



THURBER ENGINEERING LTD.
FOUNDATION INVESTIGATION AND
DESIGN REPORT

PARK ROAD SOUTH OVERPASS
REPLACEMENT
HIGHWAY 401

CITY OF OSHAWA, ONTARIO

SITE 22X-173/B1&B2

G.W.P. 2555-17-00

GEOCRES NO. 30M15-348

Latitude: 43.879802°

Longitude: -78.863556°

Client Name: Egis Canada Ltd.

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PART 1: FACTUAL INFORMATION

1. INTRODUCTION

This report presents the factual findings obtained from a foundation investigation carried out by Thurber Engineering Ltd. (Thurber) for the proposed replacement of the existing Highway 401 Park Road South overpass structure located in the City of Oshawa, Ontario.

The purpose of this investigation was to explore the subsurface conditions at the proposed foundation locations, and based on the data obtained, to provide borehole location and soil strata drawings, records of boreholes, laboratory test results, and a written description of the subsurface conditions. A model of the subsurface conditions was developed for the site, based on data obtained from the present investigation, to describe the geotechnical conditions influencing design and construction of the foundations and approach embankments for the structures.

Thurber was retained by Egis Canada Ltd. (Egis) (formerly McIntosh Perry) to carry out this foundation investigation under the Ministry of Transportation Ontario (MTO) Agreement Number 2019-E-0076. The overall assignment includes replacement of the Highway 401 at Park Road South and Cubert Street underpass structures, new and proposed retaining walls and noise barrier walls on both sides of the highway, and overhead signs. This report addresses the proposed replacement of the Park Road South overpass structures.

Reference has been made to information on subsurface conditions contained in previous foundation reports prepared by others for this site. The titles of these reports are:

- Preliminary Foundation Investigation Report, Park Road Overpass, Site No. 22-173,

Highway 401 Improvements from Brock Road to Courtice Road, Regional Municipality of Durham, W.O. 10-20011, prepared by Golder Associates Ltd., GEOCRES No.30M15-296, dated May 19, 2017 (Reference 1).

- Foundation Investigation Report for Proposed Overpass Structure Extension At the Crossing of Park Rd. and Hwy. 401, Township of E. Whitby, Site No. 22-173, W.O. 72-11149, W.P. 44-71-06, prepared by Foundations Office, Ministry of Transportation and Communications Ontario, GEOCRES No.30M15-7 dated March 21, 1973 (Reference 2).

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

2. SITE DESCRIPTION

The existing Park Road South overpass structures are located some 600 m east of Stevenson Road in the City of Oshawa, Ontario. Park Road South generally runs in a north-south direction and carries two lanes of traffic in each direction under Highway 401. The existing overpass consists of single-span concrete rigid frame structures supported on spread footings.

We understand that the original structure was constructed in 1939. In the 1970's, the bridge was widened on both the north and south sides to accommodate three travel lanes on Highway 401. There is a pedestrian tunnel running parallel to Park Road South through the east abutment of the bridge. Archive information indicates that the bridge footings are founded at approximate Elevation 104.7. The highway grade at this location is at approximate Elevation 111. The approach fills at the east and west abutments are in the order of 6.5 to 7 m above Park Road South.

It is understood that the existing structures are proposed to be replaced with twin structures, each of them to carry the Highway 401 WBL and EBL, respectively.

The overall surface topography in the vicinity of the site is relatively flat with the ground surface gently sloping towards the south. A residential development currently occupies the northeast quadrant of the bridge crossing, while some commercial development is present at the southeast quadrant. The land on the west side of the bridge is undeveloped.

Based on published geological information, the site area is located within the Iroquois Plain physiographic region. This region extends around the western shores of Lake Ontario and consists of lakebed and beaches of the former glacial Lake Iroquois. The subsoils in this area are

typically comprised of glacial tills and glaciolacustrine clays, silts and sands. Limestone bedrock underlies the soil deposits.

3. SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing program completed to date for Park Road South was carried out between October 27 and November 22, 2022 and, November 19 and 20, 2023, and consisted of drilling and sampling ten (10) boreholes, designated as Boreholes PRS-01 to PRS-10. The boreholes were terminated at depths ranging from 12.7 m to 22.6 (Elevations 97.7 to 83.9). The Record of Borehole sheets are provided in Appendix B.

The approximate locations of all available boreholes (previous and present investigations) are shown on the Borehole Location Plan and Stratigraphic Drawings in Appendix A.

Thurber obtained the co-ordinates of the as-drilled borehole locations in the field using a Trimble R10 GPS survey equipment and forwarded them to Egis, who then provided the ground surface elevations. It is understood that the horizontal and vertical accuracy of the survey results meet the MTO terms of reference requirements. The coordinates and elevations of the boreholes are given on the drawings and Record of Borehole sheets in Appendices A and B.

Lane closures and traffic control were implemented for drilling each borehole for the current investigation. Prior to commencement of drilling, utility clearances were obtained for all borehole locations.

The current boreholes were advanced using track-mounted and truck-mounted drill rigs using hollow stem augers as well as wash boring with tri-cone and PW casings. Soil samples were obtained at selected depth intervals using a 50 mm outside diameter split-spoon sampler driven in conjunction with the Standard Penetration Test (SPT) which was performed in accordance with ASTM D1586. Where encountered, rock coring was carried out using a HQ core barrel in conjunction with HW casings.

The current field investigation was supervised on a full-time basis by a member of Thurber's technical staff who marked/staked the boreholes in the field, directed the drilling, sampling and in-situ testing operations, logged the boreholes and processed the recovered soil and rock samples for transport to Thurber's laboratory for further examination and testing.

Groundwater conditions in the open boreholes were observed throughout the current drilling operations. Two monitoring wells (50 mm diameter Schedule 40 PVC) were installed and enclosed in filter sand in Boreholes PRS-01 and PRS-04 to permit groundwater level monitoring.

Details of the well installations are shown in Table 3.1.

Table 3.1: Borehole Completion Details

Foundation Unit	Borehole	Borehole Depth / Base Elevation (m)	Well Tip Depth / Elevation (m)	Completion Details
West Abutment (WBL bridge)	PRS-01	12.7 / 93.5	10.2 / 96.0	Borehole caved to 10.2 m. Monitoring well with 1.5 m slotted screen installed within sand filter from 8.6 m to 10.2 m, backfilled with bentonite from 8.6 m to 0.3 m, then with concrete to 0.15m, then asphalt to ground surface.
East Abutment (EBL bridge)	PRS-04	17.2 / 88.7	17.2 / 88.7	Monitoring well with 1.5 m slotted screen installed within sand filter from 17.2 m to 15.7m, bentonite from 15.7 m to ground surface.

All boreholes without monitoring well installations were backfilled upon completion of drilling in general accordance with O.Reg. 903. Once the final readings are taken, the two wells from the current investigation will be decommissioned in general accordance with O.Reg. 903. The asphalt surface was reinstated in boreholes drilled on the highway or road platform.

4. LABORATORY TESTING

The recovered soil samples were subjected to visual identification (VI) and natural moisture content determination. Selected soil samples were subjected to grain size distribution analyses (sieve and/or hydrometer), and Atterberg Limits testing. Point Load Testing was carried out on selected rock cores for estimating the unconfined compressive strength of intact rock. Geotechnical laboratory testing results of the current investigation are summarized on the Record of Borehole sheets in Appendix B and are presented on the figures in Appendix C.

In order to assess the potential for sulphate attack on concrete foundations, as well as the potential for metal corrosion associated with the buried portions of the structures, selected soil samples were submitted to SGS Canada Inc., a CALA accredited analytical laboratory in Lakefield, Ontario, for analytical testing for corrosivity parameters and sulphate content. The results of the analytical testing are summarized in Section 5.3 and are presented in Appendix C.

5. DESCRIPTION OF SUBSURFACE CONDITIONS

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

In general, the subsurface stratigraphy encountered at the site consists of pavement structure overlying embankment fill within the highway platform. Below the embankment fill and surficial fill beyond the highway, the native soils consist of compact to very dense sand and silt till interlayered with an extensive deposit of very stiff to hard clayey silt till. Firm to stiff clayey silt/silty clay and clayey silt till was found below the sand and silt till at the southerly portion of the crossing. The groundwater level was observed to be at approximately 5 m to 7 m depths below the Highway 401 grade, or within 2 m below the existing Park Road South grade.

More detailed descriptions of the individual stratum are presented below.

