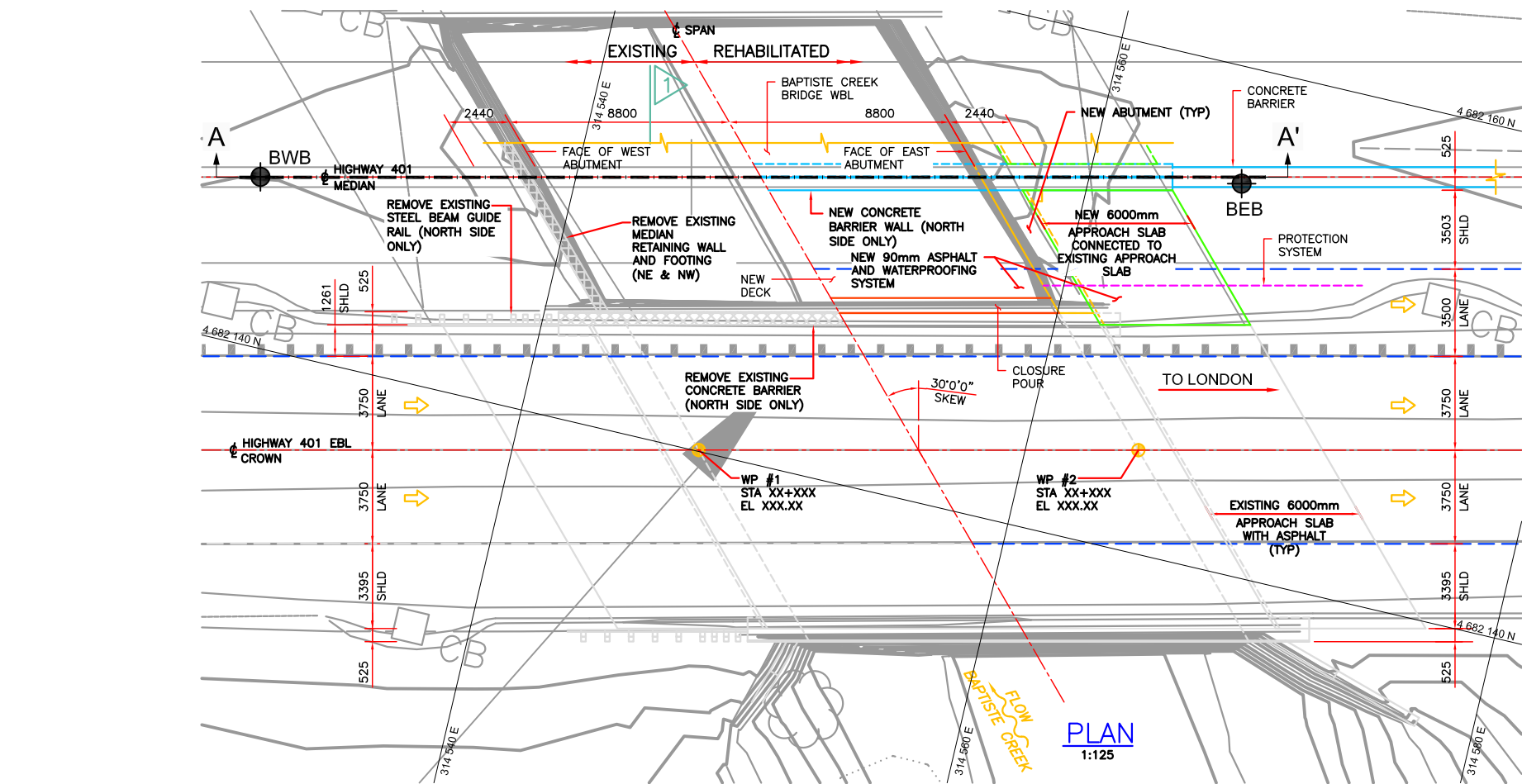
NOTE

The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

DATE	BY	DESCRIPTION

Geocres No. 40J8-71

HWY No	401	DIST WEST REGION
SUBM'D	NL	CHECKED KA DATE DEC. 10, 2019 SITE 13-187/2
DRAWN	NL	CHECKED NR APPROVED RN DWG BC-2



EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

COMPOSITION: SECONDARY SOIL COMPONENTS ARE DESCRIBED ON THE BASIS OF PERCENTAGE BY MASS OF THE WHOLE SAMPLE AS FOLLOWS:

PERCENT BY MASS	0 - 10	10 - 20	20 - 30	30 - 40	> 40
	TRACE	SOME	WITH	ADJECTIVE (SILTY)	AND (AND SILT)

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm* IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

R Q D (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S SPLIT SPOON	T P THINWALL PISTON
W S WASH SAMPLE	O S OSTERBERG SAMPLE
S T SLOTTED TUBE SAMPLE	R C ROCK CORE
B S BLOCK SAMPLE	P H T W ADVANCED HYDRAULICALLY
C S CHUNK SAMPLE	P M T W ADVANCED MANUALLY
T W THINWALL OPEN	F S FOIL SAMPLE
F V FIELD VANE	

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{v0}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_i	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	n	1, %	POROSITY	e_{max}	1, %	VOID RATIO IN LOOSEST STATE
γ_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	w	1, %	WATER CONTENT	e_{min}	1, %	VOID RATIO IN DENSEST STATE
ρ_w	kg/m ³	DENSITY OF WATER	S_r	%	DEGREE OF SATURATION	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
γ_w	kN/m ³	UNIT WEIGHT OF WATER	w_L	%	LIQUID LIMIT	D	mm	GRAIN DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_p	%	PLASTIC LIMIT	D_n	mm	n PERCENT - DIAMETER
γ	kN/m ³	UNIT WEIGHT OF SOIL	w_s	%	SHRINKAGE LIMIT	C_u	1	UNIFORMITY COEFFICIENT
ρ_d	kg/m ³	DENSITY OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	h	m	HYDRAULIC HEAD OR POTENTIAL
γ_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	q	m ³ /s	RATE OF DISCHARGE
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	v	m/s	DISCHARGE VELOCITY
γ_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL	DTPL		DRIER THAN PLASTIC LIMIT	i	1	HYDRAULIC GRADIENT
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	APL		ABOUT PLASTIC LIMIT	k	m/s	HYDRAULIC CONDUCTIVITY
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL	WTP		WETTER THAN PLASTIC LIMIT	j	kN/m ³	SEEPAGE FORCE
e	1, %	VOID RATIO						

RECORD OF BOREHOLE No BEB

1 OF 3

METRIC

G.W.P. 3034-19-00 LOCATION Coords: 4 682 155.4 N; 314 566.0 E ORIGINATED BY J.O.
DIST West Region HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.
DATUM Geodetic DATE 2019.09.26 LATITUDE 42.278908 LONGITUDE -82.381596 CHECKED BY R.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
179.4	Ground						20	40	60	80	100					
0.0	SANDY SILT TO SILTY SAND, some gravel		1	SS	19											
	Compact to loose, Light Brown, Moist (FILL)		2	SS	12											
			3	SS	9											
			4	SS	12											
176.4	CLAYEY SILT, some sand, trace gravel Very stiff, Grey, Dry															
3.0	SILTY CLAY TO CLAYEY SILT, some sand, trace gravel		5	SS	21											
	Very stiff to firm, Brown to grey, Dry to moist (TILL)		6	SS	24											
			7	SS	27											
			8	SS	19											
			9	SS	18											
			10	SS	18											
			11	SS	17											
			12	SS	14											
			13	SS	11											
			14	SS	10											
164.4																

Continued Next Page

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

RECORD OF BOREHOLE No BEB

2 OF 3

METRIC

G.W.P. 3034-19-00 LOCATION Coords: 4 682 155.4 N; 314 566.0 E ORIGINATED BY J.O.
DIST West Region HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.
DATUM Geodetic DATE 2019.09.26 LATITUDE 42.278908 LONGITUDE -82.381596 CHECKED BY R.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE						
164.4 15.0	(Cont'd) SILTY CLAY TO CLAYEY SILT, some sand, trace gravel Very stiff to firm, Brown to grey, Dry to moist (TILL)		15	SS	8		164							2 18 41 39	
							163								
			16	SS	7		162								
				VANE			161							P _c = 288 kPa C _c = 0.19 e _o = 0.674 S _G = 2.713	
			17	TW			160								
				VANE			159							1 17 63 19	
			18	SS	26		158								
							157								
			19	SS	8		156								
				VANE			155								
							154								
			20	SS	12		153							5 17 36 42	
							152								
							151								
			21	SS	6		150								
				VANE											

Continued Next Page







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

RECORD OF BOREHOLE No BEB

3 OF 3

METRIC

G.W.P. 3034-19-00 LOCATION Coords: 4 682 155.4 N; 314 566.0 E ORIGINATED BY J.O.
 DIST West Region HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.
 DATUM Geodetic DATE 2019.09.26 LATITUDE 42.278908 LONGITUDE -82.381596 CHECKED BY R.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)											
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa																				
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					WATER CONTENT (%)																	
						20	40	60	80	100	20	40	60															
149.4 30.0	End of borehole																											
	 Groundwater level measured in monitoring well NOTES: 1. Groundwater was not encountered during or upon completion of drilling. 2. No cave-in was noted upon extraction of hollow stem augers. 3. Monitoring well was installed adjacent to borehole BEB drilled location. <u>Monitoring Well Readings:</u> <table border="1"> <thead> <tr> <th>Date</th> <th>Depth (m)</th> <th>Elev.</th> </tr> </thead> <tbody> <tr> <td>Oct.01/'19</td> <td>5.7</td> <td>173.7</td> </tr> <tr> <td>Oct.11/'19</td> <td>2.5</td> <td>176.9</td> </tr> <tr> <td>Oct.24/'19</td> <td>2.5</td> <td>176.9</td> </tr> </tbody> </table> <u>Monitoring Well Legend:</u> <div style="display: flex; flex-direction: column; gap: 5px;"> <div> Stick-up Monument</div> <div> Bentonite</div> <div> Filter Sand</div> <div> 19 mm PVC Screen</div> <div> Filter Bottom</div> </div>	Date	Depth (m)	Elev.	Oct.01/'19	5.7	173.7	Oct.11/'19	2.5	176.9	Oct.24/'19	2.5	176.9															
Date	Depth (m)	Elev.																										
Oct.01/'19	5.7	173.7																										
Oct.11/'19	2.5	176.9																										
Oct.24/'19	2.5	176.9																										

