



**DETAIL DESIGN FOUNDATION INVESTIGATION AND DESIGN
REPORT**

for

WIDENING OF QUEEN STREET OVERPASSES

Site Nos. 06X-0051/B1 & B2

Highway 401 – Station 17+860

Township of Tilbury, Chatham-Kent, Ontario

GWP 3034-19-00, WP 3041-19-01 & 3043-19-01

Assignment No. 3017-E-0006/0007

Work Item No. 06

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DETAIL DESIGN FOUNDATION INVESTIGATION AND DESIGN REPORT

Widening of Queen Street Overpasses
Site No. 06X-0051/B1 & B2
Highway 401 – Station 17+860
Township of Tilbury, Chatham-Kent, Ontario
G.W.P. 3034-19-00, Assignment No. 3017-E-0006/0007, Work Item No. 06

PART A – DETAIL DESIGN FOUNDATION INVESTIGATION PORTION OF 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 two sites on Highway 401 under MTO Assignment No. 3017-E-0006/0007, Work Item No. 06. 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 MTO Assignment No. 3017-E-0006/0007, Work Item No. 06.

This report is a Detail Design Foundation Investigation and Design Report for Queen Street EBL and WBL Overpasses located along Highway 401 at the crossing of Queen Street in the Township of Tilbury, Chatham-Kent, Ontario. The subsurface investigation was limited to previously available boreholes supplemented by an additional two (2) boreholes drilled for the current assignment.

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 residential, commercial and agricultural developments, and is located approximately 1.2 km north of the residential community of Tilbury.

3. FIELD INVESTIGATION PROCEDURES

The field work for the current foundation investigation involved two (2) boreholes drilled to supplement subsurface information from the previous investigation. The new boreholes are identified as QEB and QWB, located within the Highway 401 median, east and west of Queen Street Overpass, respectively. The boreholes were drilled to depth 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		
QEB	4 681 197.2	310 477.4	42.270321	-82.431176	30.0	182.5
QWB	4 681 187.4	310 434.6	42.270233	-82.431695	30.0	182.5

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 before the commencement of drilling work.

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

The equipment used for the borehole drilling work 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 QEB and QWB were drilled between October 1 and 4, 2019. The boreholes were advanced using a D50-Turbo Track-mounted drilling rig equipped with 200 mm diameter hollow stem augers.

Refer to Drawings QS-1 and QS-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.

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

4. LABORATORY TEST PROCEDURES

Laboratory testing was carried out on representative SPT samples recovered during the fieldwork at PML's laboratory facility in Toronto, Ontario. The laboratory testing program included the following:

- Natural moisture content determinations (42)
- Grain size distribution analyses (12)
- Atterberg limit tests (11)
- Consolidation test (1)

All the laboratory tests to determine 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-QS-1 to GS-QS-4. The results of the Atterberg Limit tests are presented in Figures PC-QS-1 to PC-QS-3. One-dimensional consolidation (ASTM D-2435) test was conducted on one Shelby tube sample from borehole QWB and the results are presented in



Figure Q-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 5 and 16, 1959, and consisted of three (3) boreholes drilled to depths ranging from 12.7 m (EL. 164.3) to 14.2 m (EL. 162.8) below the ground surface existed at the time of investigation. Fourth borehole was advanced adjacent to Borehole 3 to a maximum depth of 23.5 m (EL. 153.3), to verify whether the subsoil conditions were similar to that was encountered in other three boreholes. This borehole was not sampled and there is no Record of Borehole sheet. All three sampled boreholes were complemented with Dynamic Cone Penetration Test (DCPT) adjacent to these boreholes and a fourth DCPT located in northwest corner. Based on the foundation investigation and design report (Geocres No. 40J08-002), representative soil samples were recovered from the boreholes at 1.5 m intervals to the termination depth of the boreholes, using a conventional 51 mm OD split spoon sampler, simultaneously conducting Standard Penetration Tests (SPT) to assess the strength characteristics of the substrata. In addition, 51 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. The report indicates that complex tests on undisturbed samples consisted of one-dimensional consolidation and shear strength measurements. However, laboratory records are not available with the report to evaluate or verify the test results.



Based on the previous investigation, the subsoil conditions in the area of the proposed structure is expected to consist of about 300 mm of topsoil followed by upper stiff to hard brown silty clay deposit to a depth of 7.6 m (EL. 169.2), which is underlain by soft to stiff grey silty clay stratum to the maximum borehole depth of 23.5 m (EL. 153.3). The upper most part of this silty clay deposit to a depth of about 7.6 m (EL. 169.2) appears to be desiccated and the SPT 'N'-values reported were ranging from 9 blows to as high as 45 blows, indicating stiff to hard consistency. Below these depths, the SPT 'N'-values reported range from as low as 2 blows to 16 blows, indicating soft to stiff consistency.

Based on the report, groundwater was not encountered in any of the three boreholes during and upon completion of drilling. The water level of Lake St. Clair at the time of investigation was at approximate EL.175.0

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 QS-1 and QS-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 fill, approximately 6.7 m thick, composed of gravelly sand and followed by clayey silt to silty clay, with varying proportions of sand and gravel, which is underlain by approximately 23.5 m thick deposit of very stiff to stiff silty clay to clayey silt till deposit. Boreholes QEB and QWB were terminated to 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) Gravelly Sand, Trace Silt, Trace Clay
 - ii) Silty Clay to Clayey Silt, some Sand, Trace Clay
- b) Silty Clay to Clayey Silt, Some Sand, Trace Gravel (Till)

5.3.1 Fill

i) Gravelly Sand, Trace Silt, Trace Clay

A layer of gravelly sand fill is encountered just below the existing ground surface. The layer extends to depth of 3.1 m (EL.179.4) to 4.6 m (EL.177.9) below the existing ground surface in boreholes QEB and QWB, respectively.

The SPT 'N'-values recorded in this fill ranged between 7 and 23 blows, indicating loose to compact state of compactness. The moisture content of samples tested from this fill ranged between 5.7% and 9.2%.

The grain size distribution result of selected sample from borehole QWB is provided on Figure GS-QS-2 in Appendix A. Sieve analysis test was performed on representative sample from this fill and the test result indicates that this fill consists of 16% gravel, 73%% sand, 7% silt, and 4% clay size particles.

ii) Silty Clay to Clayey Silt, Some Sand, Trace Gravel

A layer of silty clay to clayey silt fill is encountered below the gravelly sand fill in both boreholes. This layer extends to depth of 6.7 m (EL.175.8) below the existing ground surface in both boreholes. A thin layer of silty sand is encountered in the bottom of silty clay fill in borehole QWB at a depth of 6.4 m (EL.176.1) and extends to a depth of 6.7 m (EL. 175.8) from the existing ground surface.

The SPT 'N'-values recorded in this fill ranged between 6 and 10 blows, indicating firm to stiff consistency. The moisture content of samples tested from this fill ranged between 18.3% and 27.1%.

The grain size distribution results of selected silty clay samples from this fill are provided on Figures GS-QS-1, and the results of Atterberg limits for the same samples are provided on Figures PC-QS-1 in Appendix A.



Sieve analysis tests were performed on two (2) representative samples and the test results indicate that this fill consists of 2 to 6% gravel, 17% sand, 37% silt, and 40% to 44% clay. Atterberg limit tests were performed on two (2) representative samples and the test results indicate liquid limit values were 41 and 47, plastic limit values were 21 and 23, and corresponding plasticity index values were 20 and 24. Based on the test results, the clayey soil may be classified as clay of medium plasticity (CI) in the Unified Soil Classification System (USCS), i.e., silty clay and classified as silty clay in the MTO Soil Classification.

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

The fill in boreholes QEB and QWB is underlain by this silty clay to clayey silt till with varying proportions of sand and gravel. This till deposit is encountered at depth of 6.7 m (EL.175.8) and extends to the termination depths of 30.0 m (EL. 152.5) below the existing ground surface. The SPT 'N'-values in this deposit generally varies from 8 to 16 blows from EL. 175.8 to EL. 170.0, indicating stiff to very stiff consistency. Between EL. 170.0 and EL. 152.5, the SPT 'N'-values ranges from 4 to 10 blows. Within this depth, in-situ vane shear tests were carried out. The test was performed at thirteen (13) locations and the uncorrected vane shear strengths (C_u) measured ranged from 77 kPa to more than 120 kPa, with a sensitivity ratio value between 1 and 2, indicating stiff to very stiff consistency, compared to firm to stiff based on SPT 'N'-values. Generally, from El. 175.8 to El. 165.5, the C_u measured was more than 100 kPa, indicating very stiff consistency. Below El. 16.5, the C_u measured typically ranged between 77 kPa and 100 kPa, indicating stiff consistency. The moisture content of the samples tested from this till deposit were between 18.4% and 24.9%.

The grain size distribution results of selected silty clay to clayey silt samples from this till deposit are provided on Figures GS-QS-3 and GS-QS-4, and the results of Atterberg limits for the same samples are provided on Figures PC-QS-2 and PC-QS-3 in Appendix A.

Sieve analysis tests were performed on nine (9) representative samples and the test results indicate that this deposit consists of 1 to 4% gravel, 15% to 20% sand, 37% to 41% silt, and 38% to 45% clay. Atterberg limit tests were performed on nine (9) representative samples and the test results indicate liquid limit values range from 34 to 39, plastic limit values of 18, and corresponding plasticity index values range from 16 to 21. 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 in the MTO Soil Classification.

One-dimensional consolidation testing was conducted on one Shelby tube sample obtained from borehole QWB 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 2.709. Bulk unit weight of sample determined was 19.9 kN/m^3 and corresponding dry unit weight of the sample was 16.2 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)
QWB/14.0	183	365	2.0	0.637	0.170

5.3.3 Groundwater

Groundwater was not encountered in boreholes QWB and QEB during and upon completion of drilling.

A monitoring well consisting of 50 mm diameter PVC pipe was installed adjacent to boreholes QEB and QWB. 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)		
				2019/10/11	2019/10/24	2019/10/28
QEB	182.5	183.2	7.8 (EL. 174.7)	Dry	7.0 (EL. 175.5)	7.0 (EL. 175.5)
QWB	182.5	183.2	7.8 (EL. 174.7)	Dry	4.8 (EL. 177.7)	5.6 (EL. 176.9)



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 (µS/cm)	SULPHATE (µg/g)	CHLORIDE (µg/g)
QWB	4	7.5	0.03	452	8.62	8870	113	3.2	11
QWB	6	4.0	<0.02	416	8.61	5690	176	6.9	26
QEB	5	6.5	0.10	290	8.33	2160	464	290	130
QEB	7	9.5	0.08	275	7.75	2060	485	270	100

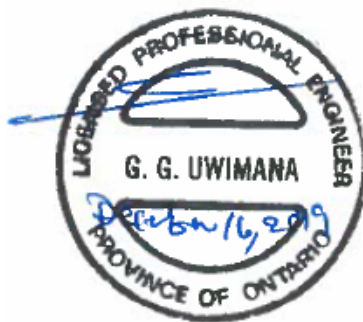
The Foundation Investigation portion of the report was prepared by Mr. K. Amatya, P.Eng. and Mr. N. Rahman P. Eng., Project Engineers and reviewed by Mr. G. Uwimana, MEng, P.Eng., Senior Engineer. Mr. R. Ng, MBA, PhD, P.Eng. Principal Consultant conducted an independent review of the report.

Yours very truly

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PART B – DETAIL FOUNDATION DESIGN PORTION OF REPORT

6. PROJECT DESCRIPTION

6.1 General

The Ministry of Transportation Ontario has proposed to widen 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.

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 Queen Street in the Town of Tilbury, Ontario. Based on the GA drawings, the initial proposal is 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 foundation investigation and design report with the interpretation and recommendations are intended for the use of 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 and Reinforcement Detail Drawings D 4422-1 and D 4422-3, dated January 1960, the existing bridges on EBL and WBL are rigid frame structures with a clear span of 16.5 m and supported on 2.1 m wide strip footings placed at about EL. 175.0. The footings were designed to carry an allowable bearing pressure of approximately 240 kPa. Both bridges



consist of about 4.6 m high toe walls at all four corners of the bridges to retain the approach fill. The toe walls are also placed at EL. 175.0 and supported on 2.4 m wide footings. The approach slabs at both bridge locations are about 6.1 m long.

The abutment walls of both bridges appear in good conditions and no major cracks, other than few surficial cracks were noticed on the abutment and toe walls. In addition, based on the PML Foundation Technical Memorandum dated June 17, 2015, it was observed that the slopes on both sides of the abutments were heavily vegetated at the time of site visit on October 20, 2013. The embankment slopes were observed to be in stable condition and no surficial erosion were noted.

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 16.5 m. It is proposed to support the widened portion of the new abutments on piles. The pile cap is shown to be founded EL. 175.0, the same as the existing strip footings. The design grade of the approaches at the east and west abutments will be approximately EL. 179.7.

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 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	Potential vibration induced during driving Potential requirement to design for corrosion protection Closed-end pipe pile is	Steel piles may require corrosion protection, in which case the corrosion protection would need to be designed	Moderate



Table 7.2: Comparison of Foundation Options

FOUNDATION TYPE	ADVANTAGES	DISADVANTAGES	RISKS / CONSEQUENCES	RELATIVE COST
	caps	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	by specialists.	
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 as 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 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. 172.0, and driven to the tip elevations suggested in Table 7.2.1a.

The construction of the deep foundation should conform to OPSS.PROV 903 and 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 (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	157.0	280	210
	18	154.0	330	250
HP 310 x 110 Steel H-Piles	15	157.0	340	260
	18	154.0	420	320
HP 310 x 79 Steel H-Piles	15	157.0	310	230
	18	154.0	380	290

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 Coefficient of Horizontal Subgrade Reaction

SOIL BOUNDARY ELEVATION		SOIL TYPE	UNDRAINED SHEAR STRENGTH (kPa)	n_h Values (kN/m ³)
FROM	TO			
172.0	165.5	Very stiff silty clay to clayey silt	110	-
165.5	157.0	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 Factor (After Davisson, 1970)¹

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. 175.0. The existing ground level in the area of the median appears to be same as that of the road (EL. 183.0) and the construction of footings for the proposed abutments may require about 5.5 m to 6.0 m deep excavation 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 at approximate elevation 175.0. The geotechnical resistances provided on Table 7.2.2 for a 2.2 m wide footing may be considered for the design of the proposed widening overpasses.

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	175.0	370	200
West Abutment			



The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.10.4 of the Canadian Highway Bridge Design Code (CHBDC, 2014). The previous foundation report from 1959 predicted settlements in the order of 76 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 factored total settlement of new footings under the recommended SLS loads could be in the order of 50 mm.

A modulus of subgrade reaction for the soil at the founding level (EL. 175.0) of footing may be estimated at 15,000 kN/m³ for the design purposes.

The existing footings are in place for almost sixty (60) years and differential settlements are expected between the new and existing structures. It is anticipated that the settlement under the existing footings may have ceased over the almost sixty (60) years. The differential settlement is expected to be in the order of 50 mm between the existing and new footings. 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 Overpasses

For the purpose of overpass rehabilitation (without addition of new median load) and evaluation of existing footings, the previous assessment in Section 6.2 of the Foundation Technical Memorandum for Queen Street EBL Overpass (Geocres No. 40J08-002), dated June 17, 2015, prepared by PML, has been re-evaluated. An examination of the current practice and CHBDC (2014), we suggest the design values to be updated to factored resistances of 280 kPa at SLS and 525 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).

The footings for the proposed bridge are expected to be placed along the longitudinal axes of the existing footings with the length of each new footing running parallel to the longitudinal axes 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 2.2 m with an approximate length (L) of 12.5 m. Assuming a footing length to width ratio (L/B) of 5.7, 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 280 kPa at SLS and 525 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.

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.