5.1 Pavement Structure

Pavement structure consisting of approximately 130 mm to 250 mm of asphalt overlying granular (gravelly sand, sand, silty sand, sand and silt) road base was encountered in the boreholes advanced through the Park Road South platform (Boreholes PRS-01 and PRS-03), and the Highway 401 platform (Boreholes PRS-05, PRS-06, PRS-7, PRS-08, PRS-09 and PRS-10). The granular fill ranged in thickness from 0.5 m to 1.3 m.

SPT 'N' values recorded in the granular fill ranged from 13 to 49 blows per 0.3 m of penetration indicating a compact to dense condition. The moisture contents measured on samples of the granular fill ranged approximately from 1 percent to 10 percent.

The results of grain size analyses conducted on samples of the granular fill are provided on the Record of Borehole sheets in Appendix B and illustrated on Figure C1 in Appendix C. The results are summarized as follows:

Soil Particle	Granular Fill (Percent)
Gravel	0 to 23
Sand	46 to 79
Silt	16 to 39
Clay	2 to 8

5.2 Fill

Embankment fill was encountered underlying the pavement structure in the boreholes advanced from the Highway 401 platform (Boreholes PRS-05, PRS-06, PRS-07, PRS-08, PRS-09 and PRS-10). Fill was contacted surficially in Boreholes PRS-02 and PRS-04 beyond the roadway.

The embankment fill typically consisted of layers of brown sand and silt and silty sand containing trace to some gravel, trace to some clay and occasional rootlets. In Boreholes PRS-08 and PRS-10, a 1.1 m to 1.5 m thick layer of clayey silt fill was encountered at 3.0 to 4.1 m depths, respectively. In Borehole PRS-09, layers of silty clay and clayey silt fill were encountered at 1.4 and 3.0 m depth. On Highway 401, the embankment fill varied in thickness from 2.7 m to 4.8 m. On Park Road South, the thickness of the fill was 0.7 m and 1.4 m. The depth to the base of the fill varied from 0.7 m to 5.6 m (Elevations 104.5 to 107.3).

The SPT 'N' values recorded in the cohesionless sands and silts fill ranged from 4 to 30 blows per 0.3 m of penetration indicating a loose to compact condition. SPT 'N' values of 2 to 6 blows per 0.3 m of penetration were measured in the cohesive clayey silt fill and silty clay fill, indicating a very soft to firm consistency.

The natural moisture contents measured on samples of the cohesionless fill generally ranged from 6 percent to 19 percent. Moisture contents of approximately 14 and 29 percent were measured in the cohesive fill.

The results of a grain size analyses conducted on samples of the cohesionless fill and a sample of the cohesive fill are provided on the Record of Borehole sheets in Appendix B and illustrated on Figures C2 and C3 in Appendix C. The results are summarized as follows:

Soil Particle	Cohesionless Fill (Percent)	Cohesive Fill (Percent)
Gravel	0 to 15	1
Sand	35 to 53	47
Silt	29 to 59	40
Clay	5 to 15	12

5.3 Sand and Silt Till

Brown to grey sand and silt till containing trace to some gravel, trace clay and occasional cobbles or boulders was encountered below the fill, interlayered with or underlying the clayey silt and clayey silt till. In Boreholes PRS-01, PRS-02, PRS-03, PRS-04 and PRS-10, the sand and silt till was found underlying existing fill at depths ranging from 0.7 m to 5.6 m. In Borehole PRS-05, the sand and silt till was contacted below the clayey silt at 5.6 m depth. In Borehole PRS-09, the sand and silt till was contacted below the clayey silt till at 5.6 m depth. Lower layers of sand and silt till were contacted at depths ranging from 10.2 m to 20.7 m depth in Boreholes PRS-03, PRS-06, PRS-07, PRS-08 and PRS-10. Where fully penetrated, the thickness of the sand and silt till varied from 1.6 m to 3.4 m.

The depth to the base of the sand and silt till ranged from 4.1 m to 10.0 m (Elevations 101.6 to 104.2). Boreholes PRS-06, PRS-07, PRS-08 and PRS-10 were terminated within the lower sand and till layer, upon refusal, at depths ranging from 13.7 to 22.6 (Elevations 88.8 to 97.7).

The SPT 'N' values recorded in the sand and silt till ranged from 10 to 84 blows per 0.3 m of penetration and increasing with depth to greater than 100 blows for less than 0.3 m penetration, indicating compact to very dense condition. An SPT 'N' value of 6 blows per 0.3 m of penetration, indicating a loose state, was measured in Borehole PRS-04 near Elevation 104. The natural moisture contents measured on samples of the cohesionless till ranged from 5 percent to 18 percent.

The results of grain size distribution analyses carried out on selected samples of the sand and silt till are shown on Figure C4 in Appendix C. The results are summarized as follows:

Soil Particle	Sand and Silt Till (Percent)
Gravel	0 to 7
Sand	40 to 68
Silt	32 to 44
Clay	0 to 13

Glacial tills inherently contain cobbles and boulders. Augers grinding and/or split spoon sampler refusal were noted in the cohesionless till in Boreholes PRS-01, PRS-02 and PRS-03.

The results of Atterberg Limits tests conducted on a sample of the sand and silt till are presented on the Record of Borehole sheets in Appendix B and illustrated in Figure C10 of Appendix C. The results are summarized as follows:

Index Property	Percentage (%)
Liquid Limit	13
Plasticity Index	3

The results of the Atterberg Limits testing indicate that the sand and silt till is non-plastic with a group symbol of ML.

5.4 Clayey Silt to Silty Clay

Layers of brown to grey clayey silt with sand and silty clay with sand containing trace gravel were contacted at 4.1 m depth in Boreholes PRS-04 and PRS-05. The thicknesses of the clayey silt to silty clay were 5.9 m and 1.5 m in Boreholes PRS-04 and PRS-05, respectively.

The depths to the base of the clayey silt with sand and silty clay with sand were at 10.0 m and 5.6 m (Elevations 95.9 and 105.7) in Boreholes PRS-04 and PRS-05, respectively.

SPT 'N' values measured in the clayey silt and silty clay ranged from 1 to 10 blows per 0.3 m of penetration. Shear strengths from field vane tests carried out in the clayey silt at selected depths in Borehole PRS-04 were approximately 65 kPa and 100 kPa. This combined information indicates that the clayey silt has a generally firm to stiff consistency at locations where the 'N' values are less than 10 blows.

Moisture contents measured in the clayey silt and silty clay ranged approximately from 10 percent to 21 percent.

The results of grain size distribution analyses carried out on selected samples of the clayey silt and silty clay are presented on the Record of Borehole sheets included in Appendix B. Grain size distribution curves of samples tested are presented on Figure C5 in Appendix C. The results of the grain size distribution analyses are summarized below:

Soil Particle	Clayey Silt to Clayey Silt (Percent)
Gravel	0 to 5
Sand	25 to 49
Silt	38 to 52
Clay	8 to 30

The results of Atterberg Limits tests conducted on samples of the clayey silt and silty clay are presented on the Record of Borehole sheets in Appendix B and illustrated in Figure C11 of Appendix C. The results are summarized as follows:

Index Property	Percentage (%)
Liquid Limit	12 to 24
Plasticity Index	8 to 15

The results of the Atterberg Limits testing indicate that the clayey silt generally of slight plasticity with a group symbol of CL-ML. The silty clay has a low plasticity with a group symbol of CL.

5.5 Clayey Silt Till

The fills and native clayey silt, silty clay and sand and silt till are underlain by an extensive deposit of brown to grey clayey silt till with sand, trace gravel and occasional cobbles. This cohesive till was encountered in all the boreholes at depths ranging between 4.1 m and 10.0 m. Where fully penetrated in Boreholes PRS-02, PRS-03, PRS-07, PRS-08 and PRS-10, the thickness of the clayey silt till ranged from 6.1 m to 13.1 m. Locally, in Boreholes PRS-01 and PRS-06, the thickness of the clayey silt till was 1.5 m and 16.6 m, respectively. A 1.5 m thick layer of clayey silt till was encountered below the fill in Borehole PRS-09.

The depth to the base of the clayey silt till varied from 5.6 m to 20.7 m (Elevations 89.3 to 101.2). Boreholes PRS-01, PRS-04, PRS-05 and PRS-09 were terminated within the clayey silt till at depths ranging from 12.7 m to 22.1 m (Elevations 88.7 to 93.5).

SPT 'N' values measured in the clayey silt till typically ranged from 8 blows per 0.3 m penetration to greater than 100 blows for less than 0.3 m of penetration, indicating a stiff to hard consistency. Higher 'N' values may be attributed to the presence of cobbles and boulders. In Boreholes PRS-03, PRS-09 and PRS-10 located along the south side of the highway corridor, lower SPT 'N' values ranging between 5 to 7 blows per 0.3 m penetration were recorded indicating a firm consistency.