RECORD OF BOREHOLE No BWB

1 OF 3

METRIC

G.W.P. 3034-19-00 LOCATION Coords: 4 682 146.7 N; 314 527.7 E ORIGINATED BY J.O.
DIST West Region HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.
DATUM Geodetic DATE 2019.09.25 LATITUDE 42.278830 LONGITUDE -82.382060 CHECKED BY R.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
179.4	Ground							20	40	60	80	100						
0.0	CLAYEY SILT, some sand, some gavel		1	SS	21		179											
	Very stiff, Grey, Dry to moist																	
	SILTY SAND TO SANDY SILT, some gavel		2	SS	11		178											
	Compact, Brown, Moist																	
	(FILL)		3	SS	10		177											
			4	SS	10		176											
176.4	SILTY CLAY, some sand, trace gravel																	
3.0	Stiff to very stiff, Brown to grey, Moist																	
	SILTY CLAY TO CLAYEY SILT, some sand, trace gravel		5	SS	23		175											
	Hard to firm, Grey, Moist		6	SS	31		174											
	(TILL)		7	SS	31		173											
			8	SS	25		172											
			9	SS	23		171											
			10	SS	18		170											
			11	SS	16		169											
			12	SS	13		168											
			13	SS	14		167											
			14	SS	11		166											
							165											
164.4																		

Continued Next Page

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

RECORD OF BOREHOLE No BWB

2 OF 3

METRIC

G.W.P. 3034-19-00 LOCATION Coords: 4 682 146.7 N; 314 527.7 E ORIGINATED BY J.O.
DIST West Region HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.
DATUM Geodetic DATE 2019.09.25 LATITUDE 42.278830 LONGITUDE -82.382060 CHECKED BY R.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
						○ UNCONFINED	+ FIELD VANE							
						● QUICK TRIAXIAL	× LAB VANE							
								WATER CONTENT (%)						
						20	40	60	80	100	20	40	60	
164.4														
15.0	(Cont'd) CLAYEY SILT, some sand, trace gravel Firm to stiff, Brown to grey, Moist (TILL)		15	SS	6									
				VANE										
			16	SS	6									
				VANE										
			17	SS	9									
			18	SS	16									
			19	SS	10									
			20	SS	8									
			21	SS	6									
				VANE										
149.4														

Continued Next Page







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

RECORD OF BOREHOLE No BWB

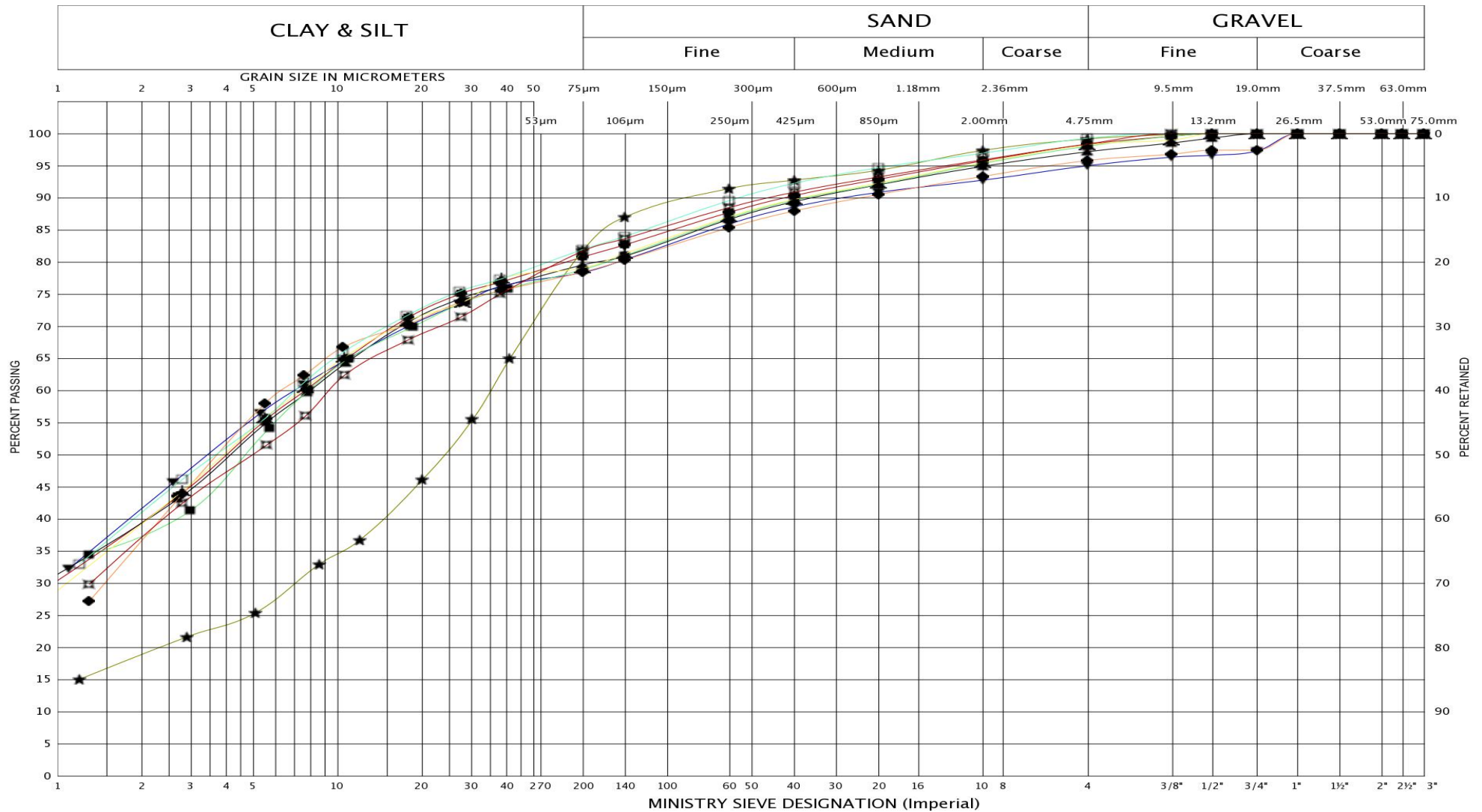
3 OF 3

METRIC

G.W.P. 3034-19-00 LOCATION Coords: 4 682 146.7 N; 314 527.7 E ORIGINATED BY J.O.
 DIST West Region HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.
 DATUM Geodetic DATE 2019.09.25 LATITUDE 42.278830 LONGITUDE -82.382060 CHECKED BY R.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)											
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)										
							20	40	60	80	100																	
149.4 30.0	End of borehole  Groundwater level measured in monitoring well NOTES: 1. Groundwater was not encountered during or upon completion of drilling. 2. No cave-in was noted upon extraction of hollow stem augers. 3. Monitoring well was installed adjacent to borehole BWB drilled location. <u>Monitoring Well Readings:</u> <table border="1"> <thead> <tr> <th>Date</th> <th>Depth (m)</th> <th>Elev.</th> </tr> </thead> <tbody> <tr> <td>Oct.01/'19</td> <td>6.6</td> <td>172.8</td> </tr> <tr> <td>Oct.11/'19</td> <td>2.1</td> <td>177.3</td> </tr> <tr> <td>Oct.24/'19</td> <td>2.1</td> <td>177.3</td> </tr> </tbody> </table> <u>Monitoring Well Legend:</u>  Stick-up Monument  Bentonite  Filter Sand  19 mm PVC Screen  Filter Bottom	Date	Depth (m)	Elev.	Oct.01/'19	6.6	172.8	Oct.11/'19	2.1	177.3	Oct.24/'19	2.1	177.3															
Date	Depth (m)	Elev.																										
Oct.01/'19	6.6	172.8																										
Oct.11/'19	2.1	177.3																										
Oct.24/'19	2.1	177.3																										