7.3 Approach Embankments

Based on the GA drawing, the existing ground elevation of the east and west approaches is at about EL. 183.0. 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 CHBDC (2014). The lateral earth pressure, p (kPa), may be computed 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)

γ = Unit weight of backfill material above assumed water level (kN/m³)

γ' = Unit weight of submerged backfill ($\gamma_{\text{sat}} - \gamma_w$) material below assumed water level (kN/m³)

γ_w = Unit weight of water (9.8 kN/m³)

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
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 μ m to 250 μ m) to prevent migration of



finer 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 Frost Protection

All pile caps or footings shall be provided with a minimum of 1.0 m of earth cover or 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. 175.0, about 5.5 m to 6.0 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. 175.0. 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. 180.0. 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 3.2 µg/g to as high as 290 µg/g (0.00032% to 0.029%). 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 11 µg/g to 130 µg/g (0.0011% to 0.013%). 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 samples ranged from 2060 ohm-cm to 8870 ohm-cm. Borehole QEB samples test results suggest that a moderate corrosive environment exists for steel elements in contact with soil and borehole QWB samples test results suggest that low to very low corrosive environment exists for steel elements in contact with soil. The pH values of fill samples ranged from 7.75 to 8.62.



11. CLOSURE

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

Yours very truly

Peto MacCallum Ltd.



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APPENDIX A

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

Explanation of Terms Used in Report

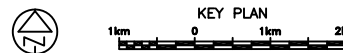
Record of Borehole Sheets

Results of Grain Size Distribution Analyses – Figures GS-QS-1 to GS-QS-4

Results of Atterberg Limit Tests – Figures PC-QS-1 to PC-QS-3

Consolidation Test Results – Figure No. Q-1

Results of Chemical Tests provided by SGS Canada Inc.



LEGEND

- Borehole Location
- Blows/0.3m (Std. Pen Test, 475 J/blow)
- Monitoring Well
- Water Level in Monitoring Well (October 2019)

BH No	ELEVATION	NORTHINGS	EASTINGS
QWB	182.5	4 681 187.4	310 434.6
QEB	182.5	4 681 197.2	310 477.4

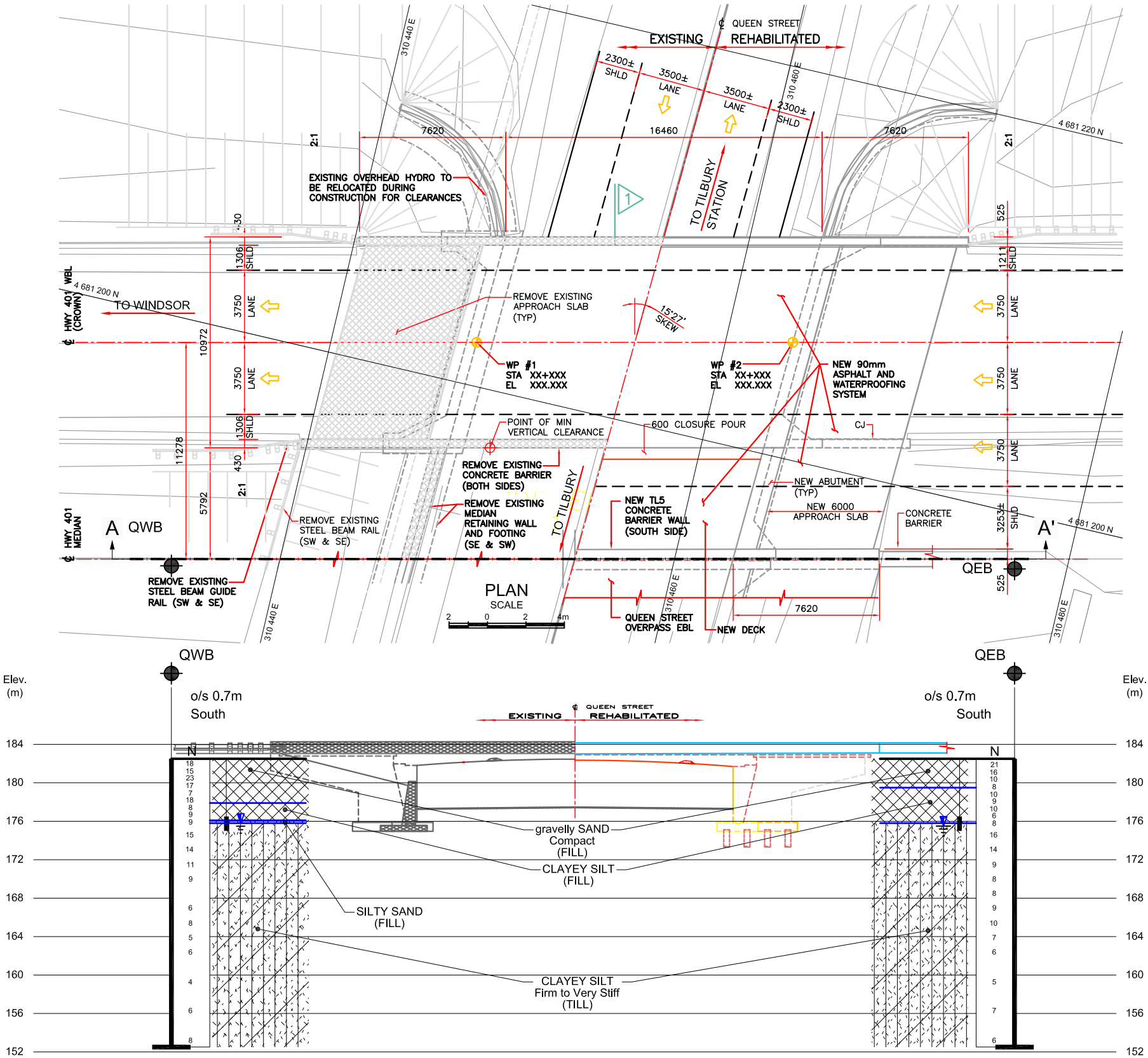
NOTE

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. 40J8-75

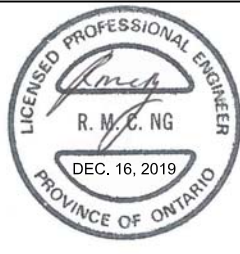
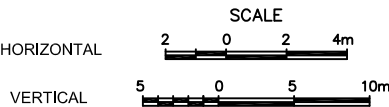
HWY No	401	DIST	WEST REGION
SUBM'D	NL	CHECKED	KA
DATE	DEC. 16, 2019	SITE	6-51/1
DRAWN	NL	CHECKED	NR
APPROVED	RN	DWG	QS-1



NOTES:

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

PROFILE ALONG A-A'



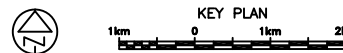
Reference WSP Ltd. Drawing: S18M-02111-06-302-001GA-WBL.dwg, dated October 2019.

Work Item No. 6
GWP No 3034-19-00

HIGHWAY 401 WIDENING
QUEEN STREET OVERPASS - EASTBOUND
BOREHOLE LOCATION PLAN AND SOIL STRATA



SHEET



LEGEND

- Borehole Location
- Blows/0.3m (Std. Pen Test, 475 J/blow)
- Monitoring Well
- Water Level in Monitoring Well (October 2019)

BH No	ELEVATION	NORTHINGS	EASTINGS
QWB	182.5	4 681 187.4	310 434.6
QEB	182.5	4 681 197.2	310 477.4

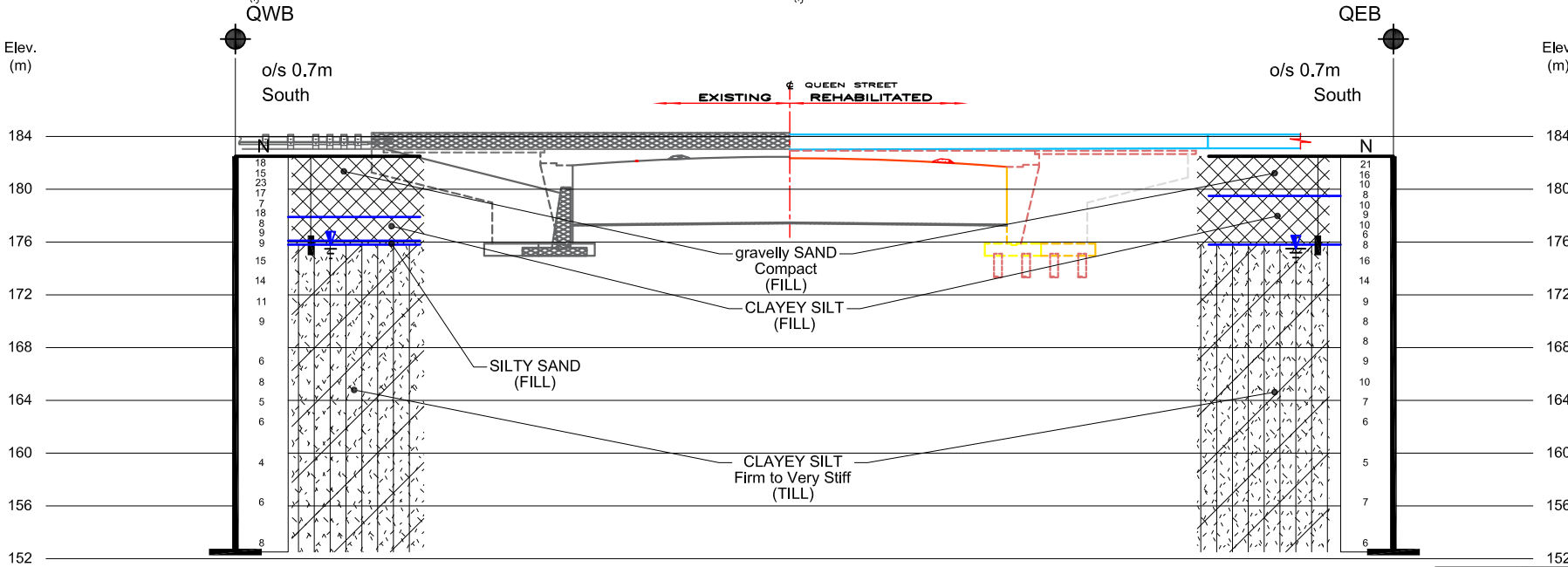
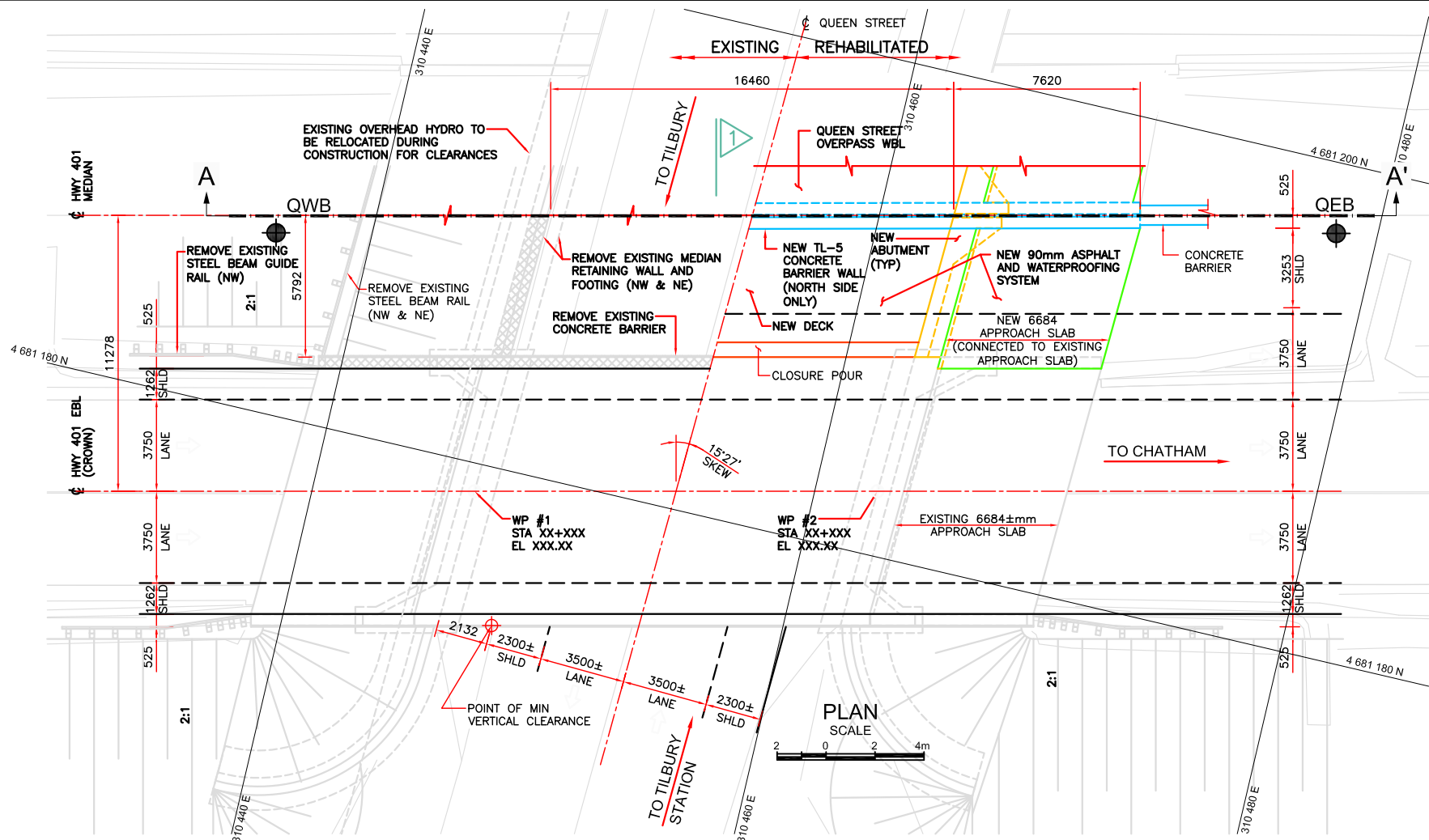
NOTE

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

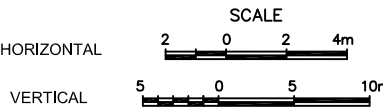
DATE	BY	DESCRIPTION

Geocres No. 40J8-75

HWY No	401	DIST	WEST REGION
SUBM'D	NL	CHECKED	KA
DATE	DEC. 16, 2019	SITE	6-51/2
DRAWN	NL	CHECKED	NR
APPROVED	RN	DWG	QS-2



PROFILE ALONG A-A'



NOTES:

- THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF REPORT AND RECORD OF BOREHOLE LOGS.
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Reference WSP Ltd. Drawing: S18M-02111-06-302-001GA-EBL.dwg, dated October 2019.

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'_{vo}	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 QEB

1 OF 3

METRIC

G.W.P. 3034-19-00 LOCATION Coords: 4 681 197.2 N; 310 477.4 E ORIGINATED BY J.O.
DIST West Region HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.
DATUM Geodetic DATE 2019.10.01 - 2019.10.02 LATITUDE 42.270321 LONGITUDE -82.431176 CHECKED BY N.R.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS		ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES				SHEAR STRENGTH kPa		WATER CONTENT (%)		W _p	W	W _L	GR		SA	SI	CL	
182.5 0.0	Ground gravelly SAND Compact, Brown, Moist (FILL)		1	SS	21																
			2	SS	16																
			3	SS	10																
			4	SS	8																
			5	SS	10																
			6	SS	9																
			7	SS	10																
			8	SS	6																
			9	SS	8																
175.8 6.7	SILTY CLAY TO CLAYEY SILT, some sand, trace gravel Stiff to very stiff, Grey, Moist (TILL)			VANE																	
			10	SS	16																
			11	SS	14																
			12	SS	9																
			13	SS	8																
				VANE																	
		14	SS	8																	
			VANE																		
167.5																					

Continued Next Page

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

RECORD OF BOREHOLE No QEB

2 OF 3

METRIC

G.W.P. 3034-19-00 LOCATION Coords: 4 681 197.2 N; 310 477.4 E ORIGINATED BY J.O.
DIST West Region HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.
DATUM Geodetic DATE 2019.10.01 - 2019.10.02 LATITUDE 42.270321 LONGITUDE -82.431176 CHECKED BY N.R.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _p	W	W _L		GR	SA	SI	CL
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	WATER CONTENT (%)								
167.5 15.0	(Cont'd) SILTY CLAY TO CLAYEY SILT, some sand, trace gravel Stiff, Grey, Moist (TILL)							20 40 60 80 100									
			15	SS	9		167										
							166										
			16	SS	10		165										
							164										
			17	SS	7		163										
				VANE			162										
			18	SS	6		161										
				VANE			160										
							159										
			19	SS	5		158										
				VANE			157										
							156										
			20	SS	7		155										
				VANE			154										
							153										
152.5			21	SS	6												
				VANE													

Continued Next Page







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

RECORD OF BOREHOLE No QEB

3 OF 3

METRIC

G.W.P. 3034-19-00 LOCATION Coords: 4 681 197.2 N; 310 477.4 E ORIGINATED BY J.O.
DIST West Region HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.
DATUM Geodetic DATE 2019.10.01 - 2019.10.02 LATITUDE 42.270321 LONGITUDE -82.431176 CHECKED BY N.R.