Shear strength from a field vane test carried out in the clayey silt till at about 10.2 m depth in Borehole PRS-04 was higher than 150 kPa, indicating a very stiff consistency. Shear strength from field vane tests carried out in the clayey silt till between 10.2 m and 16.6 m depth in Borehole PRS-09, ranged from 88 kPa to greater than 100 kPa, indicating a stiff to very stiff consistency.

Moisture contents measured in the clayey silt till ranged approximately from 6 percent to 31 percent.

The results of grain size distribution analyses carried out on selected samples of the clayey silt till are presented on the Record of Borehole sheets included in Appendix B. Grain size distribution curves of samples tested are presented on Figures C6 to C8 in Appendix C. The results of the grain size distribution analyses are summarized below:

Soil Particle	Clayey Silt Till (Percent)	
Gravel	0 to 13	
Sand	25 to 51	
Silt	32 to 49	57
Clay	8 to 29	

The results of Atterberg Limits tests conducted on samples of the clayey silt till are presented on the Record of Borehole sheets in Appendix B and illustrated in Figures C12 to C14 of Appendix C. The results are summarized as follows:

Index Property	Clayey Silt Till Percentage (%)
Liquid Limit	14 to 24
Plasticity Index	4 to 12

The results of the Atterberg Limits testing indicate that the clayey silt till are generally of low to slight plasticity with group symbols of CL to CL-ML.

Glacial tills inherently contain cobbles and boulders.

5.6 Limestone Bedrock

The clayey silt till was found to be underlain by limestone bedrock of the Lindsay Formation. Based on published geological information, the site is located at the boundary between the Lindsay Formation limestones and Whitby Formation shales. The limestone is generally described as moderately weathered becoming slightly weathered with depth, grey to black with frequent shale interbeds, laminated and horizontally bedded. The joints are largely horizontal to sub-horizontal and there are occasional fractured zones. Bedrock was proved by coring below the termination of augering in Boreholes PRS-02 and PRS-03, and inferred in Boreholes PRS-04, PRS-05 and PRS-06. Rock core photos are presented in Appendix C.

Table 5.1 below summarizes depths to bedrock and top of bedrock elevations encountered in these boreholes.

Table 5.1: Top of Bedrock

Foundation Element	Borehole Number	Approximate Top of Bedrock		
		Depth below highway grade or adjacent ground (m)	Elevation (m)	
WBL Replacement Bridge				
West Abutment	PRS-05	22.1	89.2	Inferred
East Abutment	PRS-02	17.2	89.3	Proved by coring
	PRS-06	22.6	88.8	Inferred
EBL Replacement Bridge				
West Abutment	PRS-05	22.1	89.2	Inferred
	PRS-03	16.0	89.7	Proved by coring
East Abutment	PRS-06	22.6	88.8	Inferred
	PRS-04	17.2	88.7	Inferred

Total Core Recovery (TCR) in the core runs are at 100 percent, except for Run #1 in Borehole PRS-03 where 80 percent was recorded. The Rock Quality Designation (RQD) values typically ranged from 66 percent to 92 percent indicating fair to excellent rock quality. A lower RQD value of 28 percent, indicating poor rock quality, was recorded for Run 1 in Borehole PRS-03 which contained fractured zones. The Fracture Index (FI) of the rock, expressed as number of fractures per 0.3 m of core, typically ranged from 0 to 5, except at the fractured zones where the FI is greater than 10.

The unconfined compressive strength (UCS) of the limestone, estimated from the results of point load tests, typically ranged from 40 to 93 MPa indicating a medium strong to strong rock. Occasional UCS values of over 100 MPa indicate the presence of very strong zones within the rock mass. Results of these rock strength tests are summarized on the Records of Boreholes in Appendix B and presented in Appendix C.

5.7 Groundwater Conditions

Groundwater levels in the boreholes were observed during the drilling operations and measured upon completion of drilling. Monitoring wells were installed in Boreholes PRS-01 and PRS-4 to permit monitoring of groundwater levels. Water was added to all the boreholes in conjunction with tri-coning and rock coring for borehole advancement; therefore stabilized groundwater levels might not established in some boreholes upon completion of drilling. Water levels recorded in open boreholes upon completion were likely not stabilized readings (see table below).

Water levels measured in the wells and open boreholes are presented in Table 5.2 below.

Table 5.2: Groundwater Level Measurements

Borehole	Date	Groundwater Level		Comments
		Depth (m)	Elevation (m)	
PRS-01	April 14, 2023 May 18, 2023	0.3 0.0	105.9 106.2	Monitoring Well (sealed in sand and silt till)
PRS-02	November 18, 2022	0.0*	106.5*	Open borehole upon completion
PRS-03	November 15, 2022	0.0*	105.7*	Open borehole upon completion
PRS-04	April 14, 2023 May 18, 2023	1.8 1.9	104.1 104.0	Monitoring Well (sealed in clayey silt till)
PRS-05	November 11, 2022	3.7*	107.6*	Open borehole upon completion
PRS-06	November 2, 2022	2.9*	108.5*	Open borehole upon completion
PRS-07	October 28, 2022	1.1*	110.2*	Open borehole upon completion
PRS-08	October 28, 2022	3.0*	108.4*	Open borehole upon completion
PRS-10	November 9, 2022	2.7*	108.7*	Open borehole upon completion

Note: * May not be stabilized.

It is noted that the perched water level within the cohesionless interlayers, as shown by PRS-01, is higher than the groundwater table within the cohesive till as shown by PRS-04.

The groundwater levels measured in the two monitoring wells in Table 5.2 are short-term readings where seasonal fluctuations are to be expected. In particular, the groundwater level may be at a higher elevation after periods of significant or prolonged precipitation.

5.8 Corrosivity Test Results

Selected soil samples were submitted for analytical testing of corrosivity parameters including sulphate content. The results of the analytical tests are shown in Table 5.3. The laboratory certificates of analysis and detail results are presented in Appendix C.

Table 5.3: Analytical Corrosivity Test Results

Sample ID	Depth (m)	Soil Sample Description	Sulphide (percent)	Chloride (µg/g)	Sulphate (µg/g)	pH	Resistivity (ohm.cm)	Redox Potential (mV)	Electrical Conductivity (µS/cm)
PRS-01 SS5	3.0 – 3.6	Sand and Silt Till	0.06	47	100	8.63	5,590	254	179
PRS-02 SS5	3.0 – 3.6	Sand and Silt Till	0.05	40	120	8.52	7,300	210	137
PRS-03 SS5	3.0 – 3.6	Sand and Silt Till	0.06	2,400	160	8.39	2,480	241	403
PRS-04 SS4	2.3 – 2.9	Sand and Silt Till	0.06	3,400	160	8.61	972	204	1,030

6. MISCELLANEOUS

Thurber staked and/or marked the borehole locations in the field and obtained utility clearances prior to drilling. Thurber surveyed the as-drilled boreholes in the field, and forwarded the borehole coordinates to Egis who provided the ground surface elevations.

Landshark Drilling of Brantford, Ontario supplied and operated the drilling and sampling equipment for the field program.

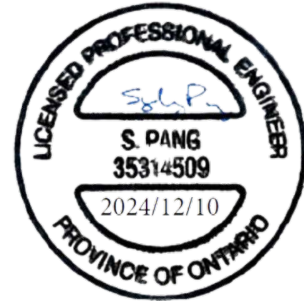
Full time supervision of the field activities was carried out by Mr. Sergey Gladkiy of Thurber. Overall supervision of the field program was performed by Messrs. Rod de Castro, P.Eng. and Cory Zanatta, P.Eng. of Thurber.

Interpretation of the field data and preparation of the report was carried out by Sydney Pang, P.Eng. and assisted by Rocio Reyna, P.Eng. This report was reviewed by P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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File: **30915**



THURBER ENGINEERING LTD.

**FOUNDATION INVESTIGATION AND DESIGN REPORT
PARK ROAD SOUTH OVERPASS REPLACEMENT
HIGHWAY 401
CITY OF OSHAWA, ONTARIO
SITE 22X-173/B1&B2
G.W.P. 2555-17-00**

GEOCRES NO. 30M15-348

PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7. GENERAL

This report presents interpretation of the geotechnical data in the factual report and provides foundation recommendations to assist the design team in selecting and designing a suitable foundation system for the proposed Highway 401 Park Road South overpass replacement structures in Oshawa, Ontario.