UNIFIED SOIL CLASSIFICATION SYSTEM



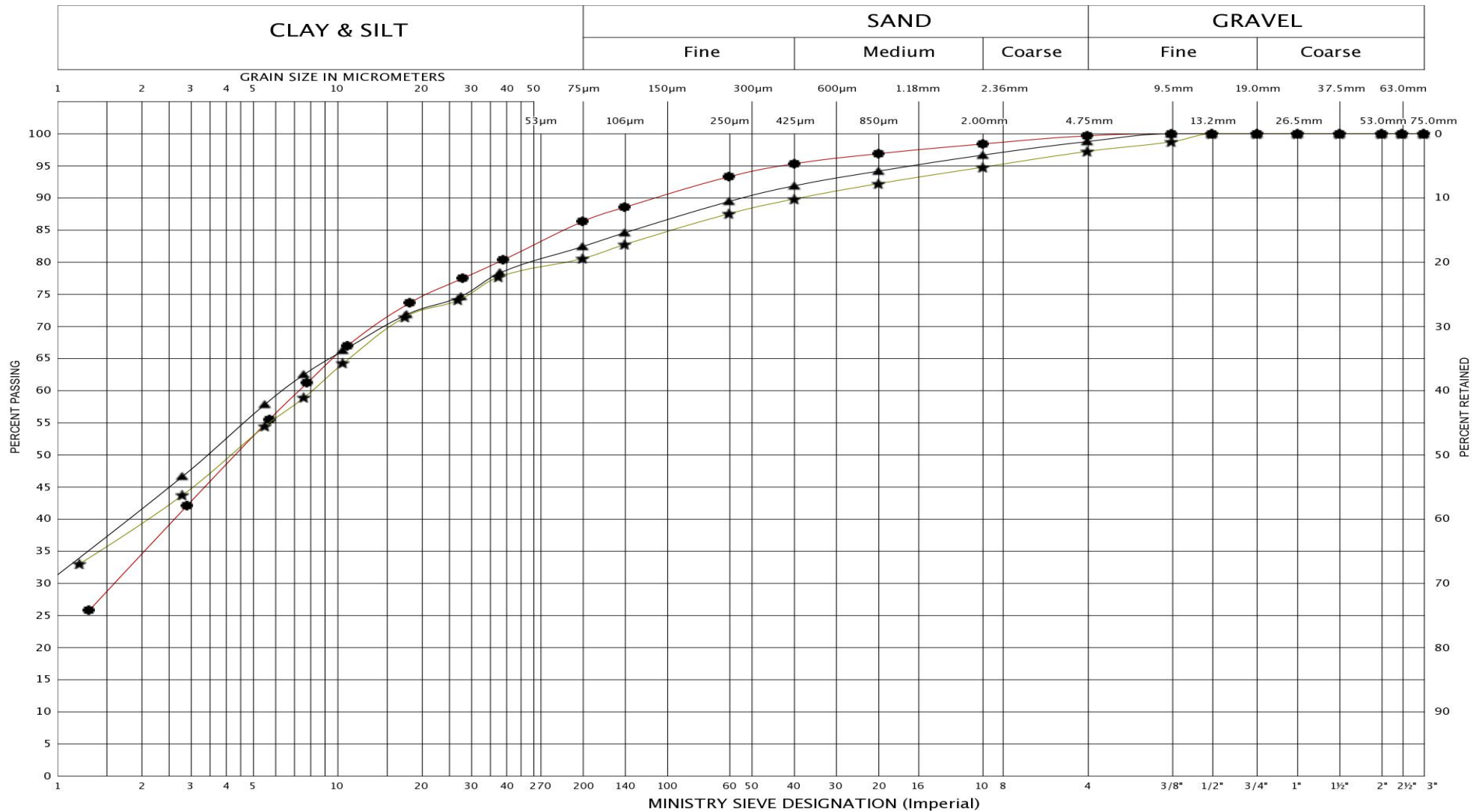
LEGEND	BH	BEB	BEB	BEB	BEB	BEB	BEB	BWB	BWB	BWB
SAMPLE	15	12	18	20	1	9	14	19	11	
SYMBOL	●	▲	★	▼	■	◆	▲	□	⊠	



GRAIN SIZE DISTRIBUTION
CLAYEY SILT, Some Sand, Trace Gravel ((Till))

FIG No.: GS-BC-1
HWY : 401
GWP 3034-19-00

UNIFIED SOIL CLASSIFICATION SYSTEM

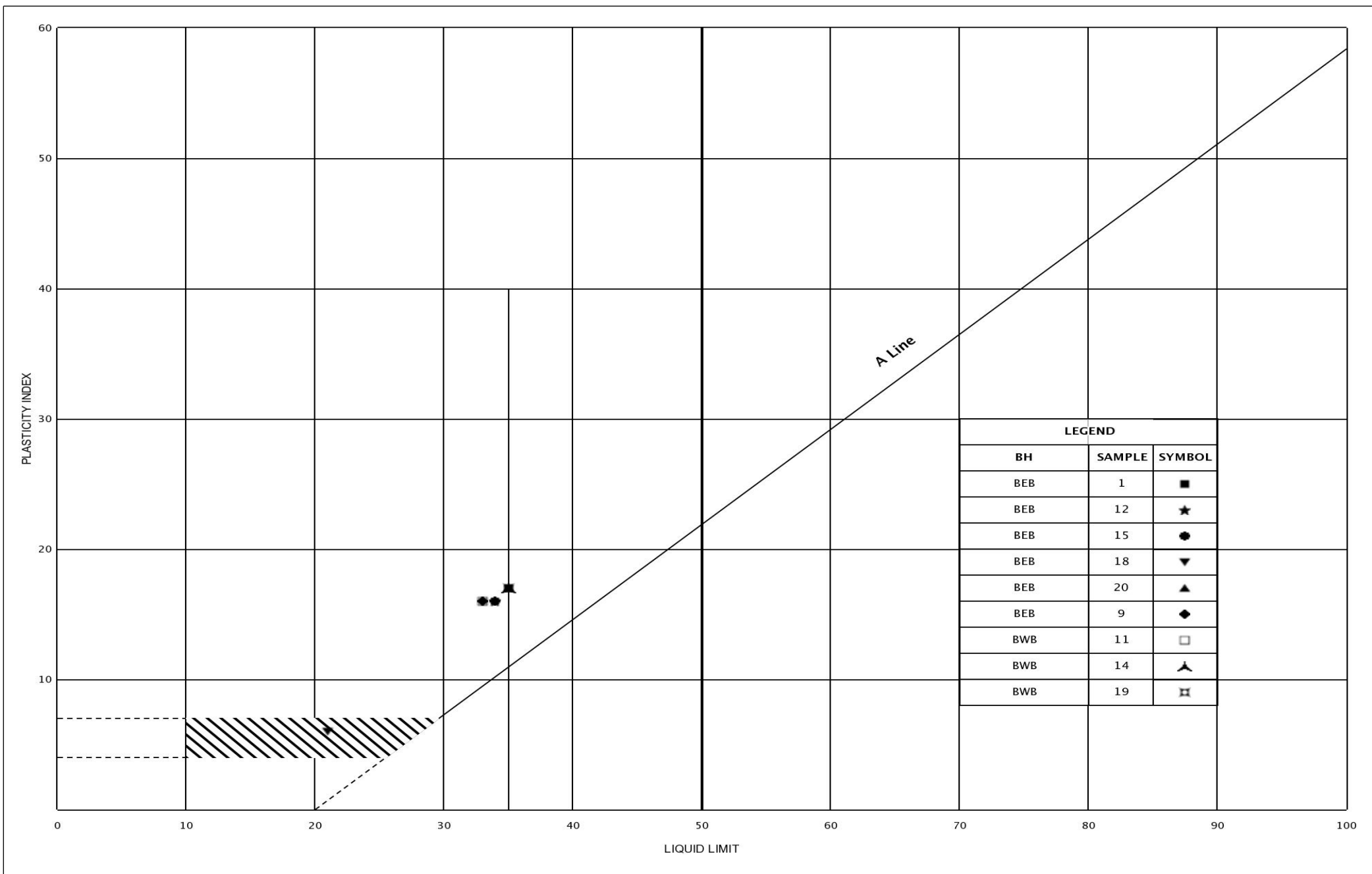


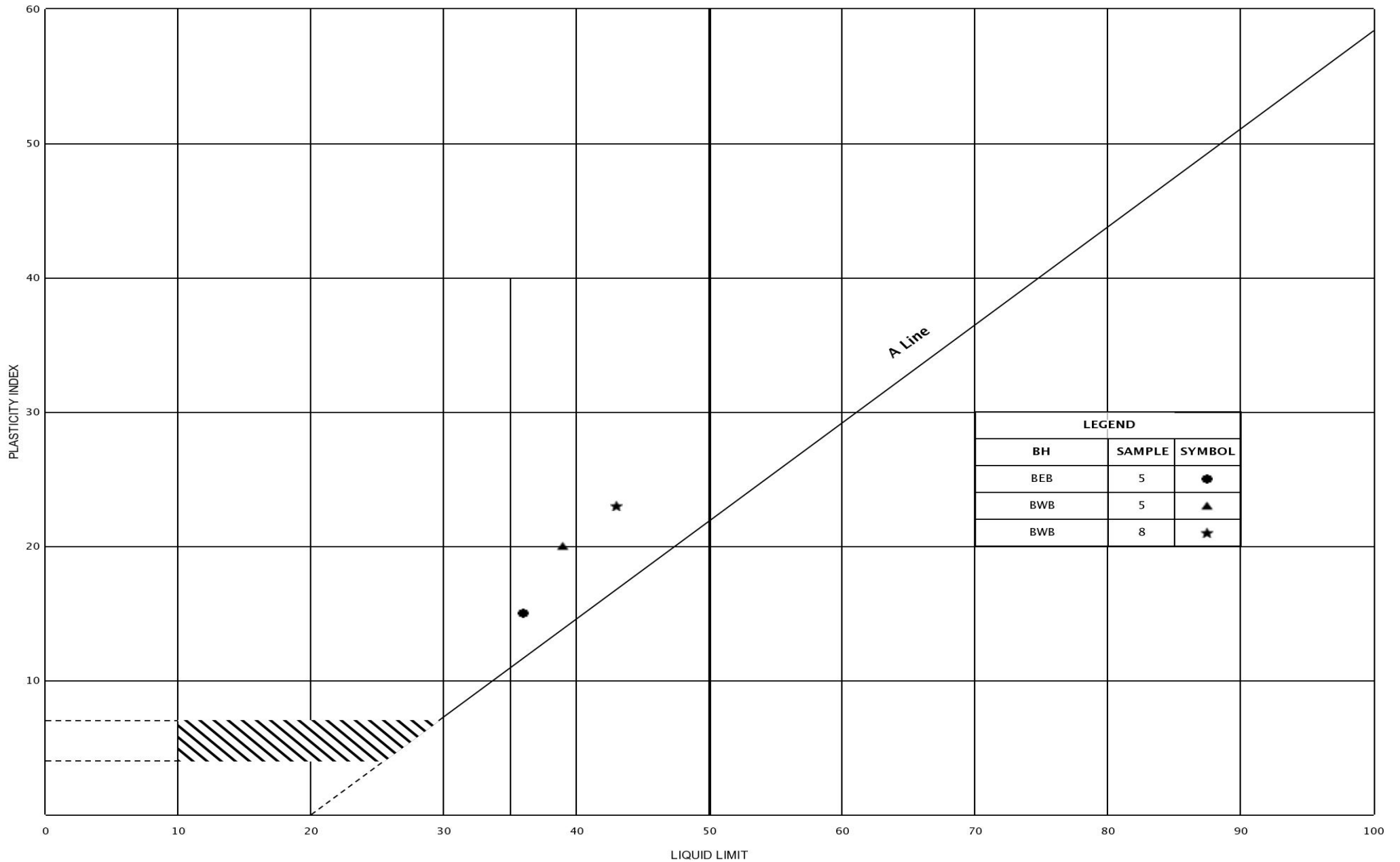
LEGEND	BH	BEB	BWB	BWB
SAMPLE	5	5	8	8
SYMBOL	●	▲	★	★