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														
152.5 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. <u>Monitoring Well Readings:</u> <table border="1"> <tr> <th>Date</th> <th>Depth (m)</th> <th>Elev.</th> </tr> <tr> <td>Oct.11/'19</td> <td>DRY</td> <td>-</td> </tr> <tr> <td>Oct.24/'19</td> <td>7.0</td> <td>175.5</td> </tr> <tr> <td>Oct.28/'19</td> <td>7.0</td> <td>175.5</td> </tr> </table> <u>Monitoring Well Legend:</u>  Stick-up Monument  Bentonite  Filter Sand  19 mm PVC Screen  Filter Bottom	Date	Depth (m)	Elev.	Oct.11/'19	DRY	-	Oct.24/'19	7.0	175.5	Oct.28/'19	7.0	175.5														
Date	Depth (m)	Elev.																									
Oct.11/'19	DRY	-																									
Oct.24/'19	7.0	175.5																									
Oct.28/'19	7.0	175.5																									

METRIC

Continued Next Page

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

ONTARIO MTO 19KF029A - WO6.GPJ ONTARIO MTO.GDT 20-11-19

RECORD OF BOREHOLE No QWB

2 OF 3

METRIC

G.W.P. 3034-19-00 LOCATION Coords: 4 681 187.4 N; 310 434.6 E ORIGINATED BY J.O.
DIST West Region HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.
DATUM Geodetic DATE 2019.10.03 - 2019.10.04 LATITUDE 42.270233 LONGITUDE -82.431695 CHECKED BY N.R.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p W W _L			GR	SA	SI	CL	
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					WATER CONTENT (%)							
167.5 15.0	(Cont'd) CLAYEY SILT TO SILTY CLAY, some sand, trace gravel Stiff, Grey, Moist (TILL)		15	SS	6		167						10				2	17	39	42
				VANE			166					>>								
			16	SS	8		165						○							
				VANE			164						10				2	17	37	44
			17	SS	5		163													
				VANE			162													
			18	SS	6		161						○							
				VANE			160													
			19	SS	4		159						○							
				VANE			158													
			20	SS	6		157						○							
				VANE			156													
			21	SS	8		155													
				VANE			154													
152.5							153						10				0	19	38	43
				VANE																

Continued Next Page







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

RECORD OF BOREHOLE No QWB

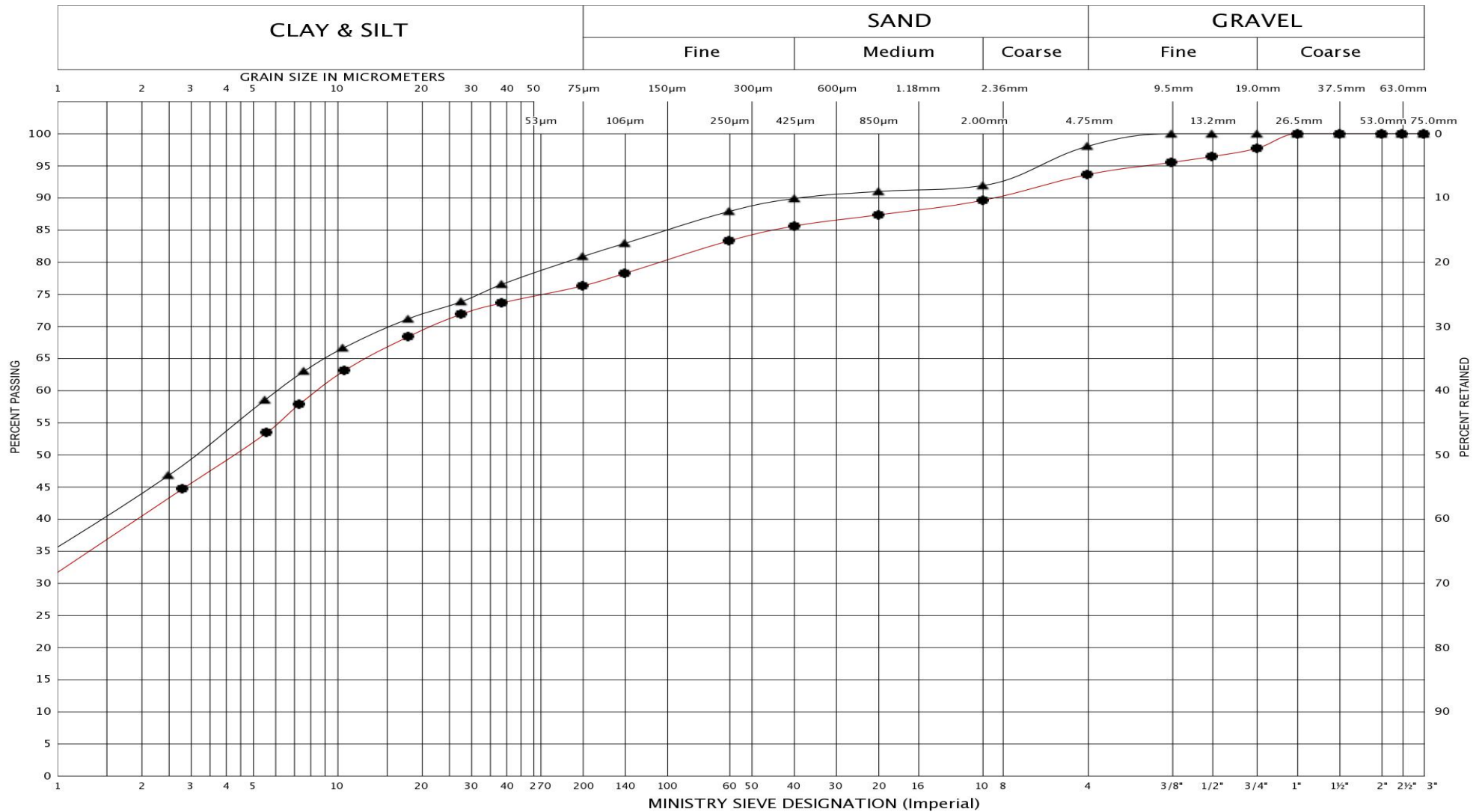
3 OF 3

METRIC

G.W.P. 3034-19-00 LOCATION Coords: 4 681 187.4 N; 310 434.6 E ORIGINATED BY J.O.
 DIST West Region HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY K.A.
 DATUM Geodetic DATE 2019.10.03 - 2019.10.04 LATITUDE 42.270233 LONGITUDE -82.431695 CHECKED BY N.R.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)											
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W _p	W	W _L													
152.5 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. <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.11/'19</td> <td>DRY</td> <td>-</td> </tr> <tr> <td>Oct.24/'19</td> <td>4.8</td> <td>177.7</td> </tr> <tr> <td>Oct.28/'19</td> <td>5.6</td> <td>176.9</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.11/'19	DRY	-	Oct.24/'19	4.8	177.7	Oct.28/'19	5.6	176.9															
Date	Depth (m)	Elev.																										
Oct.11/'19	DRY	-																										
Oct.24/'19	4.8	177.7																										
Oct.28/'19	5.6	176.9																										

UNIFIED SOIL CLASSIFICATION SYSTEM



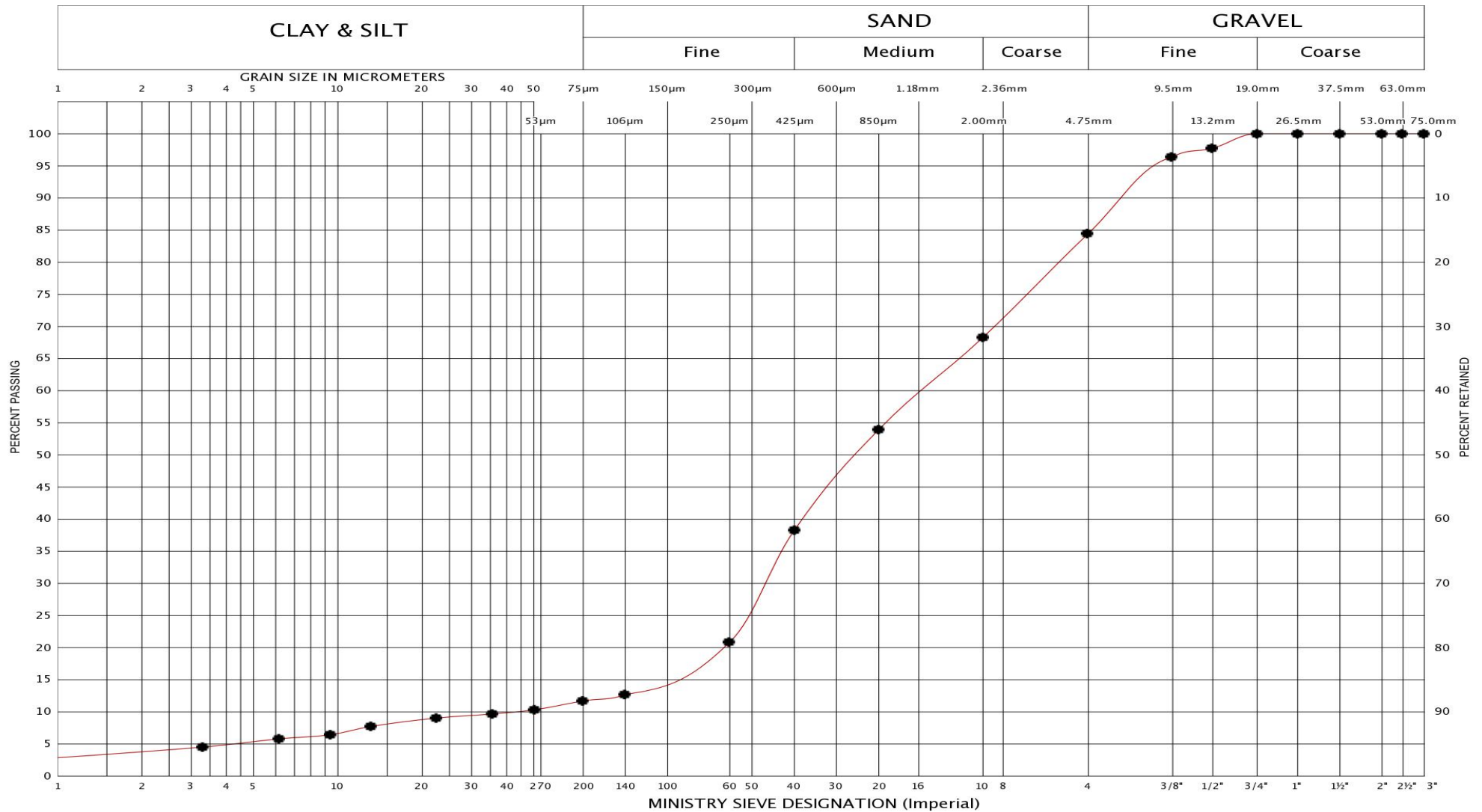
LEGEND	BH	QEB	QWB
	SAMPLE	6	8
	SYMBOL	●	▲



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

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

UNIFIED SOIL CLASSIFICATION SYSTEM



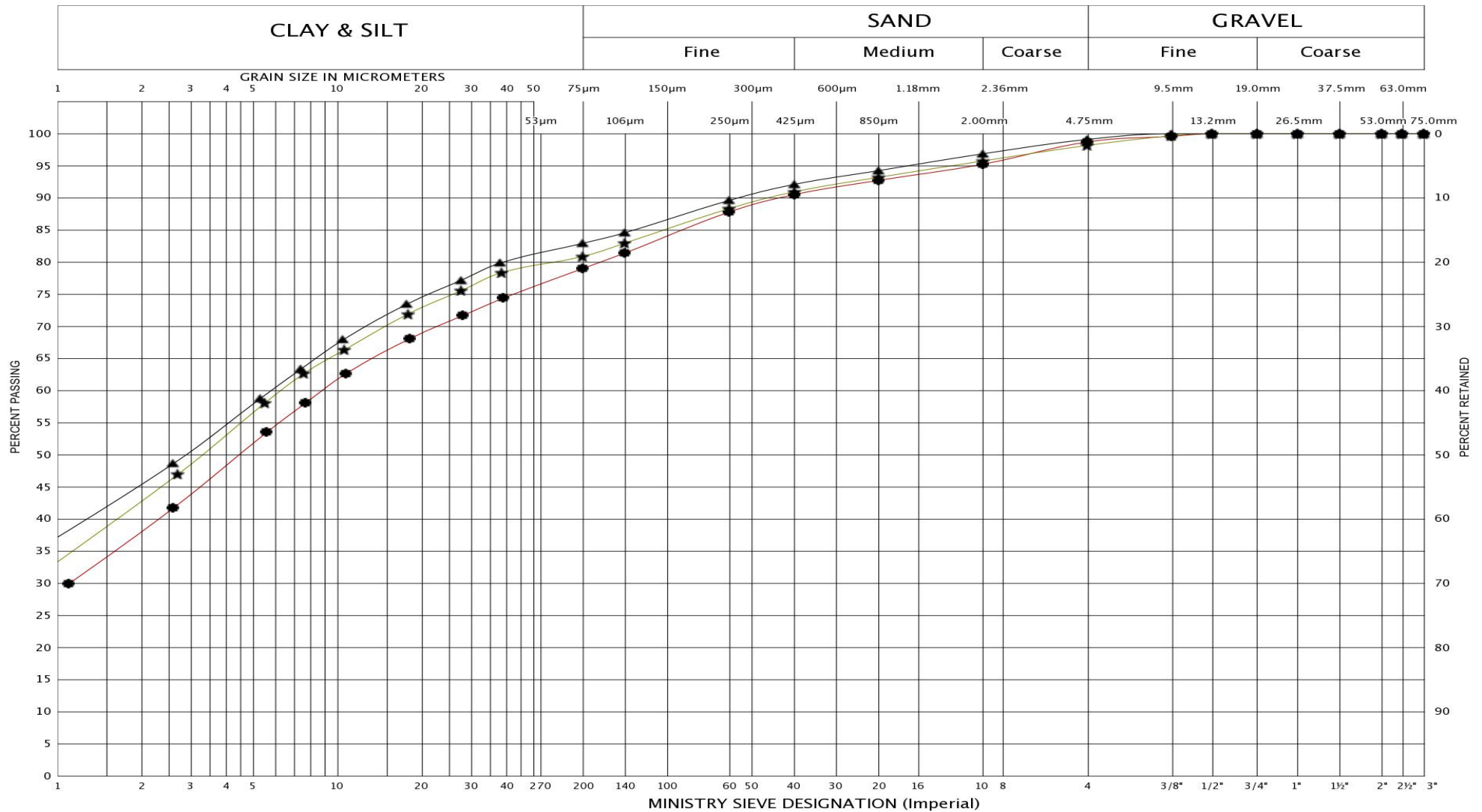
LEGEND	BH	QWB
	SAMPLE	5
	SYMBOL	•



GRAIN SIZE DISTRIBUTION
Gravelly Sand, Trace Silt and Trace Clay (Fill)