This foundation investigation and design report, with the interpretation and recommendations, is intended for the use of the Ministry of Transportation (MTO) and Egis Canada Ltd. (Egis) (formerly McIntosh Perry) and shall not be used or relied upon for any other purposes or by any other parties including the construction contractor. The contractors must make their own interpretation based on the factual data in Part 1 of the report. Where comments are made on construction, they are provided only in order to highlight those aspects, which could affect the design of the project. Contractors must make their own interpretation of the information provided as it may affect equipment selection, proposed construction methods and scheduling.

Original Structure

Information provided by Egis indicates that the original overpass structure was built in 1939. The bridge was a single-span concrete rigid frame structure with a clear span between abutment faces of about 13 m. The overall width encompassing the original WBL and EBL was approximately 30 m. It is understood that the abutments are supported by spread footings in the order of 1.1 m in width. There is no record available to confirm the founding elevation of the existing structure.

Existing Structure

The original bridge was widened and rehabilitated in 1977 to form the existing structures. The structure widening was approximately 6.2 m to the north and south, resulting in an overall width of about 42.4 m. In addition, several aspects of structural rehabilitation were carried out. As part of the rehabilitation, a pedestrian tunnel was constructed along the east side of Park Road South by tunnelling through the existing east wingwalls and approach embankment. The tunnel was constructed using a corrugated steel liner together with 120 mm reinforced concrete.

Further minor rehabilitation works were undertaken in 2002 and 2008.

Visual observations of the existing bridge did not reveal obvious signs of settlement or distress at the foundation elements. The approach slopes appeared to be stable with no obvious signs of instability.

Proposed Replacement Structures

A 90% general arrangement (GA) drawing provided by Egis shows that the existing structures will be replaced by longer and wider WBL and EBL single span structures. The new clear span between abutment faces will be approximately 22.4 m. The WBL and EBL bridge decks will be approximately 29 m and 32 m in width, respectively. The new structures comprise welded steel plate girders resting on conventional concrete abutment walls supported on spread footings. Foundation alternatives for the new structures are discussed below.

The new structures will accommodate four Highway 401 lanes in each direction for a total of eight lanes. Park Road South will also be widened to carry two lanes of traffic in each of the north and south directions, plus a southbound ramp lane, for a total of five lanes.

In order to accommodate the widened structures, the road profile of Park Road South will be lowered by up to about 1.5 m below the existing road grade. The lowest grade of the new road profile will be at approximate Elevation 104.5. It is understood that there will be no change to the Highway 401 grade.

New fill will be required on both the north (WBL) and south (EBL) sides for widening of Highway 401. The top of highway embankment will be up to the order of 7.5 m above the road grade beyond the highway. Assuming a grade at approximate Elevation 104.5 at the lowest road profile (under centreline of Highway 401), the approach embankment height will be up to the order of 8 m below highway grade at that location.

The preliminary GA drawing also indicates that RSS walls are proposed beyond the wingwalls at the northwest, northeast and southwest corners of the overpass structures for retaining the widened Highway 401 embankments. These walls will generally be parallel to Park Road South and range from 12.4 to 17.3 m in length. At the southeast corner, an RSS wall is proposed to run

parallel to Highway 401 to retain the embankment due to property restrictions at that quadrant. The foundation field work for that long wall is in progress. Foundation investigation and design for this wall will be covered in a separate report.

The discussion and recommendations presented in this report are based on the design information provided by Egis to date, the factual data obtained during the course of the current investigation and selected data from a previous preliminary investigation by others.

8. STRUCTURE CLASSIFICATION

In accordance with the currently applicable Canadian Highway Bridge Design Code (CHBDC) (2019) CSA S6-19, the analysis and design of structures are influenced by its importance category and consequence classification. Such designations are defined by the Regulatory Authority which, in this case, is the Ministry of Transportation of Ontario (MTO).

For the purpose of reporting, this structure has been classified as a Major-Route Bridge with Typical Consequence based on CHBDC S6-19 Sections 4.4.2 and 6.5.2, respectively. As per CHBDC (2019) Clause 6.5.3, a typical degree of understanding is considered for this site.

Based on the above classification and Table 6.1 in Section 6.5.2 in the CHBDC 2019, a consequence factor, ψ , of 1.0 has been used for assessing factored ULS and SLS geotechnical resistances. Should the consequence classification change, the geotechnical assessment and recommendations will need to be reviewed and revised as necessary.

9. BRIDGE FOUNDATION DESIGN

In general, the subsurface stratigraphy encountered at the site consists of pavement structure overlying embankment fill within the highway platform. Below the embankment fill and surficial fill beyond the highway, the native soils consist of compact to very dense sand and silt till interlayered with an extensive deposit of very stiff to hard clayey silt till. Firm to stiff clayey silt/silty clay and clayey silt till was found below the sand and silt till at the southerly portion of the crossing. The groundwater level was observed to be at approximately 5 m to 7 m depths below the Highway 401 grade, or within 2 m below the existing Park Road South grade.

9.1 Foundation Alternatives

Based on the subsurface conditions and project requirements, consideration was given to supporting the new bridge abutments using the following foundation types:

- Spread footings on native soils
- Spread footings on engineered fill
- Driven steel H-piles
- Drilled shafts (caissons)

A comparison of the technical advantages, disadvantages, relative risks and costs of the alternative foundation schemes is presented in Appendix F. Discussions on feasible foundation alternatives are presented in the following paragraphs. A preferred foundation scheme from a foundation engineering perspective is then recommended.

Spread Footings on Native Soils

Spread footings founded on native stiff to hard clayey silt till with some compact to very dense sand and silt till interlayers are considered technically feasible to support the new bridge abutments and new retaining walls associated with the bridges. This foundation option will preclude the use of integral abutments. Given that the Park Road South profile will be lowered up to 1.5 m and the footings will have to be founded at a minimum 1.2 m of frost depth below final road grade, the footing founding levels would likely be at or just below the groundwater table during construction. Adequate groundwater control in conjunction with temporary protection will be required for footing construction in the dry. It is anticipated that the temporary protection system will have a retained height up to the order of 8 to 9 m below Highway 401. This deep temporary protection system is likely going to require struts and/or soil anchors for support to limit lateral movements to within tolerable limits. The construction activities will have to be well co-ordinated as part of staged construction to minimize impact on the travelled lanes on the highway.

Spread Footings on Engineered Fill

Where there is the presence of less competent foundation soil near the southerly portion of the EBL bridge, the spread footings may be founded on a pad of engineered fill resting on native subgrade. This arrangement will spread the load and result in higher bearing resistances for optimizing the footing design. Localized additional excavation with deeper temporary protection system will be required.

Driven Steel H-Piles

Steel H-piles driven to practical refusal into the “100+ blow” materials, whether it be the hard clayey silt till or the very dense sand and silt till, could be used to support the new bridge abutments. This foundation option would permit integral abutment design should it be considered. The pile caps could either be perched within the approach embankments or located within the native glacial tills at shallow depths.

The driven pile option will require temporary protection and limited groundwater control during pile cap construction. There are potential obstructions due to cobbles and boulders, and hard driving conditions are anticipated within the hard or very dense glacial tills.

Drilled Shafts (Caissons)

If integral abutments are not used, augered caisson foundations founded on the underlying hard clayey silt till or very dense sand and silt till are feasible for foundation support of the proposed bridges at this site.

Construction of caissons through these soils will require use of a temporary steel liner in conjunction with water and/or slurry to control the ingress of groundwater, support the sidewalls of the hole and mitigate basal instability. The caisson equipment should also be capable of handling and otherwise removing oversize obstructions such as cobbles and boulders, and penetrating the hard and very dense soils present at this site.

Should a caisson foundation be considered, further foundation recommendations and comments will be provided.

Recommended Foundations

From a foundation technical and cost effectiveness perspective, the preferred foundation alternative for the new bridges is driven piles to practical refusal if integral abutments are considered. If not, spread footings on native soils or on engineered pads, where required, may be used for bridge foundation support. The choice of foundation types would depend on the relative cost effectiveness between foundation alternatives and preference in structural design considerations.

9.2 Spread Footings

9.2.1 Geotechnical Resistance

Given that the Park Road South profile will be lowered up to 1.5 m to accommodate the bridge widening, the new abutments may be supported by spread footings with founding levels at or below the design frost depth of 1.2 m on undisturbed, native very stiff to hard clayey silt till. Boreholes PRS-03, PRS-04 and PRS-10, located at the southerly portion of the EBL (south) bridge footprint, reveal the presence of firm to stiff clayey silt/silty clay and clayey silt till. At those locations, geotechnical resistances can be increased by founding the spread footings on a compacted Granular A pad which is itself resting on undisturbed, native clayey silt till.

Table 9.1 below presents the recommended highest founding level and anticipated founding soils at the footing locations.