GRAIN SIZE DISTRIBUTION
 SILTY CLAY,Some Sand, Trace Gravel ((Till)

FIG No.: GS-BC-2
 HWY : 401
 GWP 3034-19-00

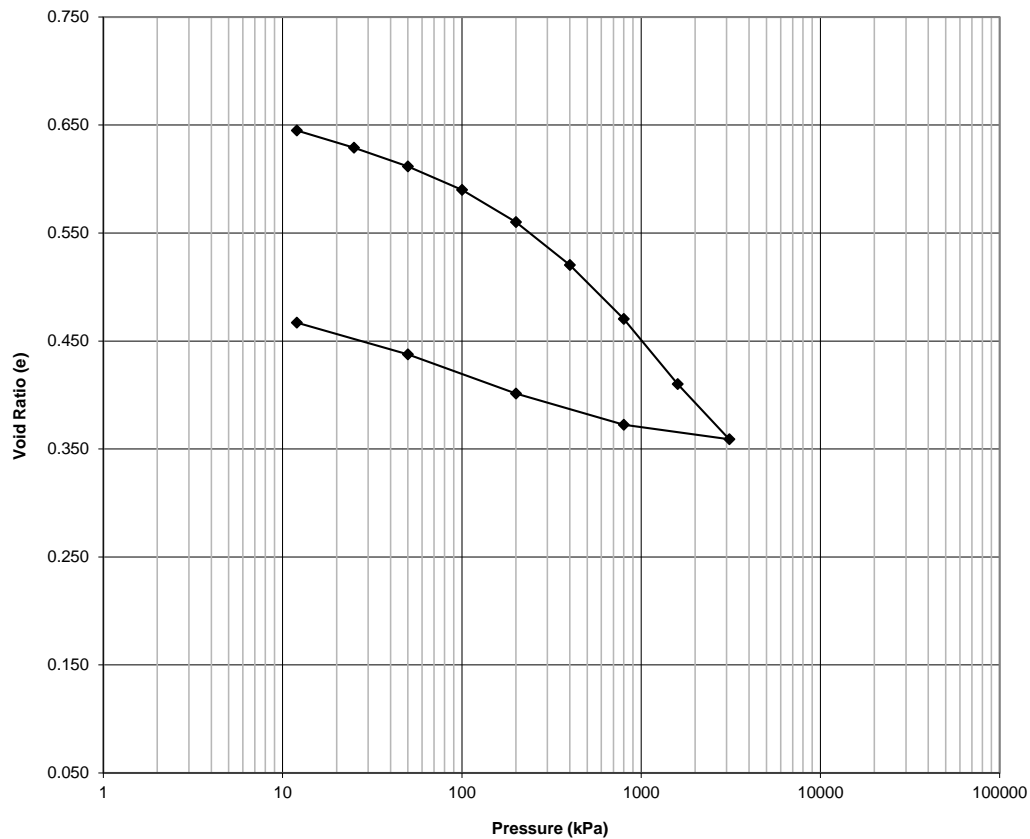




Consolidation Test Results
(ASTM D2435)
Highway 401, CA 3017-E-0006, Task 007-Tilbury

Borehole BEB, Sample TW 17, Depth 18.3-19.1 m

Void Ratio versus Log of Pressure



SOIL TYPE: Grey Clayey Silt			
e_0	=	0.674	W_L = 35
W_0	=	24.1 %	W_P = 18
γ	=	19.7 kN/m ³	PI = 17
FIGURE No: B-1			
Highway 401, CA 3017-E-0006, Task 007-Tilbury			
PML Ref: 19KF030A			



FINAL REPORT

CA14406-OCT19 R1

19KF030A Hwy 401, Tilbury (B/T)

Prepared for

Peto MacCallum Ltd

First Page

CLIENT DETAILS

Client Peto MacCallum Ltd

Address 165 Cartwright Ave
Toronto, ON
M6A 1V5, Canada

Contact Nazibur Rahman

Telephone 416-785-5110

Facsimile 416-785-5120

Email nrahman@petomacallum.com

Project 19KF030A Hwy 401, Tilbury (B/T)

Order Number

Samples Soil (8)

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 CA14406-OCT19

Received 10/11/2019

Approved 10/17/2019

Report Number CA14406-OCT19 R1

Date Reported 10/17/2019

COMMENTS

Temperature of Sample upon Receipt: 7 degrees C

Cooling Agent Present: Yes

Custody Seal Present: Yes

Chain of Custody Number: 007602

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

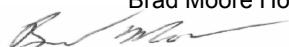




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FINAL REPORT

CA14406-OCT19 R1

Client: Peto MacCallum Ltd

Project: 19KF030A Hwy 401, Tilbury (B/T)

Project Manager: Nazibur Rahman

Samplers: Jinsuko

PACKAGE: - Corrosivity Index (SOIL)

Sample Number	5	6	7	8	9	10	11	12
Sample Name	BWB, SS4 (7.5'-9.5')	BWB, SS7 (15'-17')	BEB, SS3 (5'-7')	BEB, SS6 (12.5'-14.5')	TWB, SS4 (7.5'-9.5')	TWB, SS7 (15'-17')	TEB, SS3 (5'-7')	TEB, SS6 (12.5'-14.5')
Sample Matrix	Soil	Soil	Soil	Soil	Soil	Soil	Soil	Soil
Sample Date	10/10/2019	10/10/2019	10/10/2019	10/10/2019	10/10/2019	10/10/2019	10/10/2019	10/10/2019

Parameter	Units	RL	Result	Result	Result	Result	Result	Result	Result	Result
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Corrosivity Index

Corrosivity Index	none	1		5.5	14.5	4	11	11	9	14.5	1
Soil Redox Potential	mV	-		401	284	209	254	206	250	243	286
Sulphide	%	0.02		0.05	0.11	< 0.02	< 0.02	< 0.02	< 0.02	0.11	< 0.02
pH	pH Units	0.05		7.97	7.74	8.78	7.71	7.81	8.02	7.73	8.23
Resistivity (calculated)	ohms.cm	-9999		2860	483	7260	855	1480	1780	1120	5090

PACKAGE: - General Chemistry (SOIL)

Sample Number	5	6	7	8	9	10	11	12
Sample Name	BWB, SS4 (7.5'-9.5')	BWB, SS7 (15'-17')	BEB, SS3 (5'-7')	BEB, SS6 (12.5'-14.5')	TWB, SS4 (7.5'-9.5')	TWB, SS7 (15'-17')	TEB, SS3 (5'-7')	TEB, SS6 (12.5'-14.5')
Sample Matrix	Soil	Soil	Soil	Soil	Soil	Soil	Soil	Soil
Sample Date	10/10/2019	10/10/2019	10/10/2019	10/10/2019	10/10/2019	10/10/2019	10/10/2019	10/10/2019

Parameter	Units	RL	Result	Result	Result	Result	Result	Result	Result	Result
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General Chemistry

Conductivity	uS/cm	2		350	2070	138	1170	674	561	896	196
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PACKAGE: - Metals and Inorganics (SOIL)

Sample Number	5	6	7	8	9	10	11	12
Sample Name	BWB, SS4 (7.5'-9.5')	BWB, SS7 (15'-17')	BEB, SS3 (5'-7')	BEB, SS6 (12.5'-14.5')	TWB, SS4 (7.5'-9.5')	TWB, SS7 (15'-17')	TEB, SS3 (5'-7')	TEB, SS6 (12.5'-14.5')
Sample Matrix	Soil	Soil	Soil	Soil	Soil	Soil	Soil	Soil
Sample Date	10/10/2019	10/10/2019	10/10/2019	10/10/2019	10/10/2019	10/10/2019	10/10/2019	10/10/2019

Parameter	Units	RL	Result	Result	Result	Result	Result	Result	Result	Result
-----------	-------	----	--------	--------	--------	--------	--------	--------	--------	--------

Metals and Inorganics

Moisture Content	%	0.1		16.0	15.5	6.4	14.6	14.3	16.7	14.2	15.6
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FINAL REPORT

CA14406-OCT19 R1

Client: Peto MacCallum Ltd

Project: 19KF030A Hwy 401, Tilbury (B/T)

Project Manager: Nazibur Rahman

Samplers: Jinsuko

PACKAGE: - Metals and Inorganics (SOIL)

Sample Number	5	6	7	8	9	10	11	12
Sample Name	BWB, SS4 (7.5'-9.5')	BWB, SS7 (15'-17')	BEB, SS3 (5'-7')	BEB, SS6 (12.5'-14.5')	TWB, SS4 (7.5'-9.5')	TWB, SS7 (15'-17')	TEB, SS3 (5'-7')	TEB, SS6 (12.5'-14.5')
Sample Matrix	Soil	Soil	Soil	Soil	Soil	Soil	Soil	Soil
Sample Date	10/10/2019	10/10/2019	10/10/2019	10/10/2019	10/10/2019	10/10/2019	10/10/2019	10/10/2019