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

UNIFIED SOIL CLASSIFICATION SYSTEM



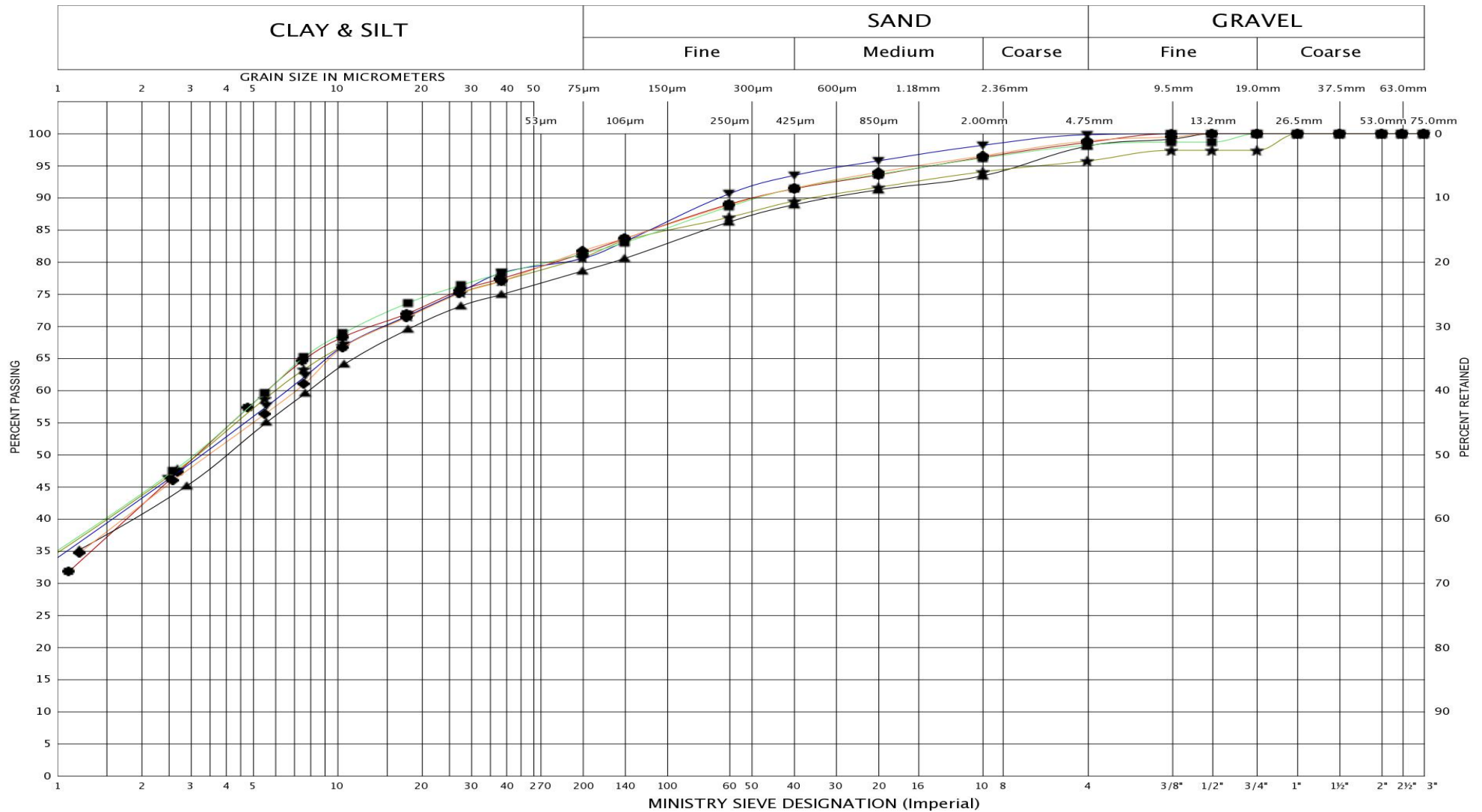
LEGEND	BH	QEB	QWB	QWB
SAMPLE	10	11	15	
SYMBOL	●	▲	★	



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

FIG No.: GS-QS-3
 HWY : 401
 GWP 3034-19-00

UNIFIED SOIL CLASSIFICATION SYSTEM

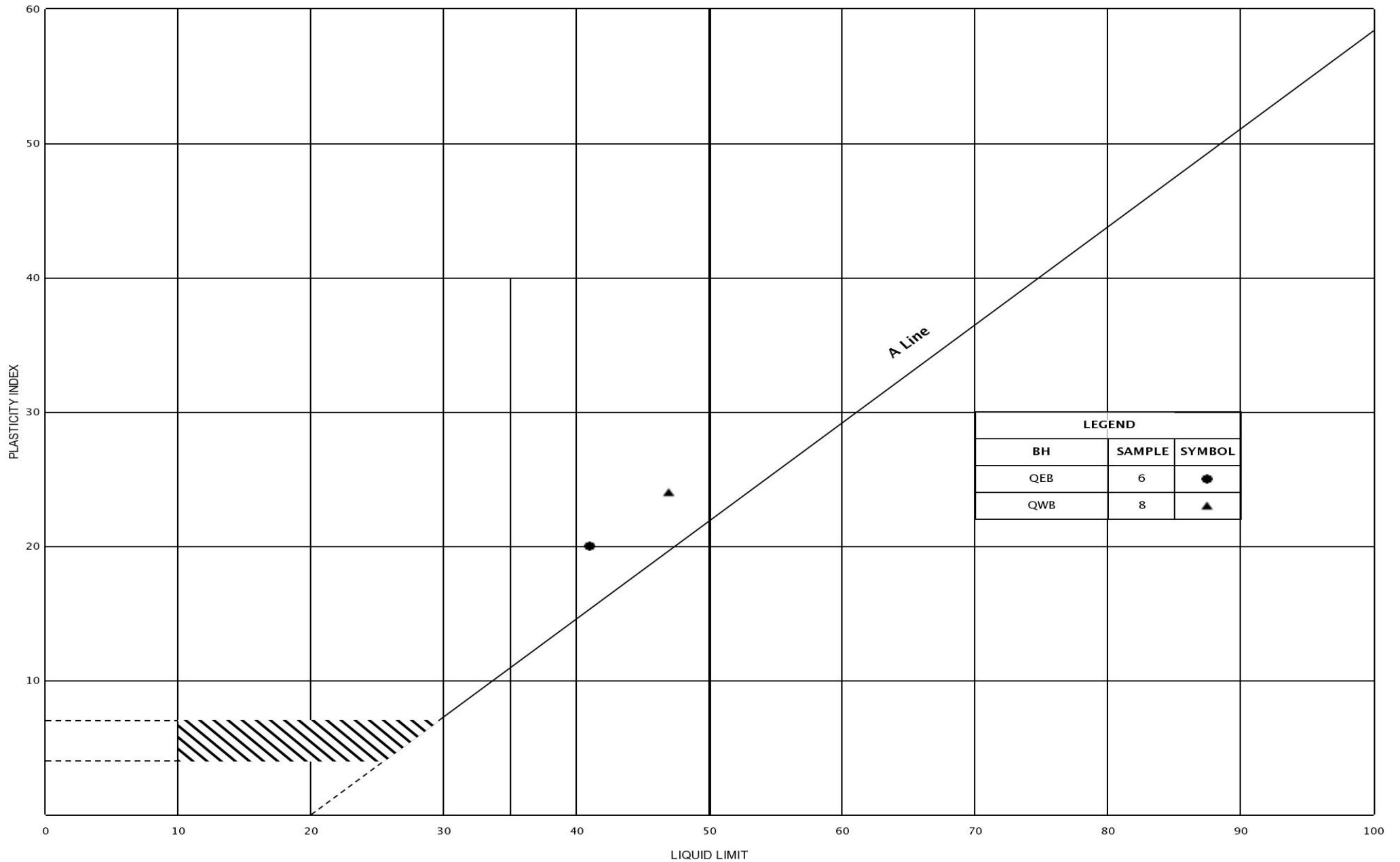


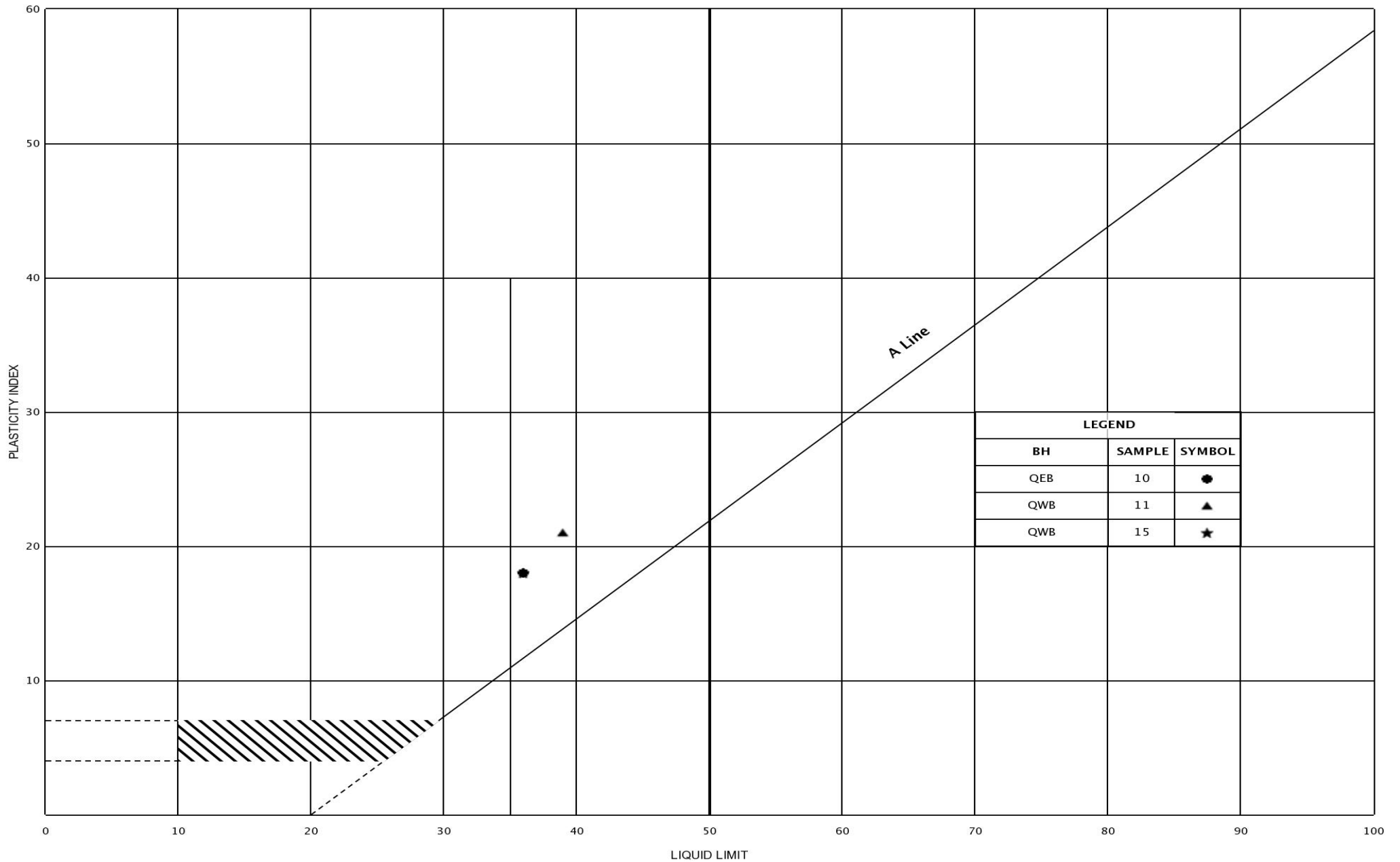
LEGEND	BH	QEB	QEB	QEB	QWB	QWB	QWB
	SAMPLE	14	18	20	21	17	14
	SYMBOL	●	▲	★	▼	■	◆



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

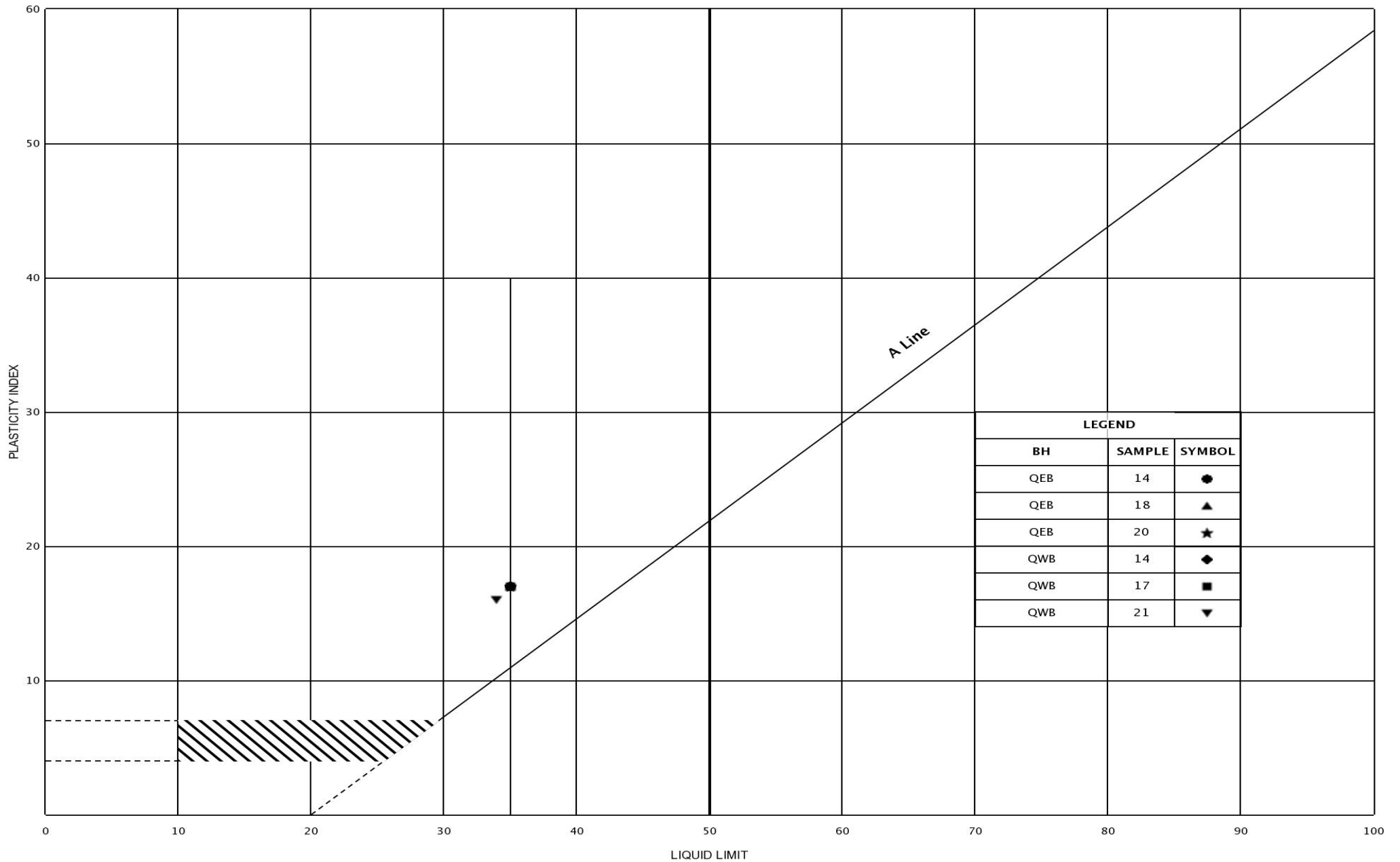
FIG No.: GS-QS-4
HWY : 401
GWP 3034-19-00





PLASTICITY CHART
 SILTY CLAY, Some Sand, Trace Gravel (Till)