Table 9.1: Recommended Founding Conditions for Spread Footings

Foundation Element	Foundation Location	Reference Boreholes	Highest Founding Elevation (m)	Subgrade Soil Types
WBL (North) Bridge	West Abutment	PRS-01 PRS-07	103.2	Very stiff to hard Clayey Silt Till Interlayered with Compact to very dense Sand and Silt Till
	East Abutment	PRS-02 PRS-08	103.2	Very stiff to hard Clayey Silt Till
EBL (South) Bridge	West Abutment	PRS-03 PRS-05	103.2	Firm to hard Clayey Silt Till with interlayers of Dense to very dense Sand and Silt Till
	East Abutment	PRS-04 PRS-06 PRS-10	103.2	Firm to stiff Clayey Silt/Silty Clay and Clayey Silt Till overlying Very stiff to hard Clayey Silt Till

Table 9.2 below provides recommended geotechnical resistances for footing design.

Table 9.2: Recommended Geotechnical Resistances for Footing Design

Foundation Element	Foundation Location	Approx. Footing Width (m)	Factored Geotechnical Resistances			
			At ULS (kPa)	At SLS (kPa)	At ULS (kPa)	At SLS (kPa)
			Compacted Granular A Pad (1 m thick)			
WBL (North) Bridge	West Abutment	4 to 5	450	300	550	350
	East Abutment		450	250	550	300
EBL (South) Bridge	West Abutment	4 to 5	400	175	500	200
	East Abutment		400	175	500	200

The values of the Factored Geotechnical Resistance at ULS were assessed assuming a Consequence Factor equal to 1 (Typical), and a Resistance Factor equal to 0.5 (Typical degree of understanding of the subsurface conditions), as per CHBDC 2019. The Factored Geotechnical Resistance at SLS was assessed assuming a factor of 0.8 for typical degree of understanding of

the subsurface conditions. The founding subgrade may be considered as a cohesive soil for footing design purposes.

The above resistance values are for vertical, concentric loads. Where eccentric or inclined loads are applied, the resistance values used in design must be reduced in accordance with the CHBDC 2019 Clause 6.10.2 and Clause 6.10.3.

The factored geotechnical SLS values given above are based on an estimated total settlement not exceeding 25 mm. This settlement is expected to be substantially complete by the end of construction. Differential settlement is not expected to exceed 20 mm across the width of the structure or between foundation elements.

The sliding resistance of cast-in-place concrete placed on the native, undisturbed very stiff to hard clayey silt till may be computed based on an ultimate coefficient of friction, $\tan \delta$, of 0.4. The sliding resistance of cast-in-place concrete placed on compacted Granular A material may be computed based on an ultimate coefficient of friction, $\tan \delta$, of 0.55.

For frost protection purposes, all footing bases should have a minimum earth cover of 1.2 m or its thermal equivalent.

9.2.2 Footing construction

The base of the footing excavation should be inspected by a Foundation Engineering Specialist to confirm that the footing subgrade is predominantly the native, undisturbed stiff to hard clayey silt till with occasional compact to very dense sand and silt till, conforming to the design requirements and has been adequately prepared to receive concrete. A concrete working slab should be placed within 4 hours following completion of excavation to prevent deterioration of the approved subgrade. The working slab should be at least 100 mm thick and formed by mass concrete fill with a minimum unconfined compressive strength of 20 MPa.

For the WBL bridge, excavation is required to expose the very stiff to hard clayey silt till subgrade for footing construction. Firm to stiff cohesive soils with SPT 'N' values ranging between 8 and 11 blows, such as those encountered in Boreholes PRS-05 and PRS-7, may be exposed at the subgrade level. Where sub-excavation is required to remove such unsuitable materials from below the design founding level, the founding surface should be re-established using engineered fill or mass concrete as outlined above.

For the EBL bridge, the southerly portion of the footings is anticipated to be founded on subgrade containing the more compressible firm to stiff clayey silt/silty and clayey silt till. The transition between the competent and less competent subgrade is not well defined and is anticipated to be variable within the foundation footprint. For the southerly portion of the new footing footprints

located beyond the existing highway EBL shoulder, sub-excavation should be carried out to 1 m below the design founding elevation and an engineered Granular A pad should be placed to reinstate the founding level. The Granular A for the engineered fill pad must be compacted to 100 percent Standard Proctor Maximum Dry Density (SPMDD) at optimum moisture content ± 2 percent and placed in 150 mm lifts. The geometry of the fill pad should generally conform with the general requirements shown in Figure 1 in Appendix H. All footing construction procedures should follow the guidelines provided in OPSS 902. Suggested wordings for an NSSP outlining the above are included in Appendix I.

The groundwater table within the clayey silt till is approximately 1 m above the highest recommended footing base level, which is up to 3 m below the perched water level in the sand and silt till. The drainage system currently in place along the existing Park Road South is anticipated to have locally drawn down the groundwater level. For permanent conditions, the lowered profile of Park Road South with its long term drainage system would further drawdown the surrounding groundwater level.

The proportion of clay particles within the clayey silt till would temporarily impede water seepage. During construction, however, dewatering in conjunction with temporary protection (shoring) will be required for groundwater control due to seepage from the sand and silt till and other water-bearing layers, accumulation of precipitation and surface runoff, to facilitate dry excavations for footing construction.

Temporary protection and an effective dewatering system must be implemented for constructing the spread footings. Further details for groundwater control and temporary protection design are presented in Sections 15 and 16.

9.3 Driven Steel H-Piles

From a foundation engineering perspective, it is feasible to support the structures on steel H-piles driven to practical refusal within the “100-blow”, hard clayey silt till or the very dense sand and silt till. It is possible that some piles could fully penetrate the soils to reach the underlying bedrock. This foundation type is compatible with an integral abutment design should it be considered.

9.3.1 Axial Geotechnical resistance

It is recommended that the H-piles be driven to achieve resistance in the underlying “100-blow” hard clayey silt till or very dense sand and silt till. Some piles may reach refusal on the underlying bedrock.

It is anticipated that the underside of the abutment stem at the west and east abutments could be higher than the final road grade. The designer will need to carry out lateral pile analysis to determine the design pile embedment depth required to satisfy lateral resistance requirements.

For reporting purposes, it is assumed that the underside of the abutment walls would be at approximate Elevation 106.

Consideration may be given to using HP 310 X 110 steel piles driven to practical refusal within the hard or very dense “100-blow” tills. The recommended design axial factored geotechnical resistance at Ultimate Limit States (ULS) and factored geotechnical resistance at Serviceability Limit States (SLS), as well as the estimated pile tip elevations, are summarized in Table 9.3 below.

Table 9.3: Estimated Pile Tip Elevation and Geotechnical Resistances for H-Piles

Foundation Element	Foundation Location	Borehole	Approx. Pile Tip Elevation (m)	Minimum Pile Embedment Length (m)	HP 310 X 110	
					Factored ULS (kN)	Factored SLS (kN)
WBL (North) Bridge	West and East Abutments	PRS-01 PRS-07 PRS-02 PRS-08	90.0	16	1,500	1,300
EBL (South) Bridge	West and East Abutments	PRS-05 PRS-03 PRS-06 PRS-04	90.0	16	1,500	1,300

9.3.2 Lateral Geotechnical Resistance

Lateral bridge loadings can be geotechnically resisted by the driven H-piles through passive pressure developed along the embedded portion of the piles below the pile cap or under the abutment stem. Battered piles may also be used for conventional (non-integral) pile groups.

The geotechnical lateral resistance acting on a pile in cohesive soils may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

$$\begin{aligned}
 k_s &= 67 C_u / B \quad (\text{kPa/m}) \\
 p_{ult} &= 9 C_u \quad (\text{kPa}) \\
 \text{Where } C_u &= \text{undrained shear strength (kPa)} \\
 B &= \text{pile width (m)}
 \end{aligned}$$

The geotechnical lateral resistance of a pile in cohesionless soil may be calculated using a coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

$$\begin{aligned}
 k_s &= n_h z / B \quad (\text{kPa/m}) \\
 p_{ult} &= 3 \gamma' z K_p \quad (\text{kPa}) \\
 \text{Where } z &= \text{depth of embedment along pile (m)} \\
 B &= \text{pile width (m)} \\
 n_h &= \text{coefficient related to soil density (kPa/m)} \\
 \gamma' &= \text{submerged unit weight (kN/m}^3\text{)} \\
 K_p &= \text{coefficient of passive lateral earth pressure}
 \end{aligned}$$

The above equations and recommended parameters in Table 9.4 below may be used to analyse the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis must not exceed the ultimate lateral resistance.