Parameter	Units	RL	Result	Result	Result	Result	Result	Result	Result	Result
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Metals and Inorganics (continued)

Sulphate	µg/g	0.4		320	2800	8.5	900	650	200	950	50
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PACKAGE: - Other (ORP) (SOIL)

Sample Number	5	6	7	8	9	10	11	12
Sample Name	BWB, SS4 (7.5'-9.5')	BWB, SS7 (15'-17')	BEB, SS3 (5'-7')	BEB, SS6 (12.5'-14.5')	TWB, SS4 (7.5'-9.5')	TWB, SS7 (15'-17')	TEB, SS3 (5'-7')	TEB, SS6 (12.5'-14.5')
Sample Matrix	Soil	Soil	Soil	Soil	Soil	Soil	Soil	Soil
Sample Date	10/10/2019	10/10/2019	10/10/2019	10/10/2019	10/10/2019	10/10/2019	10/10/2019	10/10/2019

Parameter	Units	RL	Result	Result	Result	Result	Result	Result	Result	Result
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Other (ORP)

Chloride	µg/g	0.4		53	42	17	99	33	290	42	16
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FINAL REPORT

CA14406-OCT19 R1

QC SUMMARY

Anions by IC
Method: EPA300/MA300-Ions1.3 | Internal ref.: ME-CA-IENVIIC-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	DIO0296-OCT19	µg/g	0.4	<0.4	1	20	99	80	120	106	75	125
Sulphate	DIO0296-OCT19	µg/g	0.4	<0.4	6	20	97	80	120	97	75	125

Carbon/Sulphur
Method: ASTM E1915-07A | Internal ref.: ME-CA-IENVIARD-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	ECS0023-OCT19	%	0.02	<0.02	3	20	116	80	120			

Conductivity
Method: SM 2510 | Internal ref.: ME-CA-IENVIEWL-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	EWL0294-OCT19	uS/cm	2	0.002	0	10	99	90	110	NA		



QC SUMMARY

pH
Method: SM 4500 | Internal ref.: ME-CA-|ENVIEWL-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	EWL0294-OCT19	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

Samples analysed as received. Solid samples expressed on a dry weight basis. "Temperature Upon Receipt" is representative of the whole shipment and may not reflect the temperature of individual samples.

Analysis conducted on samples submitted pursuant to or as part of Reg. 153/04, are in accordance to the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act" published by the Ministry and dated March 9, 2004 as amended.

SGS provides criteria information (such as regulatory or guideline limits and summary of limit exceedances) as a service. Every attempt is made to ensure the criteria information in this report is accurate and current, however, it is not guaranteed. Comparison to the most current criteria is the responsibility of the client and SGS assumes no responsibility for the accuracy of the criteria levels indicated. This document is issued, on the Client's behalf, by the Company under its General Conditions of Service available on request and accessible at http://www.sgs.com/terms_and_conditions.htm. The Client's attention is drawn to the limitation of liability, indemnification and jurisdiction issues defined therein. Any other holder of this document is advised that information contained hereon reflects the Company's findings at the time of its intervention only and within the limits of Client's instructions, if any. The Company's sole responsibility is to its Client and this document does not exonerate parties to a transaction from exercising all their rights and obligations under the transaction documents.

This report must not be reproduced, except in full. This report supersedes all previous versions.

-- End of Analytical Report --

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
for Design-Build Ready Alternative Bid Package
Widening of Baptiste Creek Bridges, Site Nos. 13X-0188/B1 & B2
Highway 401, Station 22+065, Township of Tilbury, Chatham-Kent, Ontario
G.W.P. 3034-19-00, Assignment No. 3017-E-0006/0007, Work Item No. 07
PML Ref.: 19KF030A, December 16, 2019



APPENDIX B

Previous Borehole Logs and Drawings (GEOCRES No. 40J08-018)

#59-F-206C

W.P. #164-58

HWY. #401

BAPTISTE





CREEK

CROSSING

TILBURY EAST

e. m. peto associates ltd.
SOIL ENGINEERING SERVICE - TORONTO, ONTARIO
BOREHOLE LOG

Job Name **Highway 401 - Baptiste Creek Crossing.** Job No. **58160** Borehole No. **1.**
 Client **Dep't. of Highways of Ontario** Casing **BX (2 1/2" Dia.)** Boring Date **Jan. 6th. - 12th, 1959.**
 Down **Geodetic.** Compiled By **M. Mindess.** Checked By **E.M. Peto.**

SAMPLE CONDITION		SAMPLE TYPE		ABBREVIATIONS	
	UNDISTURBED	S.S.	2" STANDARD SPLIT TUBE SAMPLE	Y.T.	IN SITU VANE SHEAR TEST
	FAIR	S.L.	SPLIT BARREL WITH LINERS	Q.U.	UNCONFINED COMPRESSIVE STRENGTH
	DISTURBED	S.T.	THIN-WALLED SHEET PILE SAMPLE	W.L.	WATER LEVEL IN CASING
	LOST	W.S.	WASH SAMPLE	W.T.	GROUND WATER TABLE IN SOIL
		R.C.	ROCK CORE		

SOIL DESCRIPTION	COLOUR	Consistency	Depth (Feet)	Sample No.	Notes
SILTY CLAY, GRITS AND PEBBLES, SOME FINE SAND.	MOTTLED GREY-BROWN.	STIFF	0' 0" - 5' 0"	1	S.S. 14 NAT. M.C.=17.5% DRIER THAN PLASTIC LIMIT.
CLAYEY AND SANDY SILT, GRITS AND PEBBLES	MOTTLED GREY-BROWN	COMPACT	5' 0" - 10' 0"	2	S.S. 20 MOIST, NON-PLASTIC. NATURAL M.C.=16.9%.
SILTY CLAY, GRITS AND PEBBLES.	GREY-BROWN.	STIFF	10' 0" - 15' 0"	3	S.S. 25 NAT. M.C.=18.9% AT PLASTIC LIMIT.
STRATIFIED SILTY CLAY, GRITS AND PEBBLES, THIN FINE SAND SEAM.	BROWNISH-GREY.	VERY STIFF	15' 0" - 20' 0"	4	S.S. 32 NAT. M.C.=20.0% DRIER THAN PLASTIC LIMIT.
STRATIFIED SILTY CLAY, GRITS AND PEBBLES.	DARK GREY	STIFF	20' 0" - 25' 0"	5	S.S. 27 NAT. M.C.=16.6% AT PLASTIC LIMIT.
STRATIFIED SILTY CLAY, BLACK GRITS.	GREY	STIFF	25' 0" - 30' 0"	6	S.S. 22 NAT. M.C.=15.2% WETTER THAN PLASTIC LIMIT.
AS ABOVE, NUMEROUS GRITS.	GREY	STIFF	30' 0" - 35' 0"	7	2" S.L. TAPPED
			35' 0" - 40' 0"	8	S.S. 23 WETTER THAN PLASTIC LIMIT.
SILTY CLAY, GRITS.	GREY	FIRM	40' 0" - 45' 0"	9	S.S. 14 NAT. M.C.=21.0% MUCH WETTER THAN PLASTIC LIMIT.
STRATIFIED SILTY CLAY, GRITS	GREY	FIRM	45' 0" - 50' 0"	10	2" S.L. PUSHED
			50' 0" - 55' 0"	11	S.S. 14 MUCH WETTER THAN PLASTIC LIMIT.
SILTY CLAY, GRITS	GREY	FIRM	55' 0" - 60' 0"	12	2" S.L. PUSHED
			60' 0" - 65' 0"	13	2" S.L. PUSHED
AS ABOVE	"	"	65' 0" - 70' 0"	14	S.S. 10 MUCH WETTER THAN PLASTIC LIMIT.
			70' 0" - 75' 0"	15	2" S.L. PUSHED
			75' 0" - 80' 0"	16	S.S. 7 AS ABOVE. SPECIFIC GRAVITY=2.76
HOLE TERMINATED NO STIFFENING OR REFUSAL.					

508 ENGINEERING SERVICE - TORONTO, ONTARIO

Highway 401 -
Job Name Baptiste Creek Crossing. Job No. 58160
Client Dep't. of Highways of Ontario. Drawing BX (2 1/2" Dia.)
Nature Geodetic. Prepared by M. Mindess.

2.
Jan. 12th. - 15th. 1959.
E.M. Peto.

[illegible]

AB25191A1;049

$$\begin{aligned} & \mathcal{S}_1 \mathcal{S}_2: \mathcal{S}^2 \times \mathcal{S}^2 \rightarrow \mathcal{S}^2 \times \mathcal{S}^2, (x, y) \mapsto (x, y) \\ & \mathcal{S}_1 \mathcal{S}_3: \mathcal{S}^2 \times \mathcal{S}^2 \rightarrow \mathcal{S}^2 \times \mathcal{S}^2, (x, y) \mapsto (x, y) \\ & \mathcal{S}_1 \mathcal{S}_4: \mathcal{S}^2 \times \mathcal{S}^2 \rightarrow \mathcal{S}^2 \times \mathcal{S}^2, (x, y) \mapsto (x, y) \\ & \mathcal{S}_2 \mathcal{S}_3: \mathcal{S}^2 \times \mathcal{S}^2 \rightarrow \mathcal{S}^2 \times \mathcal{S}^2, (x, y) \mapsto (x, y) \\ & \mathcal{S}_2 \mathcal{S}_4: \mathcal{S}^2 \times \mathcal{S}^2 \rightarrow \mathcal{S}^2 \times \mathcal{S}^2, (x, y) \mapsto (x, y) \\ & \mathcal{S}_3 \mathcal{S}_4: \mathcal{S}^2 \times \mathcal{S}^2 \rightarrow \mathcal{S}^2 \times \mathcal{S}^2, (x, y) \mapsto (x, y) \end{aligned}$$
$$\begin{aligned} \text{例 1.1} \quad & \text{设 } f(x) = x^2 + 2x + 1, g(x) = x^2 + 1, \text{ 求 } f(x) \text{ 与 } g(x) \text{ 的 gcd.} \\ \text{解} \quad & f(x) - g(x) = (x^2 + 2x + 1) - (x^2 + 1) = 2x, \text{ 故 } (f(x), g(x)) = (2x, x^2 + 1) \\ \text{由 1.1,} \quad & (2x, x^2 + 1) = (2, x^2 + 1) \text{ 且 } (2, x^2 + 1) = 1, \text{ 故 } (f(x), g(x)) = 1. \\ \text{例 1.2} \quad & x^2 + 1 \text{ 与 } x^2 + x + 1 \text{ 互素吗?} \\ \text{解} \quad & (x^2 + 1, x^2 + x + 1) = (x^2 + 1, (x^2 + x + 1) - (x^2 + 1) = x) \\ & = (x^2 + 1, x) = (1, x) = 1. \end{aligned}$$