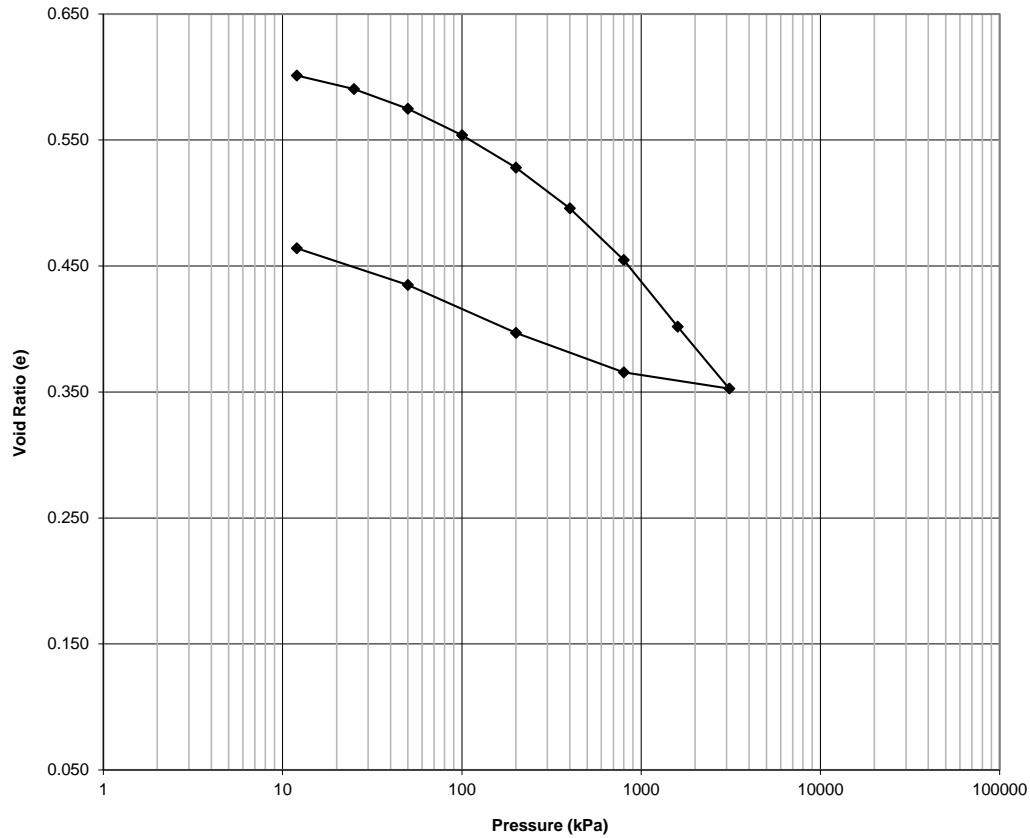
FIG No.: PC- QS-2
 HWY.: 401
 GWP 3034-19-00



Consolidation Test Results
(ASTM D2435)
Highway 401, CA 3017-E-0006, Task 006-4 Bridges Detail Design

Borehole QWB, Sample TW 14, Depth 13.7-14.3 m

Void Ratio versus Log of Pressure



SOIL TYPE: Grey Clayey Silt			
e_0	= 0.637	W_L	= 35
W_0	= 22.8 %	W_P	= 18
γ	= 19.9 kN/m ³	PI	= 17
FIGURE No: Q-1			
Highway 401, CA 3017-E-0006, Task 006-4 Bridges Detail Design			
PML Ref: 19KF029A			



FINAL REPORT

CA14407-OCT19 R1

19KF029A Hwy 401, Tilbury (M/Q)

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 19KF029A Hwy 401, Tilbury (M/Q)

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 CA14407-OCT19

Received 10/11/2019

Approved 10/17/2019

Report Number CA14407-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:007603

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

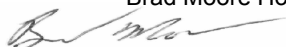




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FINAL REPORT

CA14407-OCT19 R1

Client: Peto MacCallum Ltd

Project: 19KF029A Hwy 401, Tilbury (M/Q)

Project Manager: Nazibur Rahman

Samplers: Jinsuko

PACKAGE: - Corrosivity Index (SOIL)

Sample Number	5	6	7	8	9	10	11	12
Sample Name	MEB, SS3 (5'-7')	MEB, SS5 (10'-12')	MWB, SS4 (7.5'-9.5')	MWB, SS6 (12.5'-14.5')	QWB, SS4 (7.5'-9.5')	QWB, SS6 (12.5'-14.5')	QEB, SS5 (10'-12')	QEB, SS7 (15'-17')
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
Corrosivity Index											
Corrosivity Index	none	1		2	1	1	4.5	7.5	4	6.5	9.5
Soil Redox Potential	mV	-		228	222	280	383	452	416	290	275
Sulphide	%	0.02		< 0.02	< 0.02	< 0.02	0.30	0.03	< 0.02	0.10	0.08
pH	pH Units	0.05		7.79	7.98	8.01	8.08	8.62	8.61	8.33	7.75
Resistivity (calculated)	ohms.cm	-9999		2520	3100	3340	3160	8870	5690	2160	2060

PACKAGE: - General Chemistry (SOIL)

Sample Number	5	6	7	8	9	10	11	12
Sample Name	MEB, SS3 (5'-7')	MEB, SS5 (10'-12')	MWB, SS4 (7.5'-9.5')	MWB, SS6 (12.5'-14.5')	QWB, SS4 (7.5'-9.5')	QWB, SS6 (12.5'-14.5')	QEB, SS5 (10'-12')	QEB, SS7 (15'-17')
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	
General Chemistry											
Conductivity	uS/cm	2		397	323	299	316	113	176	464	485

PACKAGE: - Metals and Inorganics (SOIL)

Sample Number	5	6	7	8	9	10	11	12
Sample Name	MEB, SS3 (5'-7')	MEB, SS5 (10'-12')	MWB, SS4 (7.5'-9.5')	MWB, SS6 (12.5'-14.5')	QWB, SS4 (7.5'-9.5')	QWB, SS6 (12.5'-14.5')	QEB, SS5 (10'-12')	QEB, SS7 (15'-17')
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	
Metals and Inorganics											
Moisture Content	%	0.1		17.5	14.2	14.1	13.8	7.0	8.2	16.6	17.4



FINAL REPORT

CA14407-OCT19 R1

Client: Peto MacCallum Ltd

Project: 19KF029A Hwy 401, Tilbury (M/Q)

Project Manager: Nazibur Rahman

Samplers: Jinsuko

PACKAGE: - Metals and Inorganics (SOIL)

Sample Number	5	6	7	8	9	10	11	12
Sample Name	MEB, SS3 (5'-7')	MEB, SS5 (10'-12')	MWB, SS4 (7.5'-9.5')	MWB, SS6 (12.5'-14.5')	QWB, SS4 (7.5'-9.5')	QWB, SS6 (12.5'-14.5')	QEB, SS5 (10'-12')	QEB, SS7 (15'-17')
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
-----------	-------	----	--	--------	--------	--------	--------	--------	--------	--------

Metals and Inorganics (continued)

Sulphate	µg/g	0.4		130	70	52	340	3.2	6.9	290	270
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PACKAGE: - Other (ORP) (SOIL)

Sample Number	5	6	7	8	9	10	11	12
Sample Name	MEB, SS3 (5'-7')	MEB, SS5 (10'-12')	MWB, SS4 (7.5'-9.5')	MWB, SS6 (12.5'-14.5')	QWB, SS4 (7.5'-9.5')	QWB, SS6 (12.5'-14.5')	QEB, SS5 (10'-12')	QEB, SS7 (15'-17')
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
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Other (ORP)

Chloride	µg/g	0.4		210	130	140	54	11	26	130	100
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FINAL REPORT

CA14407-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			



FINAL REPORT

CA14407-OCT19 R1

QC SUMMARY

Conductivity

Method: SM 2510 | Internal ref.: ME-CA-1ENVIEWL-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		
Conductivity	EWL0295-OCT19	uS/cm	2	0.002	0	10	98	90	110	NA		

pH

Method: SM 4500 | Internal ref.: ME-CA-1ENVIEWL-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		
pH	EWL0295-OCT19	pH Units	0.05	NA	0		100			NA		

QC SUMMARY

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 --



APPENDIX B

Previous Borehole Logs and Drawings (GEOCRES No. 40J08-002)

59-F-2
W.P.# 161-58
Hwy. # 401 E
QUEEN ST.
CROSSING AT
TILBURY

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW
OFFICE REPORT ON SOIL EXPLORATIONDRILL RIG 54-6 OPERATION BORE & PENETRATION JOB F-59-2 WP 161-59 BORING 1 STA. 358+65.45' RT.
CASING B X (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT FEB. 1959
SAMPLER HAMMER WT. 250 LBS. DROP 19 INCHES COMPILED BY H.S. CHECKED BY V.K. DATE BORING 7 JAN. 1959

ABBREVIATIONS

V - INSITU VANE SHEAR TEST
M - MECHANICAL ANALYSIS
U - UNCONFINED COMPRESSION
Q_c - TRIAXIAL CONSOLIDATED QUICKQ - TRIAXIAL QUICK
S - TRIAXIAL SLOW
WL - WATER LEVEL IN CASING
WT - WATER TABLE IN SOILK - PERMIABILITY
C - CONSOLIDATION
CA - CASING
γ - UNIT WEIGHT

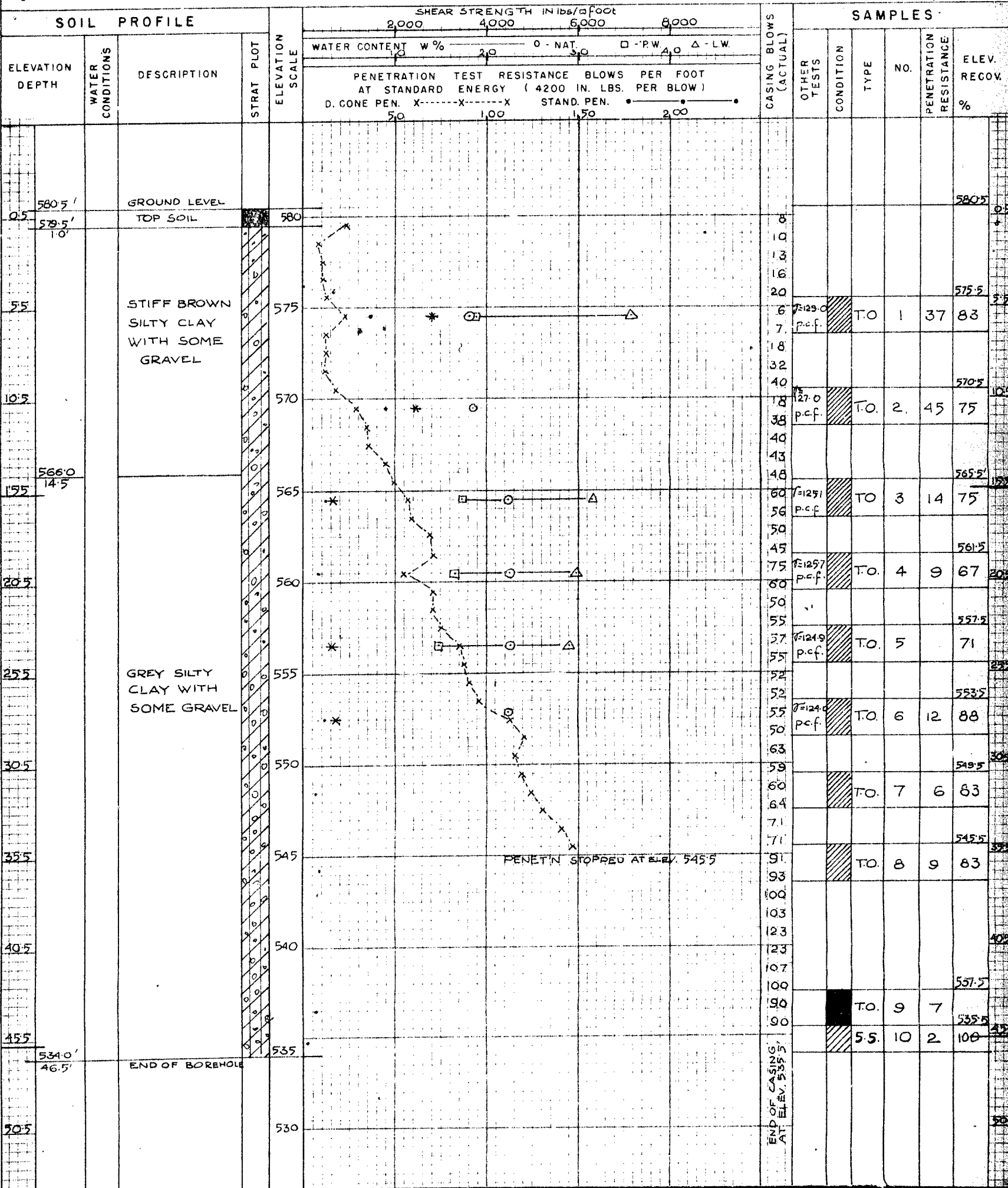
SAMPLE TYPES

C.S. - CHUNK
D.O. - DRIVE OPEN
D.F. - DRIVE FOOT VALVE
T.O. - THIN WALLED OPEN
SS - SLEEVE SAMPLE
PS - PISTON SAMPLE
WS - WASHED SAMPLE
RC - ROCK CORE

SAMPLE CONDITION

- DISTURBED
- FAIR
- GOOD
- LOST

SOIL PROFILE



DEPARTMENT OF HIGHWAYS - ONTARIO
 MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW
OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG 54-6 OPERATION BORE & PENETRATION JOB F-59-2 WP 161-58 BORING 2 STA 359+00 45' LT
 CASING B X (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT FEB 1959
 SAMPLER HAMMER WT. 250 LBS. DROP 19 INCHES COMPILED BY H.S. CHECKED BY V.K. DATE BORING 9 JAN 1959

ABBREVIATIONS

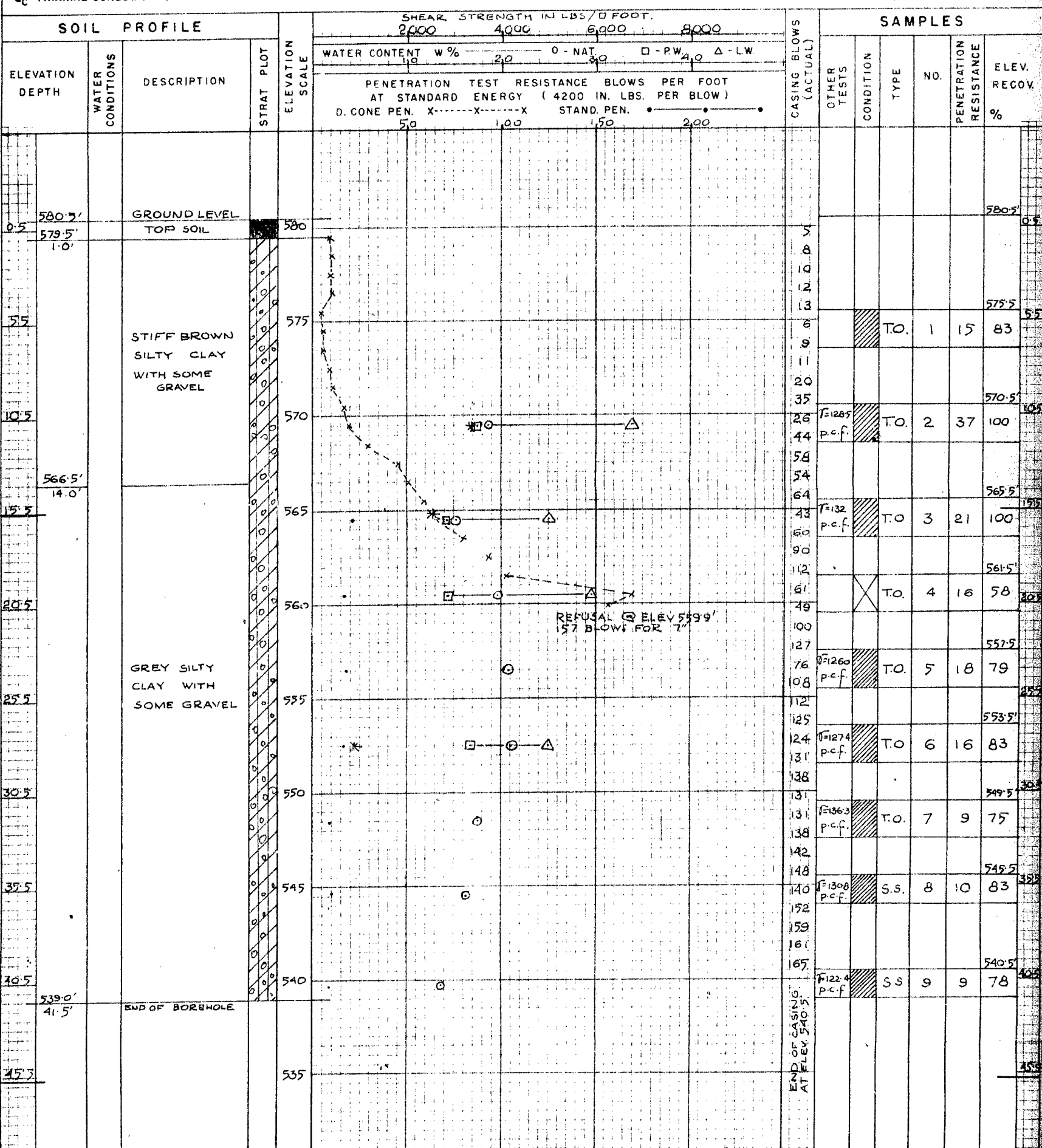
V - INSITU VANE SHEAR TEST Q - TRIAXIAL QUICK K - PERMIABILITY
 M - MECHANICAL ANALYSIS S - TRIAXIAL SLOW C - CONSOLIDATION
 U - UNCONFINED COMPRESSION WL - WATER LEVEL IN CASING CA - CASING
 QC - TRIAXIAL CONSOLIDATED QUICK WT - WATER TABLE IN SOIL γ - UNIT WEIGHT