Table 9.4: Recommended Geotechnical Parameters for Lateral Resistance Design

Foundation Element	Reference Boreholes	Approx. Elevation (m)	Undrained Shear Strength C_u (kPa)	Unit Weight γ (kN/m ³)	K_p	n_h (kN/m ³)	Soil Conditions
WBL Bridge	PRS-01 PRS-02 PRS-05 PRS-06 PRS-07 PRS-08	111.5 to 107	-	20	3.3	3,000	Loose to compact Sand and silt fill
		107 to 100	100	19	-	-	Stiff to very stiff Clayey silt till
		100 to 90	200	21	-	-	Very stiff to hard Clayey silt till
EBL Bridge	PRS-05 PRS-06 (north)	111.5 to 107	-	20	3.3	3,000	Loose to compact Sand and silt fill
		107 to 100	100	19	-	-	Stiff to very stiff Clayey silt till
		100 to 90	200	21	-	-	Very stiff to hard Clayey silt till
	PRS-03 PRS-04 PRS-10	106 to 104	-	20	3.3	3,000	Loose to compact sand and silt fill
		104 to 102	-	21	3.1	2,500	Compact Sand and silt till
		102 to 93	75	19	-	-	Firm to stiff Clayey silt till, clayey silt/silty clay
		93 to 90	200	21	-	-	Very stiff to hard Clayey silt till

The spring constant, K , for analysis may be obtained by the expression, $K = k_s \times d_z \times B$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kPa/m), B is the pile width (m), d_z is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance on any one segment of pile, P_{ult} , may be obtained from the expression, $P_{ult} = p_{ult} \times d_z \times B$. This represents the ultimate load at the contact between the pile and the surrounding soil above which any additional load will not be supported at greater displacements.

The group efficiency factors can be calculated based on side-by-side and line-by-line factors shown in Figures C6.22, C6.23, and C6.24 of the CHBDC 2019, S6.1.19 (Commentary).

9.3.3 Pile Installation

Pile installation shall be in accordance with OPSS.PROV 903.

Pile driving must be controlled by the use of the Hiley Formula for acceptance. Appropriate pile driving notes are “Piles to be driven in accordance with Standard SS 103-11 using an ultimate geotechnical resistance equal to two (2) times the ULS value per pile”, and “Piles are to driven to the tip elevations shown in the table below”.

Foundation Unit	Estimated Pile Tip Elevations (m)
WBL Bridge West and East Abutments	90.0
EBL Bridge West and East Abutments	90.0

In addition, high strain dynamic testing (also commonly known as PDA testing) should be carried out for selected piles as required to confirm the pile resistance. PDA testing should also be carried out at the beginning of pile driving to establish a target set for subsequent driving. A minimum of 10 percent of the total number of piles and not less than 2 piles per foundation element should be subjected to PDA testing.

To facilitate pile installation, any fill through which piles will be driven must not contain any material with particle sizes greater than 75 mm.

Glacially derived soils inherently contain cobbles and boulders. At some locations at this site, the piles will have to be driven through very dense or hard soils, and therefore difficult driving conditions should be expected. In order to protect the piles while being driven through obstructions and denser or harder zones to achieve the required tip elevations and soil resistance, it is recommended that the pile tips be reinforced with Titus Steel Standard H-points or an approved equivalent. Over-driving must be avoided to minimize the risk of damaging the pile.

The presence of boulders and cobbles, and the “100-blow” soils, could prevent some piles from reaching the design pile tip elevations. An NSSP on pile installation is included in Appendix I with statements stipulating that pre-augering should be carried out at some pile locations prior to driving the piles. Each pre-augered hole should not be greater than 200 mm in diameter and reverse augering should be carried out as the auger is retrieved to leave as much soil cuttings as possible inside the hole.

Pile driving can induce pore pressure build-up within the silty soils immediately surrounding the pile resulting in temporary reduction of the pile geotechnical resistance. Such resistance is anticipated to increase with time as the pore pressure dissipates after initial installation. It is recommended that a wait period of at least 3 days be specified before allowing retapping for confirmation of the pile geotechnical resistance.

9.4 Frost Cover

The design depth of frost penetration at this site is 1.2 m. The base of footings or pile caps must be provided with a minimum of 1.2 m of earth cover, or its thermal insulation equivalent, as protection against frost action.

10. RETAINED SOIL SYSTEMS (RSS) WALLS

The 90% GA and wall layout drawings indicate that RSS walls are proposed beyond the wingwalls at all four corners of the overpass structures for retaining the widened Highway 401 embankments. A summary of RSS walls details is provided in Table 10.1.

Table 10.1: Summary of Proposed RSS Walls

RSS Wall Location	Length (m)	Skew relative to abutment centreline	Proposed Underside Elevation of Granular Pad (m)
Northwest	9.5	0° ⁽¹⁾ Parallel to Park Rd.	103.2
Northeast	10.0	25° ⁽²⁾	103.2
Southwest	10.2	17° ⁽¹⁾	103.0
Southeast	2.5	0° ⁽²⁾ then turn 90° east ⁽³⁾	102.6 to 104.1

⁽¹⁾ Relative to west abutment centreline.

⁽²⁾ Relative to east abutment centreline.

⁽³⁾ Refer to report on SE retaining wall.

The proposed wall height is anticipated to be up to the order of 6.5 m to 7 m adjacent to the

bridges and progressively decrease away from the bridges.

RSS walls used for this project must be specified to be “High Performance” and “High Appearance”. The soil conditions encountered near the wall alignment are generally suitable for the support of RSS walls. The contract drawings should include information on the longitudinal alignment of the wall in plan, the top and base elevations of the wall in profile, cross-sections, space constraints and NSSPs for the RSS wall. The underside of the RSS mass, including the concrete levelling pads supporting the front panels, may be stepped to accommodate topographic variations.

The performance of a RSS mass is dependent on, amongst other factors, the characteristics of its foundation. Failure to provide an adequate foundation may lead to settlement and distortion of the RSS and, in severe cases, to possible failure of the system. The foundation of the entire RSS mass must be considered, i.e. from the face of the wall to the furthest extent of the reinforcement. To provide acceptable foundation performance and based on available borehole information, it is recommended that the RSS masses be founded at or below elevations presented in Table 10.1, which also contains recommended geotechnical resistances.

10.1 Geotechnical Resistance

Based on the subsurface conditions near the wall alignments, the wall footings may be founded at or below the design frost penetration depth of 1.2 m below the existing ground. For design and planning purposes, an existing ground surface at approximate Elevation 106 has been assumed. Given a proposed road profile lowering up to 1.5 m, the proposed ground surface will be at approximate Elevation 104.5 at the deepest location. The founding elevations and geotechnical resistances recommended for these footings are presented in Table 10.2 below.

Table 10.2: Recommended Parameters for RSS Wall Foundation Design

RSS Wall	Reference Borehole	Highest Granular Pad Founding Elevation (m)	Founding Soil Type	Factored Geotechnical Resistance	
				ULS (kPa)	SLS (kPa)
Northwest	PRS-01 PRS-07	103.2	Compact to very dense Sand and silt till interlayered with Very stiff to hard Clayey silt till	450	300
Northeast	PRS-02 PRS-08			450	250
Southwest	PRS-03	103.0	Compact to very dense Sand and silt till overlying Firm to very stiff Clayey silt till, clayey silt/silty clay	400	175

The southeast RSS wall is covered in a separate report.

The RSS mass should be founded on a minimum 0.5 m thick of engineered fill consisting of OPSS.PROV Granular A compacted to 100 percent of its Standard Proctor Maximum Dry Density (SPMDD) at a moisture content within 2 percent of optimum. The engineered pad must laterally extend at least 500 mm beyond the footprint of the RSS mass and levelling strip.

As per MTO RSS Design Guidelines, the top of the levelling pad should be placed at a depth below final grade not less than the larger of 0.8 m or 40% of frost depth (1.5 m), or 0.8 m in front of the wall.

The geotechnical resistances provided above are for concentric, vertical loading. The effects of load inclination and eccentricity need to be taken into account according to the CHBDC 2019 Clauses 6.10.2 and 6.10.3.

The entire block of reinforced earth must be designed against various modes of failure including sliding and overturning. Sliding resistance along the base of the wall may be estimated using an ultimate friction coefficient of 0.55 for an engineered granular fill subgrade and 0.4 for a compact to very dense sand and silt till subgrade. A Resistance Factor of 0.8 should be applied for the cohesionless soils as indicated in Table 6.2 of the CHBDC 2019.