DEPTH (ft)	SOIL DESCRIPTION	COLOR	STATE	TESTS	MOISTURE CONTENT (%)	PLASTICITY INDEX (%)	REMARKS
0.0	ORGANIC, SANDY AND CLAYEY SILT	GREY-BLACK	FIRM	1	16.0	12	NAT. M.C. = 16.0% DRIER THAN PLASTIC LIMIT
1.0	CLAYEY SILT, MANY GRITS, MOTTLED MINOR ORGANIC CONTENT	GREY-BROWN	FIRM	2	16.1	21	NAT. M.C. = 16.1% DRIER THAN PLASTIC LIMIT
5.0	AS ABOVE	AS ABOVE	STIFF	2			
10.0	STRATIFIED SILTY CLAY, GRITS, OCCASIONAL THIN SAND SEAMS	BROWNISH-GREY	STIFF	3	16.9	20	NAT. M.C. = 16.9% DRIER THAN PLASTIC LIMIT
15.0				4			2" S.L. TAPPED
20.0	STRATIFIED SILTY CLAY, MANY BLACK GRITS	GREY	FIRM	5	20.3	12	NAT. M.C. = 20.3% WETTER THAN PLASTIC LIMIT
25.0				6			2" S.L. PUSHED
30.0	SILTY CLAY, GRITS, INDISTINCTLY STRATIFIED	GREY	FIRM	7	20.0	11	NAT. M.C. = 20.0% WETTER THAN PLASTIC LIMIT
35.0				8			2" S.L. PUSHED
40.0	SILTY CLAY, GRITS	GREY	FIRM	9	22.7	10	SPECIFIC GRAVITY = 2.75 NAT. M.C. = 22.7% MUCH WETTER THAN PLASTIC LIMIT
45.0				10			2" S.L. PUSHED
50.0	AS ABOVE	GREY	SOFT	11	27.7	6	NAT. M.C. = 27.7% MUCH WETTER THAN PLASTIC LIMIT
55.0				12			2" S.L. PUSHED
58.0				13			2" S.L. PUSHED
60.0				14			2" S.L. PUSHED
65.0	AS ABOVE	GREY	FIRM	15	24.2	10	NAT. M.C. = 24.2% MUCH WETTER THAN PLASTIC LIMIT
70.0				16			2" S.L. PUSHED
75.0	SILTY CLAY, GRITS	GREY	FIRM	17	24.5	9	SMALL NATURAL GAS POCKET ENCOUNTERED WHEN HOLE FIRST REACHED 71' 6" ALL THE PRESSURE HAD DISSIPATED BY THE NEXT MORNING NAT. M.C. = 24.5%
80.0				18			2" S.L. PUSHED
81.6	SILTY CLAY, BLACK SHALE FRAGMENTS UP TO 2" SIZE	GREY		19			2" S.L. TAPPED MUCH WETTER THAN PLASTIC LIMIT
85.0				20			2" S.L. TAPPED
90.0	SANDY AND SILTY CLAY	GREY TO GREY-BLACK	FIRM	21		13	
95.0	SILTY CLAY, GRITS	GREY	FIRM	22		10	
95.0	SILTY CLAY, BLACK SHALE FRAGMENTS TO 1" SIZE	GREY	FIRM	22		11	MUCH WETTER THAN PLASTIC LIMIT
101.0	SILTY CLAY, GRITS	GREY, SOME POCKETS BROWNISH-RED	FIRM	23	33.3	10	MUCH WETTER THAN PLASTIC LIMIT NAT. M.C. = 33.3%
482.4	HOLE TERMINATED NO STIFFENING OR REFUSAL						

BOREHOLE LOG

Borehole No. 3.
Boring Date Jan. 13th. - 14th. 1959.
Checked By M. Mindess

SAMPLE TYPE

ABBREVIATIONS

UNDISTURBED

FAIR

DISTURBED

LOST

S. L. SPLIT BARREL WITH LINERS

S. T. THIN-WALLED SHELL BY TUBE SAMPLE

W.S. WASH SAMPLE

R. C. ROCK CORE

V T. IN SITU VANE SHEAR TEST

Q.4 UNCONFINED COMPRESSIVE STRENGTH

W. L. WATER LEVEL 'N CASING

W. T. GROUND WATER TABLE IN SOIL

[illegible]

BOREHOLE LOG

Borehole No. 4.
Boring Date Jan. 12th. - 13th. 1950.
Checked By M. Mindess.