SAMPLE TYPES

C.S. - CHUNK SS - SLEEVE SAMPLE
 D.O. - DRIVE OPEN PS - PISTON SAMPLE
 D.F. - DRIVE FOOT VALVE WS - WASHED SAMPLE
 T.O. - THIN WALLED OPEN R.C. - ROCK CORE

SAMPLE CONDITION

 - DISTURBED
 - FAIR
 - GOOD
 - LOST



DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW
OFFICE REPORT ON SOIL EXPLORATIONDRILL RIG 54-G OPERATION BORE & PENETRATION JOB F-59-2 WP 161-50 BORING 3 STA. 00+55.47 LT
CASING B-X (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT FEB 1955
SAMPLER HAMMER WT. 250 LBS. DROP 19 INCHES COMPILED BY H.S. CHECKED BY V.K. DATE BORING 13 JAN. 1959

ABBREVIATIONS

V - INSITU VANE SHEAR TEST
M - MECHANICAL ANALYSIS
U - UNCONFINED COMPRESSION
Q_c TRIAXIAL CONSOLIDATED QUICKQ - TRIAXIAL QUICK
S - TRIAXIAL SLOW
WL - WATER LEVEL IN CASING
WT - WATER TABLE IN SOIL
K - PERMIABILITY
C - CONSOLIDATION
CA - CASING
γ - UNIT WEIGHT

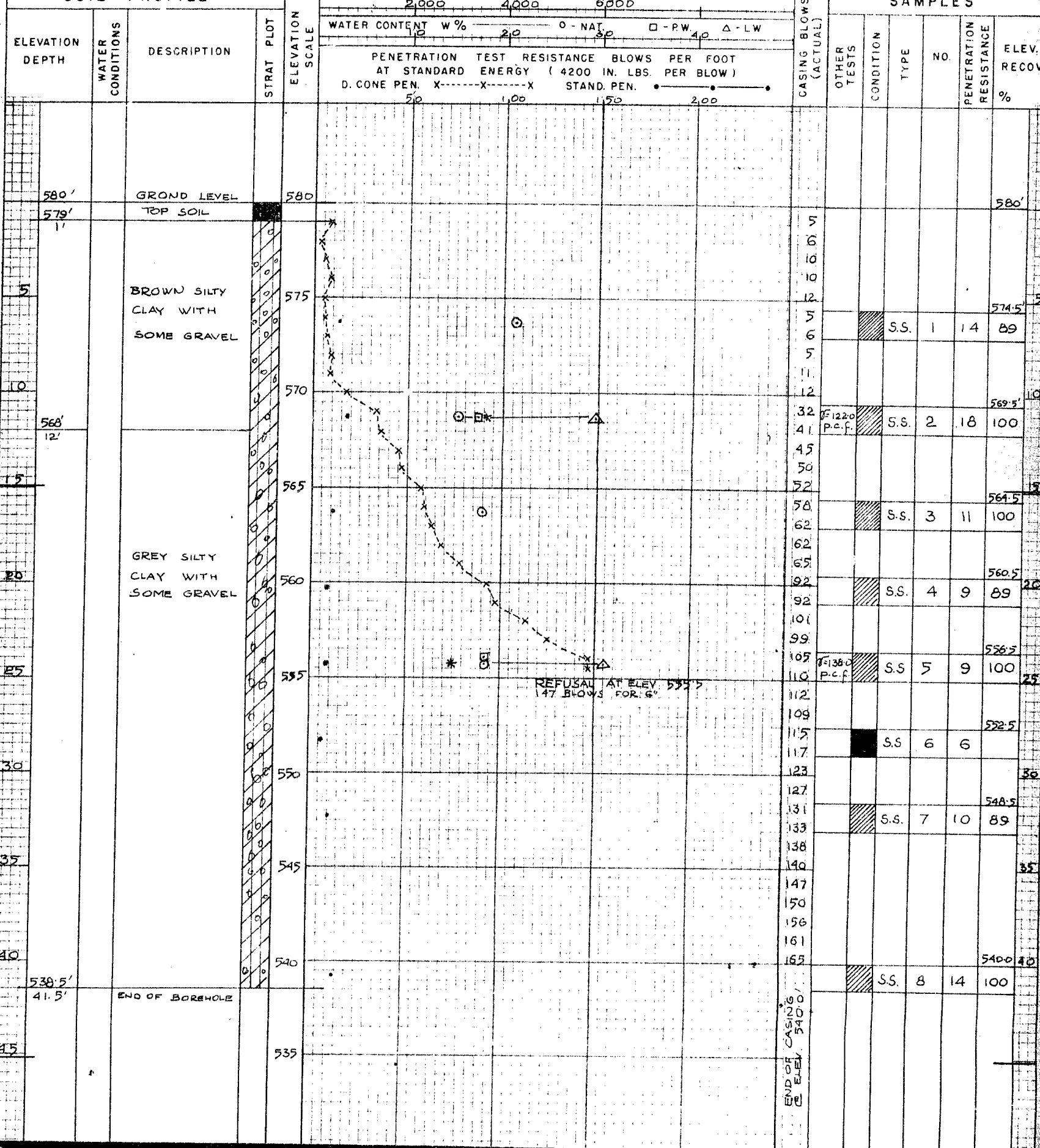
SAMPLE TYPES

C.S. - CHUNK
D.O. - DRIVE OPEN
D.F. - DRIVE FOOT VALVE
T.O. - THIN WALLED OPENS.S. - SLEEVE SAMPLE
P.S. - PISTON SAMPLE
W.S. - WASHED SAMPLE
R.C. - ROCK CORE

SAMPLE CONDITION

 - DISTURBED
- FAIR
- GOOD
- LOST

SOIL PROFILE



DEPARTMENT OF HIGHWAYS - ONTARIO
 MATERIALS & RESEARCH BRANCH - FOUNDATIONS SECTION - DOWNSVIEW
OFFICE REPORT ON SOIL EXPLORATION

DRILL RIG 54-6 OPERATION PENETRATION JOB F-59-2 W.P. 161-5B BORING 4 STA. 00+20 45' RT.
 CASING B X (standard samplers to fit unless noted) DATUM GEODETIC DATE REPORT FEB. 1959
 SAMPLER HAMMER WT. 250 LBS. DROP 19 INCHES COMPILED BY H.S. CHECKED BY V.K. DATE BORING 20 JAN. 1959

ABBREVIATIONS

V - INSITU VANE SHEAR TEST Q - TRIAXIAL QUICK K - PERMIABILITY
 M - MECHANICAL ANALYSIS S - TRIAXIAL SLOW C - CONSOLIDATION
 U - UNCONFINED COMPRESSION WL - WATER LEVEL IN CASING CA - CASING
 QC - TRIAXIAL CONSOLIDATED QUICK WT - WATER TABLE IN SOIL γ - UNIT WEIGHT

SAMPLE TYPES

C.S. - CHUNK S.S. - SLEEVE SAMPLE
 D.O. - DRIVE OPEN P.S. - PISTON SAMPLE
 D.F. - DRIVE FOOT VALVE W.S. - WASHED SAMPLE
 T.O. - THIN WALLED OPEN R.C. - ROCK CORE