Topsoil, organics, loose/soft, wet materials and debris must be stripped from the footprint of the RSS. The subgrade under the RSS foundation should be inspected and any loose/soft spots sub-excavated and replaced with compacted granular materials prior to placing fill. The subgrade preparation for the RSS wall, placement and compaction of the granular fill must be carried out in the dry.

The proprietary RSS system must meet the MTO's specifications for performance and appearance. The RSS supplier/designer may specify more stringent criteria or other requirements related to the particular design. The internal stability of the RSS wall should be analyzed by the supplier/designer of the proprietary product selected for this site.

RSS walls for bridge abutments must be selected from MTO DSM List 9.70.52.

RSS walls must be designed and constructed in accordance with MTO RSS SP 599S22 and SP 599S23.

10.2 Global Stability at Retaining Wall

Preliminary global stability analyses were carried out to assess stability of the RSS walls founded on native, undisturbed glacial tills. These analyses were carried out for the proposed RSS walls utilizing the commercially available slope stability analysis program Slope/W (Version 2022) of

the GeoStudio software package developed by Geo-Slope International with the option for Morgenstern-Price method of slices for the limit equilibrium analyses. Analyses were completed for both static and seismic loading conditions. These analyses should be revisited and confirmed after the final location and detail configurations of the walls are finalized.

The soil parameters used in the analyses were estimated from empirical correlations using the results of the in situ Standard Penetration Tests (SPTs), field vane shear tests and geotechnical laboratory testing. The groundwater level in our analysis was based on readings obtained to date from standpipe piezometers. The stability of the embankment was checked for the pseudo-static condition assuming a peak horizontal acceleration of 0.0845 g.

Results of the stability analyses are presented on Figures G1 to G3 in Appendix G. The results are also summarized in Table 10.3 below.

Table 10.3: Computed Factors of Safety

Condition	Factor of Safety	Figure (Appendix G)
RSS Walls beyond abutment wingwalls (except SE side)		
Static Drained	1.55	G1
Static Undrained	1.50	G2
Seismic (PGA 0.0845g)	1.32	G3

As per typical MTO requirements, a Factor of Safety (F.S.) of 1.5 is acceptable for long term (drained) conditions. Under the assumed seismic loading, the minimum acceptable F.S. is 1.1. The estimated F.S. values for global stability in the table above are acceptable for the proposed RSS walls.

11. EARTH CUTS

As part of construction of the longer and wider replacement structures and the proposed widening of Highway 401 on both sides, earth cuts will be required along Park Road South to lower the entire road profile up to about 1.5 m below the existing grade. In the west and east abutment areas, earth cuts will be required to widen the roadway and to accommodate the larger replacement structures. The existing structures and substructures including the pedestrian tunnel at the east abutment will be demolished and removed. It is our understanding that a staged approach will be adopted during construction, where highway widening (see section below) and the northerly and southerly portions of the new structures will be constructed first to be followed

by the middle section below the highway.

Preliminary information provided by MP indicates that road cuts along Park Road South, to the north and south of Highway 401, range from about 100 m to 120 m in length. The grade lowering would be at a maximum of 1.5 m depth below existing road grade at approximate Elevation 104.5. The highway grade will remain unchanged. The cut will largely be formed through surficial compact to dense sand and silt fill overlying native, compact to dense sand and silt till. The final road grade is at or above the groundwater table within the clayey silt till, and up to 1.5 m below the perched water level in the sand and silt till. During bulk excavation for the cut, water seepage from water-bearing sands and silts, perched water from the fills, accumulation of surface runoff and precipitation should be expected.

Temporary drainage of the cuts should be provided to maintain relatively dry and stable excavations. Surface runoff and precipitation should be diverted away from the excavations at all stages during construction. Permanent drainage will be required along the widened Park Road South. It is recommended that the water be controlled by means of permanent drains incorporated within the roadway design.

The inclination of permanent slopes may be designed for 2H : 1V, or flatter. Vegetative cover will be required on all exposed earth cut slopes to protect against surficial erosion. Reference may be made to OPSS.PROV 804. For temporary slopes, plastic sheetings or tarps may be used for covering where required.

Temporary protection (shoring) will be required for the earth cut operations as part of staged construction. Recommendations for temporary protection (shoring) are presented in Section 16 of this report.

11.1 Earth Cut Stability

Analyses of global stability were conducted for the proposed earth cuts along Park Road South as part of the road deepening and highway widening.

Selected graphical output of the stability analyses are presented on Figures G4 to G6 in Appendix G. The results are also summarized in Table 11.1 below.

Table 11.1: Computed Factors of Safety for Earth Cuts

Condition	Park Road South Deepening Cut	Factor of Safety	Figure (Appendix G)
Highway Embankment Side Slope 2H : 1V			
Static Drained	1.5 m	1.46	G4
Static Undrained	1.5 m	1.32	G5
Seismic (PGA 0.0845g)	1.5 m	1.15	G6

As per typical MTO requirements, a Factor of Safety (F.S.) of 1.3 is acceptable for short term and total stress (undrained) conditions. A F.S. of 1.5 is acceptable for long term and effective stress (drained) conditions. Under the assumed seismic loading, the minimum acceptable F.S. is 1.1. Accordingly, the analysed cases satisfy global stability requirements.

12. EMBANKMENT WIDENING

Highway 401 will be widened on the north and south sides to accommodate the new and wider underpass structures, and new WBL and EBL lanes. These widenings will be up to approximately 7 m and 14 m at the south (EBL) and north (WBL) sides, respectively. On the north (WBL) side, the required new fill will up to a maximum of about 7.5 m. On the south (EBL), new fill will largely be placed on the existing slope and the required new fill will be up to about 3 m in height. There is no grade change for the highway.

Sideslopes of the widened embankments should be designed for an inclination of 2H : 1V or flatter.

Prior to fill placement, the subgrade must be adequately prepared to receive the new fill. All vegetation, topsoil, organics, soft/loosened, wet or otherwise disturbed soils should be sub-excavated. All subgrade should be inspected and approved prior to placing fill.

Embankment widening should be carried out in accordance with OPSS.PROV 206 and OPSS.PROV 501 requirements. Materials used to construct the embankment widening should comprise granular materials, Select Subgrade Material (SSM) in compliance with OPSS.PROV 1010 or approved earth fill. Clayey earth material, especially those containing high plastic clay, is not recommended for embankment widening at this site due to potentially greater settlement after construction, difficulties in achieving the specified compaction and potential embankment stability issues. Where new embankment fill is placed against the existing embankment slopes, the existing fill slope must be benched in accordance with OPSD 208.010.

It is recommended that all exposed slope surfaces be vegetated and seeded in accordance with current MTO practice with reference to OPSS.PROV 804. Surface runoff and precipitation must be prevented from flowing perpendicularly down any slope surface. Erosion protection measures must be provided for the slopes.

12.1 Widened Embankment Stability

Analyses of global stability were conducted for the new highway embankment widening at the northwest, northeast and southwest quadrants. Due to property restrictions, embankment widening at the southeast quadrant will be done in conjunction with a long RSS wall parallel to the highway. This new wall and related global stability will be addressed in another report.

Selected graphical output of the stability analyses are presented on Figures G4 to G6 in Appendix G. The results are also summarized in Table 12.1 below.

Table 12.1: Sideslope Factors of Safety

Condition	Embankment Widening Fill	Factor of Safety	Figure (Appendix G)
Highway Embankment Side Slope 2H : 1V (except SE side)			
Static Drained	Granular fill	1.50	G7
Static Undrained	Granular fill	1.38	G8
Seismic (PGA 0.0845g)	Granular fill	1.04	G9
Seismic (PGA 0.0845g)	Granular B Type I	1.12	G10

As per typical MTO requirements, a Factor of Safety (F.S.) of 1.3 is acceptable for short term and total stress (undrained) conditions. A F.S. of 1.5 is acceptable for long term and effective stress (drained) conditions. Under the assumed seismic loading, the minimum acceptable F.S. is 1.1. Accordingly, the analysed cases satisfy global stability requirements.

12.2 Settlement

The new fill will induce immediate (elastic) settlement in the cohesionless sands and silts and re-compression of the typically very stiff to hard clayey silt till.

Northwest and Northeast Quadrants

For the WBL widening on the north side, the anticipated elastic deformation of the subgrade soils should occur relatively quickly as the new fill is placed. Post construction settlement may be considered negligible for all practical purposes.

Based on the borehole information, it is estimated that not more than 25 mm of foundation settlement will occur beneath the new widened embankments. The majority of this settlement is

expected to take place as the fill is placed and be completed by the end of construction.

Southwest Quadrant

At the southerly portion of the proposed EBL bridge where there are localized deposits of firm to stiff clayey silt/silty clay and clayey silt till, more settlement should be anticipated.