SAMPLE TYPE

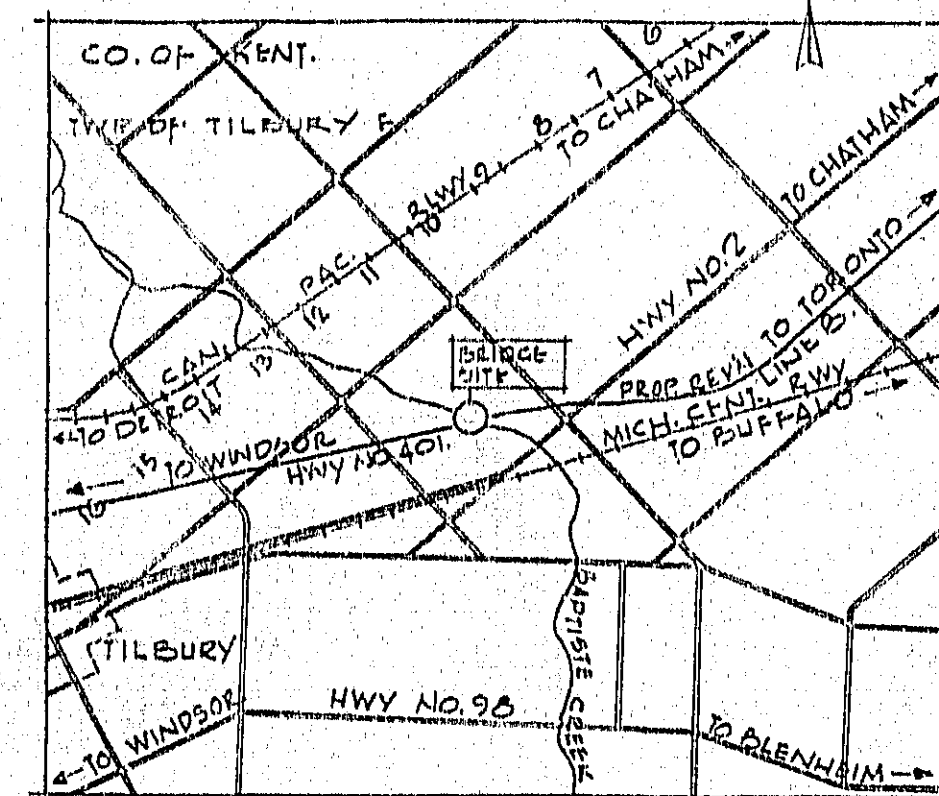
ABBREVIATIONS

LOST

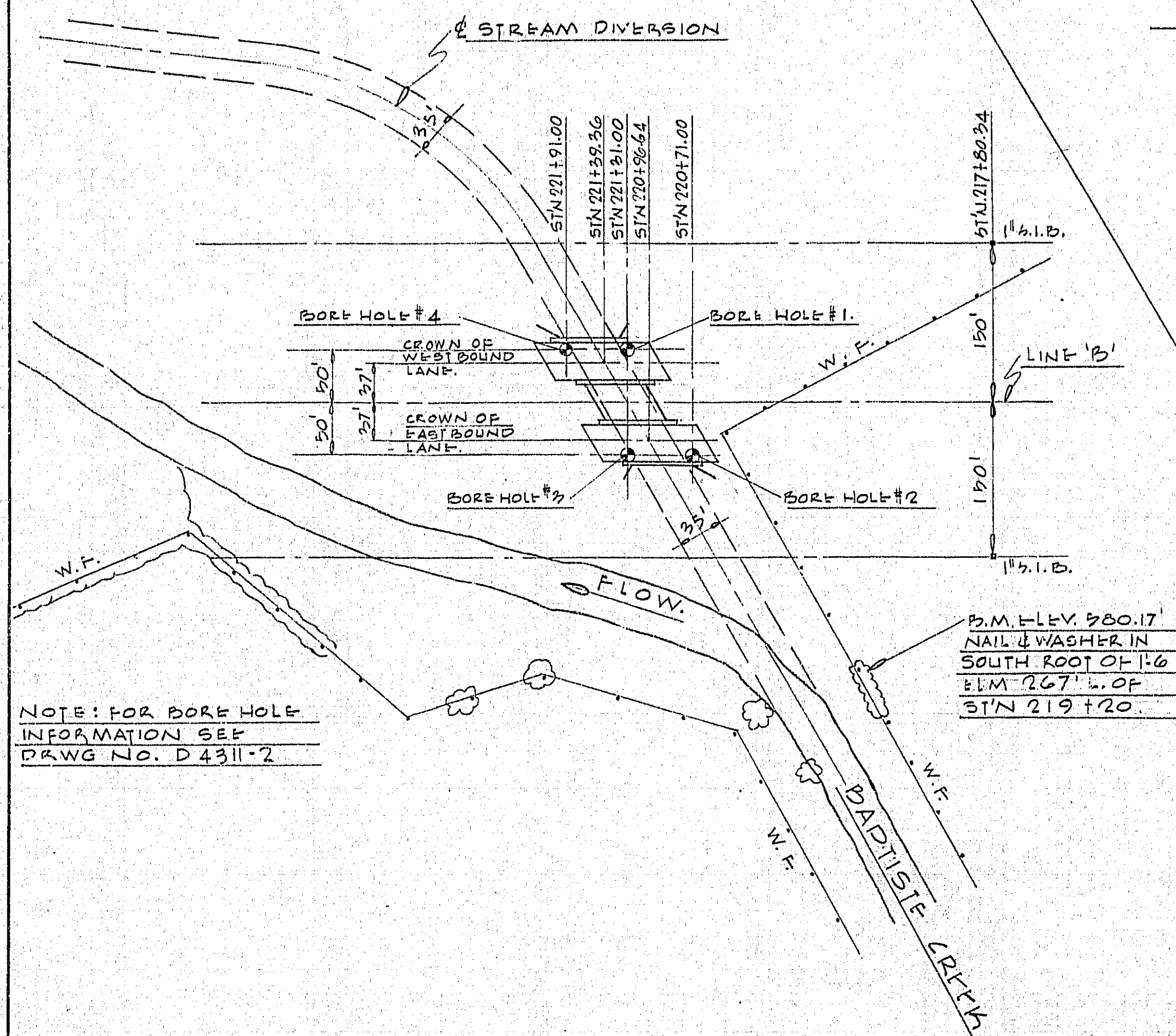
S.S. 2" STANDARD SPLIT TUBE SAMPLE
S.L. SPLIT BARREL WITH LINERS
S.T. THIN-WALLED SHELBY TUBE SAMPLE
W.S. WASH SAMPLE
R.C. ROCK CORE

V. T. IN SITU VANE SHEAR TEST
Q/u UNCONFINED COMPRESSIVE STRENGTH
W. L. WATER LEVEL IN CASING
W. T. GROUND WATER TABLE IN SOIL

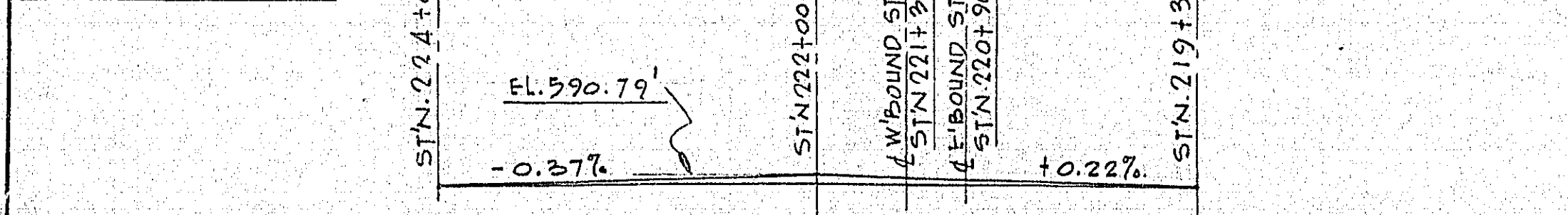
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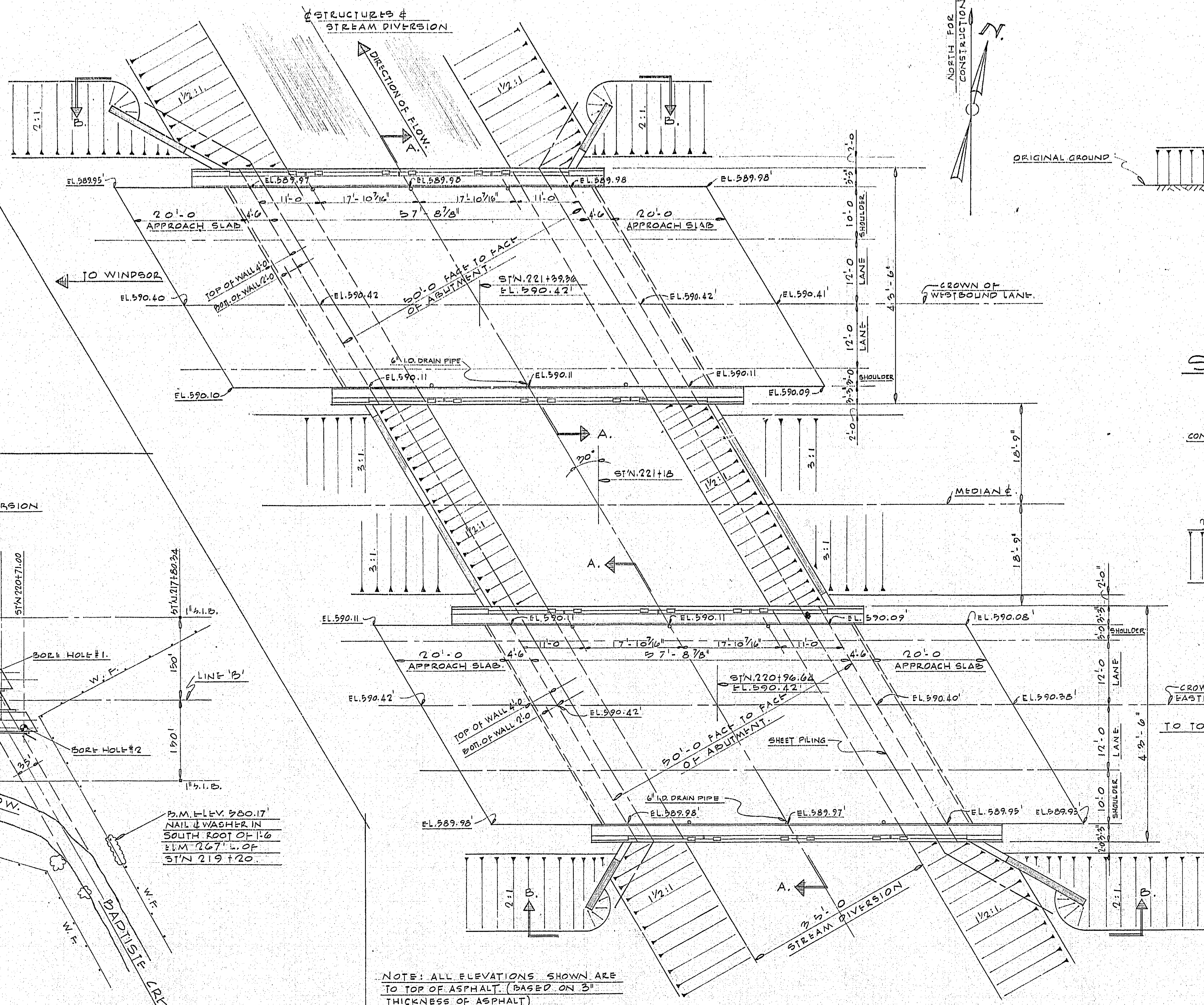
KEY PLAN
1 INCH = 1 MILE