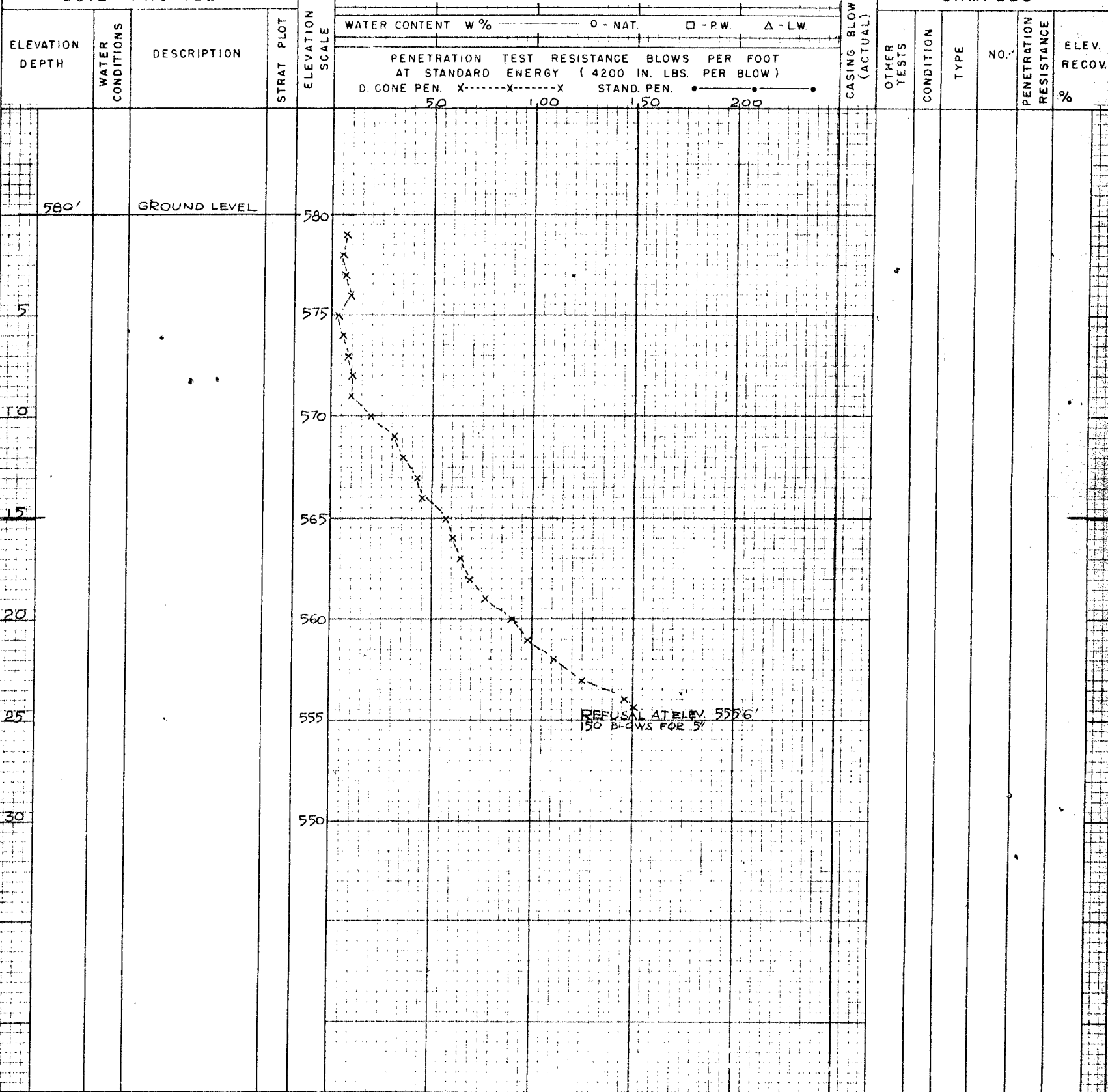
SAMPLE CONDITION

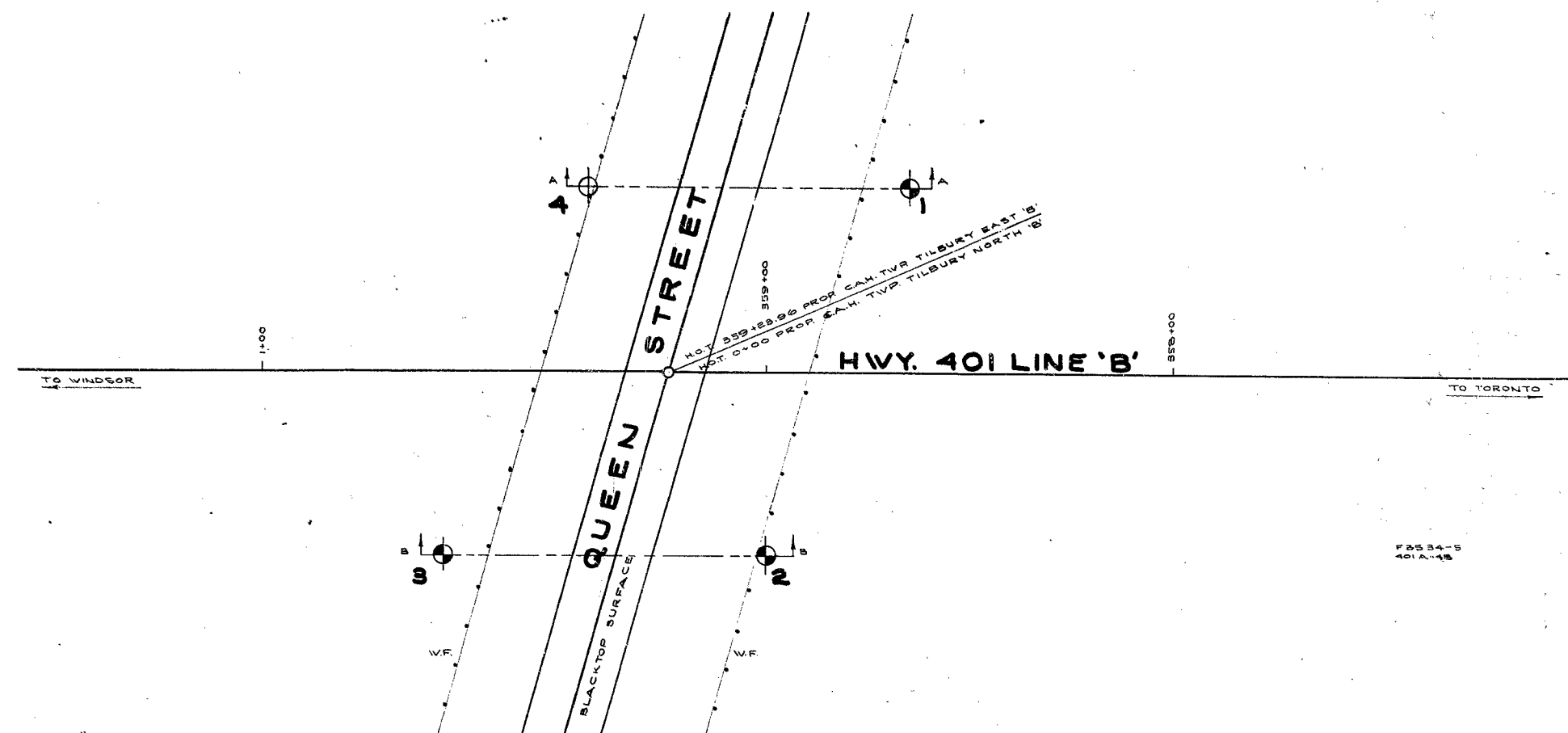


- DISTURBED
 - FAIR
 - GOOD
 - LOST

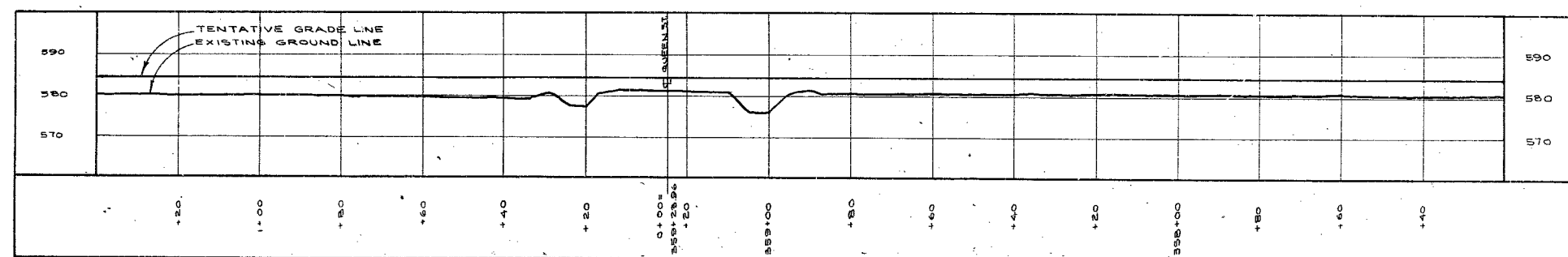
SOIL PROFILE

SAMPLES

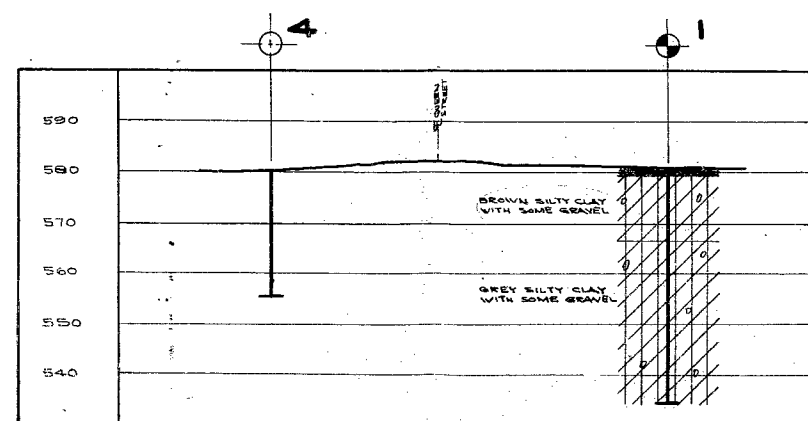




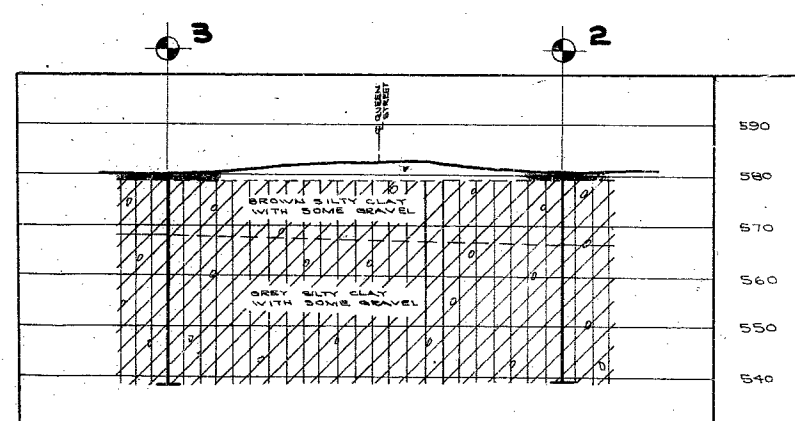
PLAN



PROFILE



A-A



B-B

LEGEND

- BORE HOLE
- PENETRATION HOLE
- BORE & PENETRATION HOLE

HOLE NO.	ELEVATION	STATION	DISTANCE FROM E.
1	580.5'	358+65	45' RT.
2	580.5'	359+00	45' LT.
3	580.0'	00+55	45' LT.
4	580.0'	00+20	45' RT.

NOTE

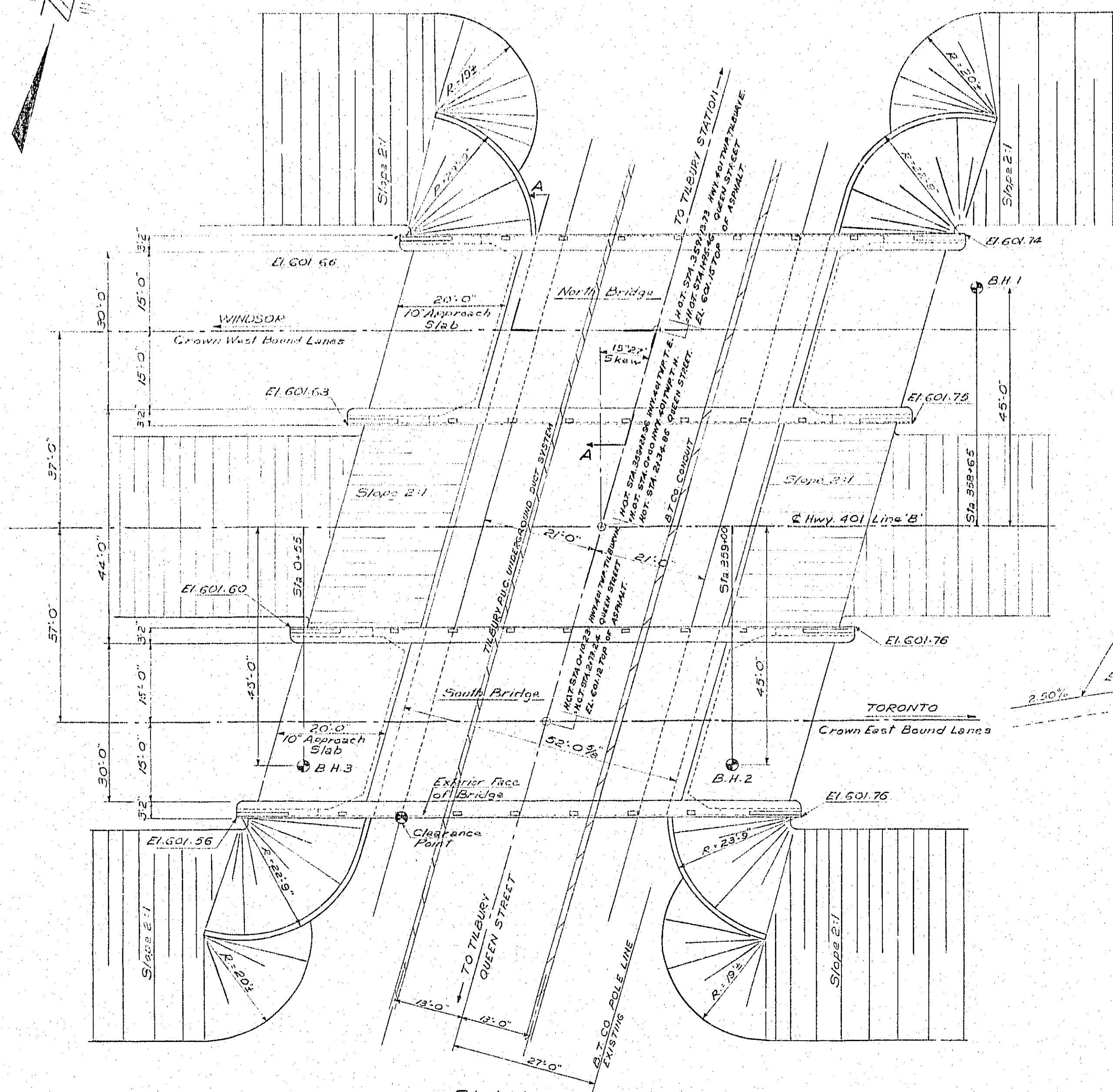
THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT BORE HOLE LOCATIONS. BETWEEN BORE HOLES THE BOUNDARIES ARE ASSUMED FROM GEOLOGICAL EVIDENCE AND MAY BE SUBJECT TO CONSIDERABLE ERROR.

DEPARTMENT OF HIGHWAYS-ONTARIO
MATERIALS & RESEARCH SECTION

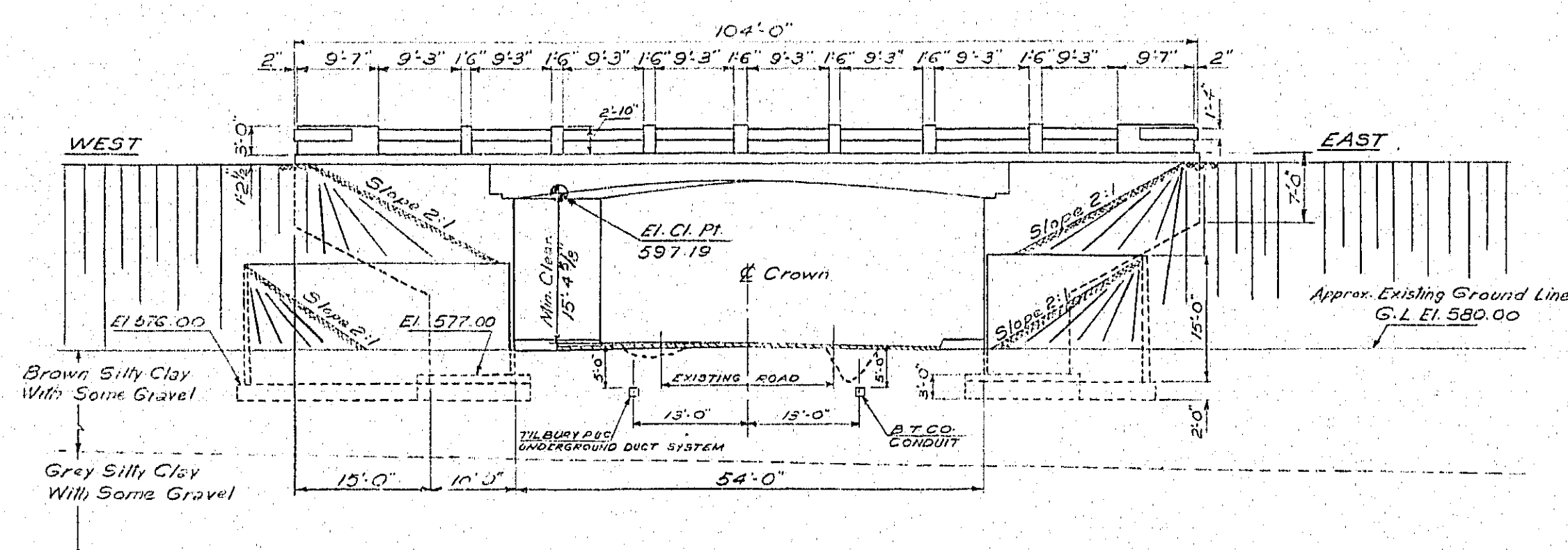
QUEEN STREET PROPOSED CROSSING

SHOWING POSITIONS & ELEVATIONS OF HOLES

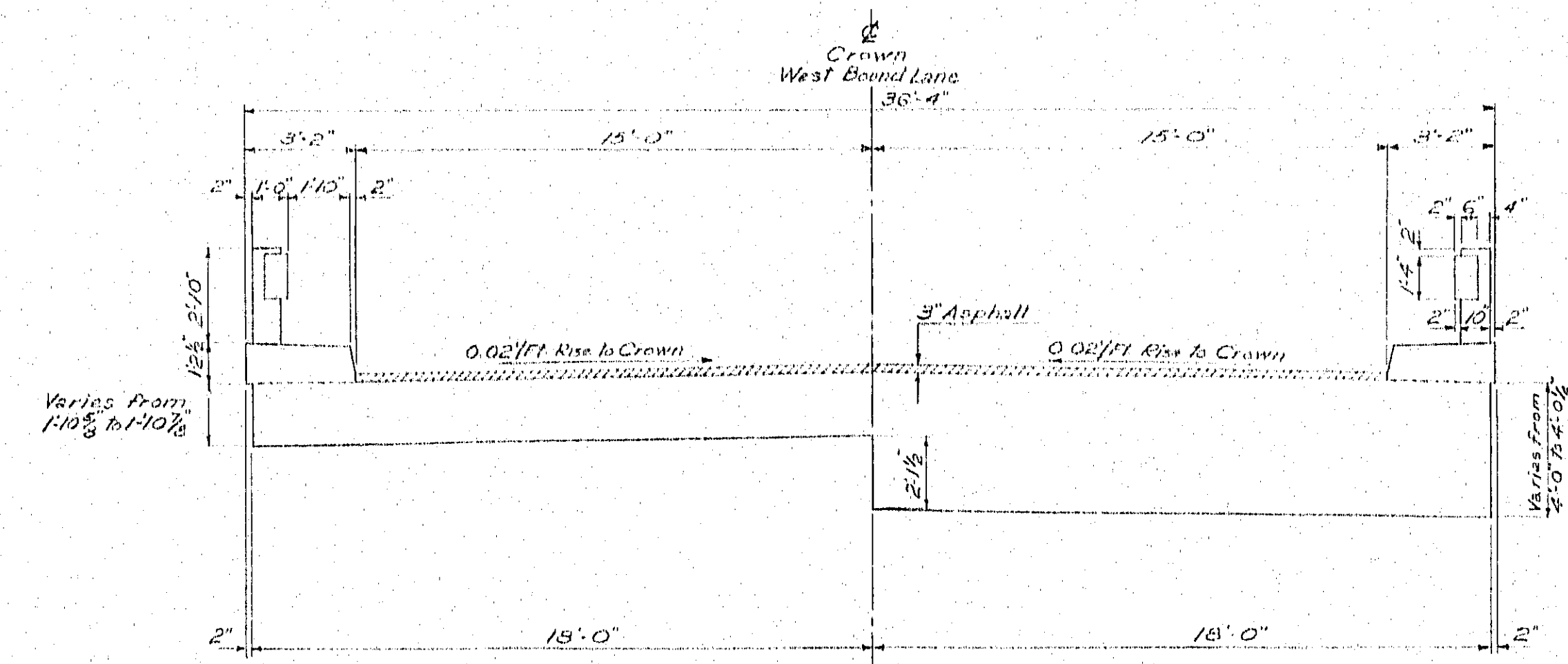
HWY 401 DISTRICT 1 COUNTY ESSEX & KENT
TOWNSHIP TILBURY EAST & NORTH LOT 22 CON. 4
LOCATION AT TILBURY
DRAWN BY: T. MELLORES CHECKED BY: W.P. 161-59
DATE: MARCH 12/59 APPROVED BY: DRAWING NO.
SCALE: 1"=20' F 59-2A



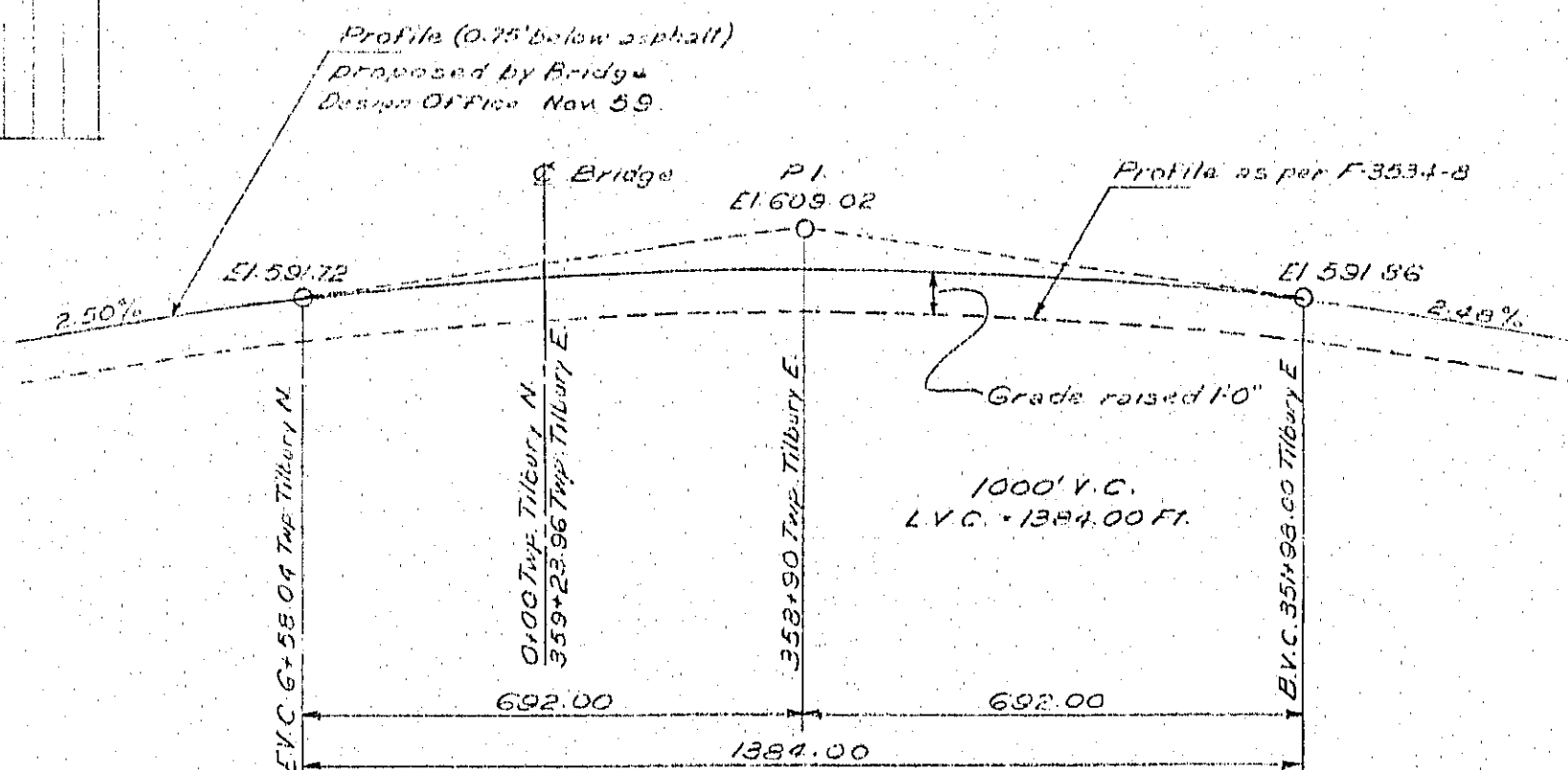
PLAN
Scale: 1/4" = 1'-0"



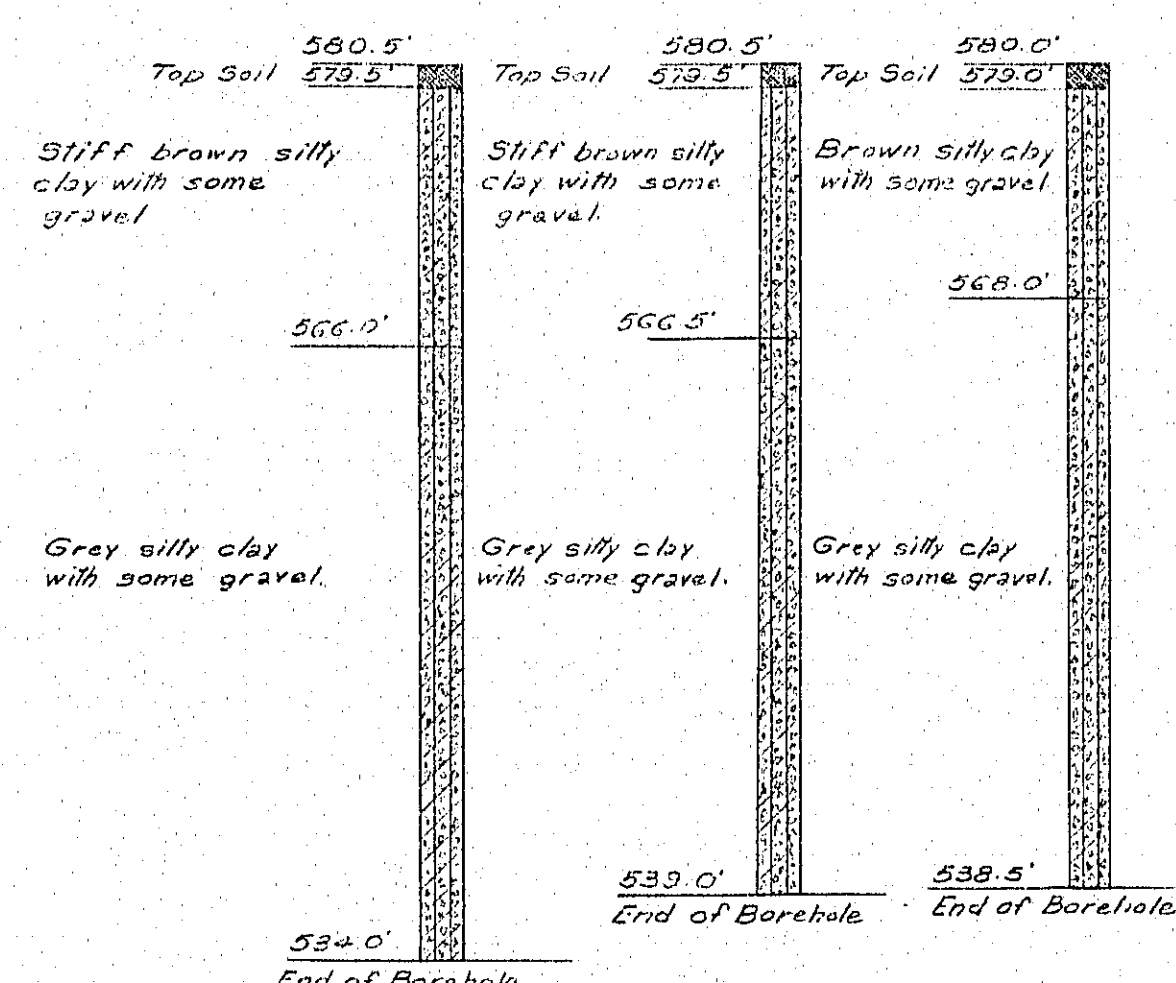
SOUTH ELEVATION
Scale: 1/8" = 1'-0"



SECTION A-A
Scale: 1/4" = 1'-0"



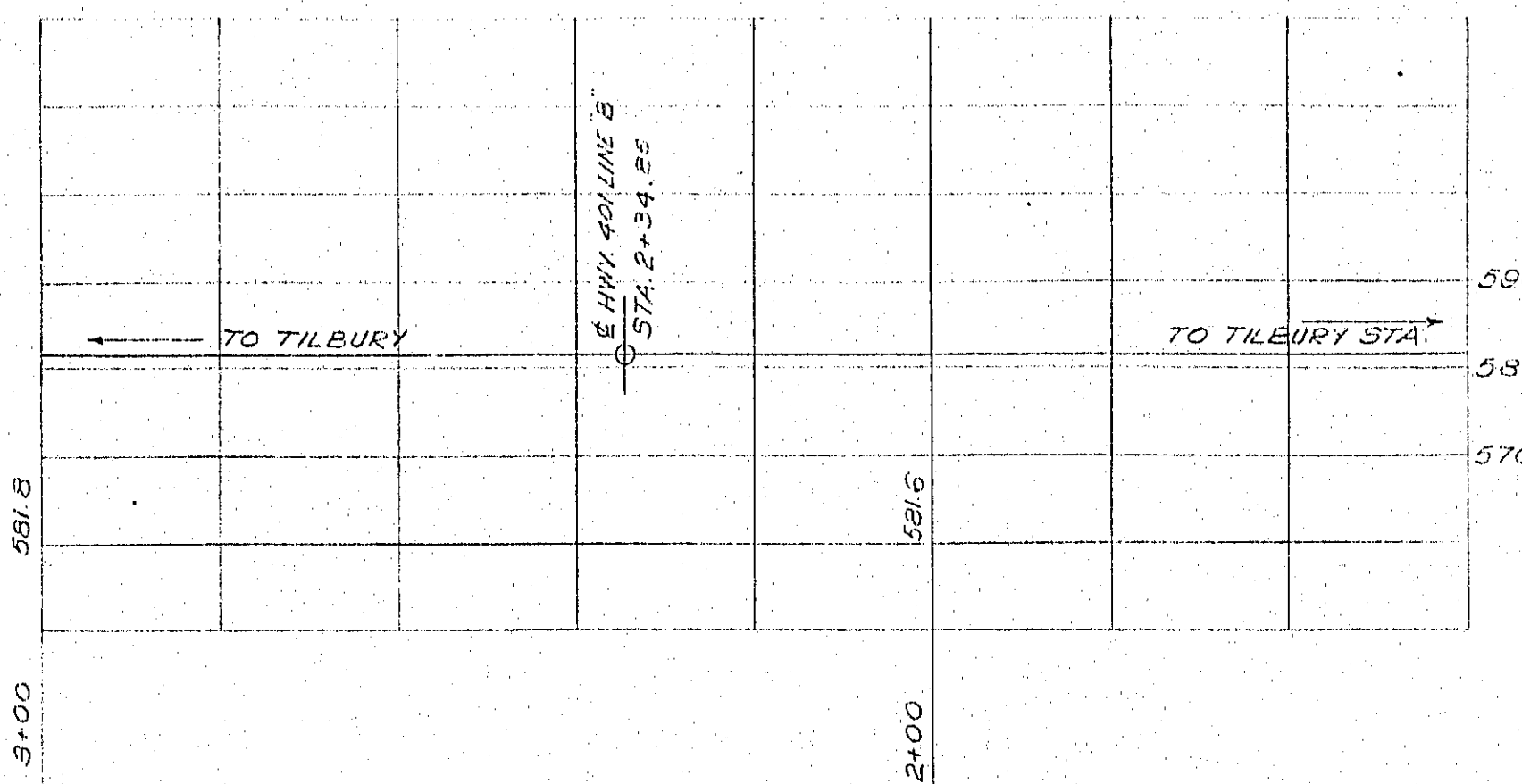
PROFILE HWY 401 LINE 'B'
0.75' Below Top of Asphalt at Crown



PROFILE OF QUEEN STREET
TOP OF ASPHALT AT CROWN

LIST OF DRAWINGS

- D-4420-1 General Plan
- 2 Footings, Abutments & Wing Walls
- 3 Reinforcing Details
- 4 Retaining Walls
- 5 Approach Slab - Hard Grade
- 6 Steel Table - Structural Grade
- 7 Steel Table - Structural Grade



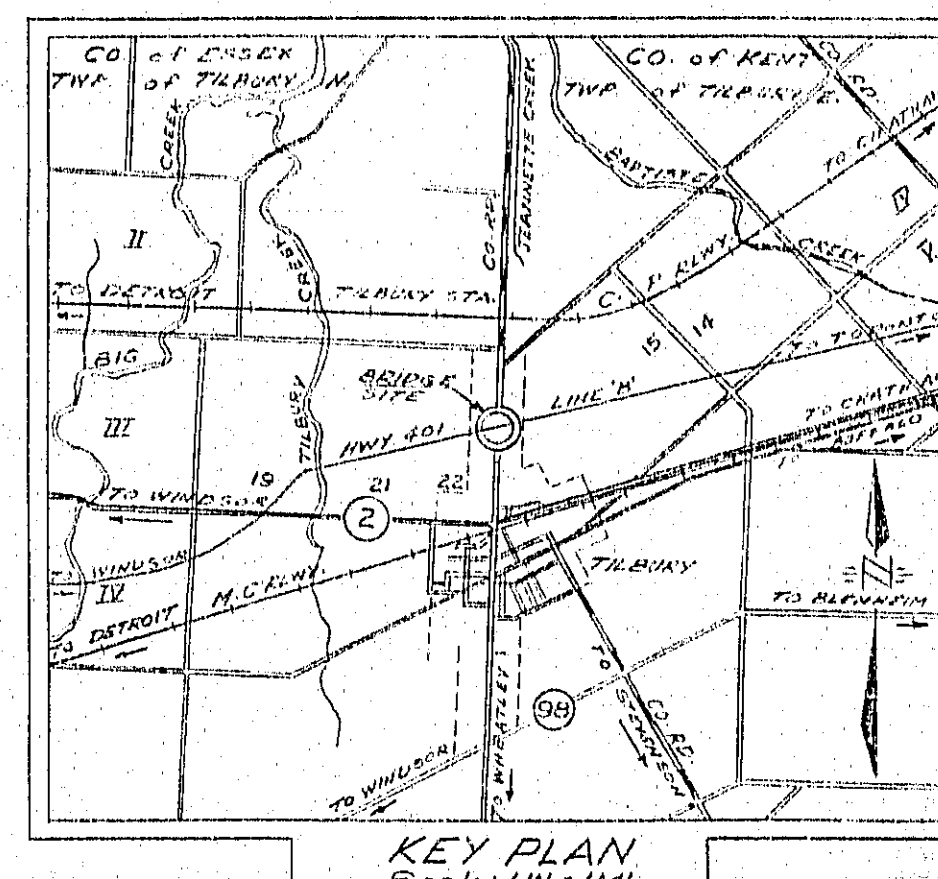
NOTES

To District Engineer - Concrete work on this structure must not be commenced until monuments to fix control points have been surveyed and checked by the District Engineer.
To Contractor - Structure to be built in accordance with Form 9 revised March 1957 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.
Concrete Mix - Minimum strength at 28 days
Footings: 2500 P.S.I. Structure: 3000 P.S.I.
Retaining Walls: 3000 P.S.I.
An approved admixture supplied by the Department will be added to all concrete as specified by the Engineer.
Reinforcing Steel - The complete soil investigation report, B-1505 may be examined at the Bridge Office, Downsview, Ontario. The Department does not guarantee the accuracy of this report or the bridged version shown on these plans.
Construction Notes - All exposed edges to be chamfered 1/4".
Formwork supporting wing walls not to be removed until the curb concrete has reached a strength of 2400 P.S.I.
Footings to be excavated to neat dimensions as shown and concrete placed against undisturbed ground.

REVISIONS	DATE	BY	DESCRIPTION

REFERENCE PLANS	DESIGN	CHECK	CONTRACT	DESIGNER
PROFILE F-3534-2	K.G.B.	L.N.F.	61-109	60-111
PLAN F-3534-7	F.W.	L.N.F.		
PLAN F-3516-1				
SOIL REPORT B-1505				

SITE PLAN
Scale: 1" = 100'-0"



KEY PLAN
Scale: 1/4" = 1'-0"

WP 161-58

DEPARTMENT OF HIGHWAYS-ONTARIO
BRIDGE OFFICE-TORONTO

TILBURY NORTH TWP. BR. 1
QUEEN ST. OVERPASS

THE KING'S HIGHWAY No. 401 DIST. No. 1
CO. Essex & Kent
TWP. Tilbury N. & Tilbury E. LOT CON.

GENERAL PLAN

APPROVED	DESIGN ENGINEER	CONTRACT	DESIGNER
		61-109	60-111

TWP #105-51-1-A



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 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