NOTE: FOR BORE HOLE INFORMATION SEE DRAWG. NO. D-4311-2



FINISHED ROAD PROFILE AT CROWN OF HWY 401
SCALE: 1" = 10' VERTICAL, 1" = 100' HORIZONTAL

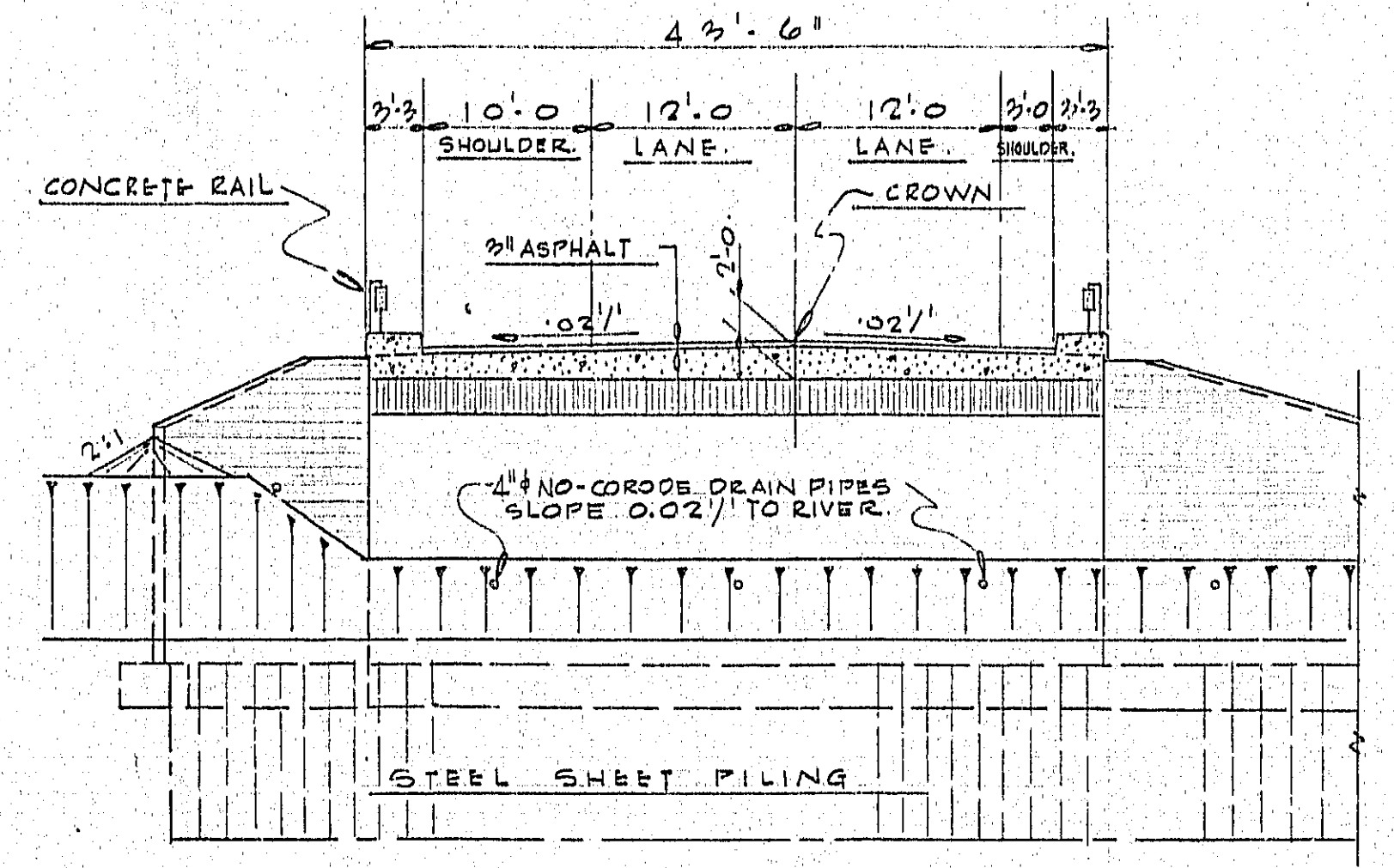
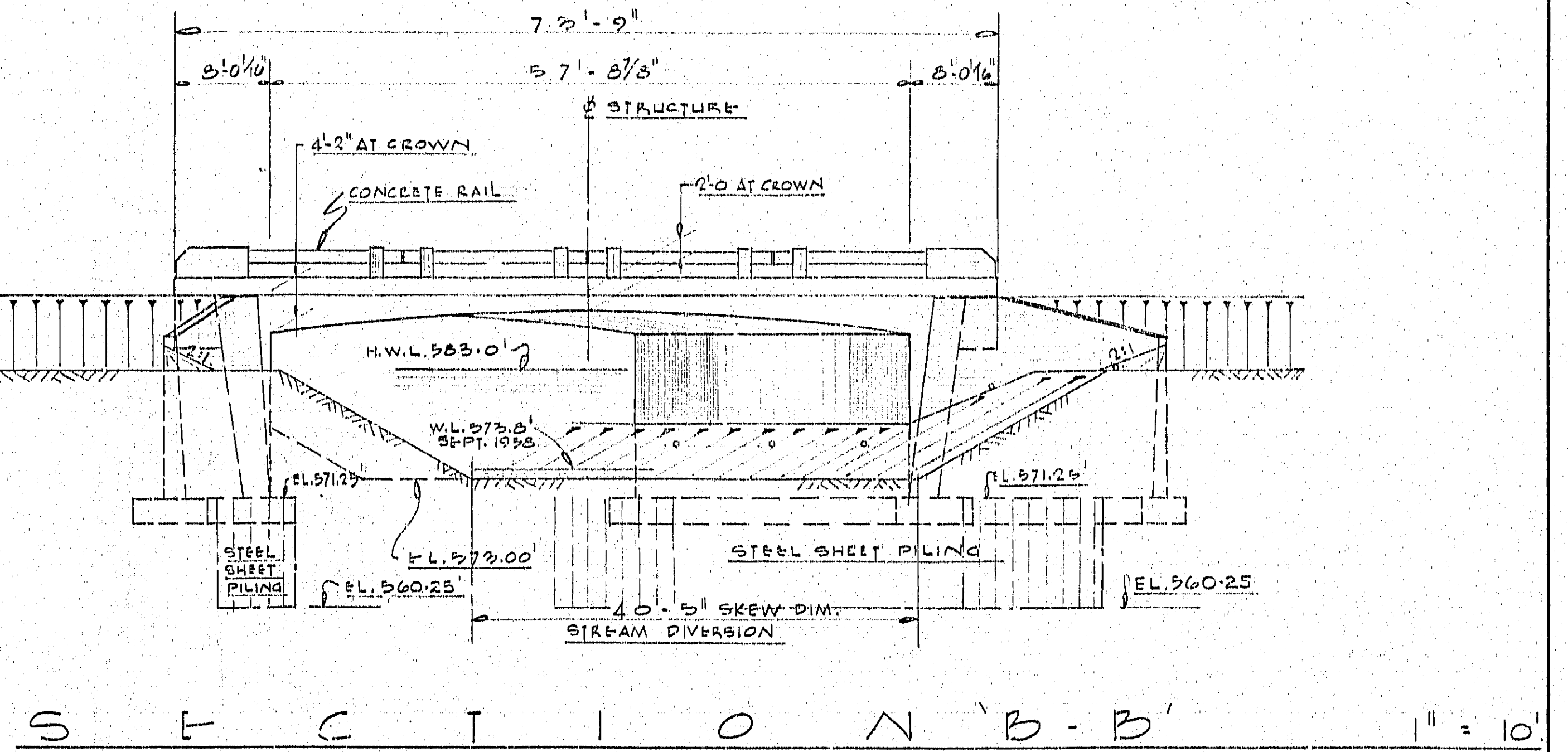


NOTE: ALL ELEVATIONS SHOWN ARE TO TOP OF ASPHALT (BASED ON 3" THICKNESS OF ASPHALT)

NOTE TO DISTRICT ENGINEER: CONCRETE WORK ON THIS STRUCTURE MUST NOT BE COMMENCED UNTIL MONUMENTS TO FIX CONTROL POINTS HAVE BEEN ERECTED AND CHECKED BY THE DISTRICT ENGINEER. NOTE TO CONTRACTOR: STRUCTURE TO BE BUILT IN ACCORDANCE WITH FORM NO. 9 AND THE SPECIAL PROVISIONS, EXTRA COPIES OF WHICH MAY BE OBTAINED FROM THE DISTRICT ENGINEER. ALL CONSTRUCTION JOINTS MUST BE APPROVED BY THE BRIDGE ENGINEER.

INDEX TO DRAWINGS

D-4311-1	GENERAL ARRANGEMENT
D-4311-2	BRIDGE FRAME DIMENSIONS & SOIL BORING DATA
D-4311-3	FOUNDATION PLAN
D-4311-4	ABUTMENT ELEV. & DECK CROSS SECTION
D-4311-5	DECK PLAN, DETAILS & LONGITUDINAL SECTION
D-4311-6	RETAINING WALLS
D-4311-7	REINFORCING SCHEDULE NO. 1
D-4311-8	REINFORCING SCHEDULE NO. 2
D-4311-9	REINFORCING SCHEDULE NO. 3
D-4311-10	REINFORCING SCHEDULE NO. 4



GENERAL NOTES:
1. ALL CONCRETE TO DEVELOP A COMPRESSIVE STRENGTH OF 3000 P.S.I. AT 28 DAYS.
2. AN ADMIXTURE IS TO BE ADDED TO THE CONCRETE AS SPECIFIED BY MATERIALS RESEARCH SECTION D.H.O.
3. MAX. CONC. AGGREGATE SIZE TO BE 1 1/4".
4. CONC. COVER TO REINFORCEMENT: UNDERSIDE DECK SLAB 1 1/2" TOP OF DECK SLAB 1 1/2" INTERNAL & EXTERNAL FACES OF ABUTMENT WALLS & RETAINING WALLS 2" FOOTINGS IN CONTACT WITH SOIL 3"
5. ALL EXPOSED EDGES OF CONC. TO HAVE A 1" CHAMFER EXCEPT AS NOTED.
6. NO FILL SHALL BE PLACED BEHIND ABUTMENTS UNTIL COMPLETE DECK SLAB HAS BEEN POURED AND HAS SET.
7. FOR PROPERTIES OF SKEW ANGLE SEE DRAWG. D-4311-3.

WP 164-58

A.D. MARGISON AND ASSOCIATES LIMITED
CONSULTING PROFESSIONAL ENGINEERS
TORONTO CANADA

DEPARTMENT OF HIGHWAYS-ONTARIO
BRIDGE OFFICE-TORONTO

TILBURY EAST TOWNSHIP
BRIDGE NO. 8 OVER BAPTISTE CREEK

THE KING'S HIGHWAY No. 401 DIST. No. 1
CO. OF KENT
TWP. EAST TILBURY LOT II CON. V

GENERAL ARRANGEMENT

APPROVED: *Am L*
BRIDGE ENGINEER

DESIGN ENGINEER

DESIGN	E.S.	CHECK	G.P.M.	CONTRACT NUMBER	59-304-61-09	59-332
DRAWING	G.L.H.	CHECK	E.S.	LOADING	H20-S16	D-4311-1
DATE	MAY	1959				

7WP 104-188-1-A 13-188 D-4311 1 to 10



APPENDIX C

List of Standard Specifications Relevant to Report



LIST OF STANDARD SPECIFICATIONS RELEVANT TO REPORT

DOCUMENT	TITLE
OPSS.PROV 405	Construction Specification for Pipe Subdrains
OPSS 501	Construction Specification for Compacting
OPSS.PROV 511	Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting
OPSS.PROV 539	Temporary Protection Systems
OPSS 902	Excavation and Backfilling of Structures
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material
OPSD 3090.101	Foundation, Frost Penetration depths for Southern Ontario
OPSD 3101.150	Walls, Abutment, Backfill, Minimum Granular Requirement
OPSD 3121.150	Wall, Retaining, Backfill, Minimum Granular Requirement
SP 105S09	Amendment to OPSS 539, November 2014
SP 109F57	Amendment to OPSS 903, April 2016
SP 109S12	Amendment to OPSS 902, November 2010
NSSP FOUN0003	Dewatering Structure Excavations, Amendment to OPSS 902