Settlement analyses were carried out to estimate the magnitude of foundation settlements under the weight of the new widening fill. A selected cross-section was used for the settlement analyses which were conducted using the commercially available software Settle3 (Version 3) developed by Rocscience Inc. The analyses were conducted using soil parameters based on correlation with index properties and soil shear strengths established during the current site investigation.

For the EBL widening on the south side, the estimated settlements due to fill placement are presented in Table 12.2 below.

Table 12.2: Subgrade Settlements under New Widening Fill at EBL

Quadrant	Location	Fill Type	Vertical Fill Height (m)	Estimated Maximum Subgrade Settlement (mm)
Southwest	EBL embankment widening	Granular (assumed)	Up to 3 m	35 to 40

It is anticipated that subgrade settlement within the widening portion of the EBL embankment would take 2 to 3 weeks to complete after the new fill reaches its design height. Consideration should be given to preloading for, say, 1 month to induce ground settlement. Preloading is a feasible means to mitigate post construction ground settlement taking into consideration the subsurface conditions and the project requirements. The preloading should be co-ordinated with the staging plans currently considered by MP. Co-ordination with footing and RSS construction will mitigate post construction footing settlement, thus enhancing foundation performance.

Suggested wording for an NSSP in this regard is included in Appendix I.

Southeast Quadrant

Similar compressible cohesive soils are present in this area within and adjacent to the embankment widening footprint. Due to property restrictions, an RSS wall greater than 100 m in length has been proposed to limit the lateral extent of the widening fill and maintain global stability of the highway embankment. Foundation investigation and design for this wall are addressed in a separate report.

12.2.1 Settlement due to Fill Compression

In addition, settlement due to embankment fill compression will occur. Well constructed fill embankments using approved granular materials typically compress between 0.5 percent of the height of fill or less. Much of this compression will occur as the fill is placed and compacted. Post construction fill compression may be considered negligible.

The settlement predictions in this report have been carried out based on a field and laboratory program, and on assumptions based on our experience with other embankments founded on similar soils. Notwithstanding the care taken in predicting the embankment performance, the settlement values observed in the field could vary from the predictions. This is due to the degree of variability of the soil properties along the embankment alignment. Therefore, the results of the settlement analyses should be used to assess the most likely performance of the embankments.

12.2.2 Impact on Highway

The currently applicable MTO criteria for post construction settlement for freeway embankment widening (July 2010) are applicable with the limit stated below.

- The maximum limit during pavement design life for freeway widening is a total settlement of 50 mm and a differential settlement rate of 200 : 1.

The settlement across the widened embankment should transition uniformly from the widening point (existing highway embankment rounding) to the new embankment rounding such that surface drainage is not impeded.

The analysis results indicate that the above MTO settlement criteria for embankment widening would be met.

Depending on the location, it is estimated that the embankment widening fill could result in approximately 10 to 15 mm settlement of the adjacent existing embankment fill. As such, the existing pavement may require asphalt resurfacing after the new fill is placed.

12.3 Embankment Construction

Prior to fill placement, the subgrade must be adequately prepared to receive the new fill. All vegetation, topsoil, organics, soft/loosened, wet or otherwise disturbed soils should be sub-excavated. All subgrade should be inspected and approved prior to placing fill.

Embankment widening should be carried out in accordance with OPSS.PROV 206 and OPSS.PROV 501 requirements. The existing embankments are comprised of predominantly sand and silt materials. Materials used to construct the embankment widenings should comprise granular materials, Select Subgrade Material (SSM) or other approved earth fill in compliance

with OPSS.PROV 1010. Clayey earth material, especially those containing high plastic clay, is not recommended for embankment widening at this site due to potentially greater settlement after construction, difficulties in achieving the specified compaction and potential embankment stability issues. Where new embankment fill is placed against the existing embankment slopes, the existing fill slope must be benched in accordance with OPSD 208.010.

It is recommended that all exposed slope surfaces be vegetated and seeded in accordance with current MTO practice with reference to OPSS.PROV 804. Surface runoff and precipitation must be prevented from flowing perpendicularly down any slope surface. Erosion protection measures should be provided as necessary for the slopes.

13. SEISMIC CONSIDERATIONS

In accordance with the CHBDC 2019, the selection of the seismic site class is based on the subsurface conditions encountered in the upper 30 m of the stratigraphic profile. In general, the subsurface stratigraphy at shallower depths at the site consists of a pavement structure on embankment fill (on the highway) or surficial fill (on Park Road South). Underlying the fills are surficial deposits of compact to dense sand and silt till, which overlie an extensive deposit of typically very stiff to hard, but occasionally firm to stiff, clayey silt till and clayey silt/silty clay. Interlayers of compact to very dense sand and silt till are present. The site is underlain by limestone bedrock at about 21 to 22 m depths below highway grade. The groundwater level was recorded at within 2 m depth below the existing Park Road South grade.

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

Based on the National Building Code of Canada (NBCC 2020), the peak horizontal ground acceleration (PGA), corresponding to a design earthquake having a 2 percent probability of being exceeded in 50 years (i.e. 2,475-year return period) is 0.169 g at the site. Based on the site class and the PGA, the Site Coefficient is determined to be 1.0.

The new structure is classified to have a Seismic Performance Category of 1 based on Table 4.10 of the CHBDC 2019.

Based on review of the SPT and field vane shear strength data, seismically induced liquefaction of foundation soils is not anticipated under the design earthquake.

14. WALL BACKFILL AND LATERAL EARTH PRESSURES

Backfill to the abutment walls should consist of free-draining granular material conforming to OPSS.PROV 1010 Granular A or B Type II specifications. The granular material should be placed to the extents shown in OPSD 3101.150 or OPSD 3121.150 where applicable. Compaction should be carried out in accordance with OPSS.PROV 206 and OPSS.PROV 501.

Earth pressures acting on the structure may be assumed to impose a triangular distribution governed by the characteristics of the backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC 2019 but generally are given by the expression:

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

Where:

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

The earth pressure coefficients are dependent on the material used as backfill. Recommended unfactored values are shown in Table 14.1. The at-rest coefficients should be employed for restrained walls. Active pressures should be used for any wingwalls or unrestrained walls.

Table 14.1: Lateral Earth Pressure Coefficients

Loading Condition	Earth Pressure Coefficient (K)			
	OPSS Granular A or Granular B Type II $\phi' = 35^\circ$, $\gamma' = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi' = 32^\circ$, $\gamma' = 21.2 \text{ kN/m}^3$	
	Horizontal Backfill	Sloping Backfill (2H : 1V)	Horizontal Backfill	Sloping Backfill (2H : 1V)
Active (Unrestrained Wall)	0.27	0.40	0.31	0.48
At-rest (Restrained Wall)	0.43	0.62	0.47	0.70
Passive	3.7	-	3.2	-

The parameters in the table correspond to full mobilization of active and passive earth pressures and require certain relative movements between the wall and adjacent soil to produce these conditions. The values to be used in design can be assessed from Figure C6.27 of the Commentary to the CHBDC 2019.

A compaction surcharge should be added in accordance with Clause 6.12.3 of the CHBDC 2019.

The use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) is generally preferred as it results in lower earth pressures acting on the wall.

The design of the walls must incorporate measures such as weep holes and/or subdrains, where applicable, to permit drainage of the backfill and avoid the potential build-up of hydrostatic pressures behind the walls.

15. EXCAVATION AND WATER CONTROL

All excavations must be carried out in accordance with OPSS.PROV 902 and the Occupational Health and Safety Act (OHSA). For the purposes of assessing excavation slope and temporary support requirements in compliance with the OHSA, embankment fill, compact to very dense sand and silt till are classified as a Type 3 soil above the groundwater level and a Type 4 soil below the groundwater level. The stiff to hard clayey silt till is classified as a Type 3 soil.

Earth cuts will be required for the Park Road South profile lowering which will extend to approximately Elevation 104.5 at the deepest location. Bulk excavation within the highway embankment will also be required for footing and abutment wall construction as part of the staging schedule.

For spread footing construction, sub-excavation below the new Park Road South grade will be required. The base of the footings will extend up to about 1 m below the groundwater level within the clayey silt till and up to 3 m below the perched water level in the sand and silt till. The drainage system currently in place along the existing Park Road South is anticipated to have locally drawn down the groundwater level. For permanent conditions, the lowered profile of Park Road South with its long term drainage system would further drawdown the surrounding groundwater level.

It is anticipated that the clay size particles within the subgrade soils will temporarily impede water seepage. However, flow of perched water from the embankment fill into the excavations and from water-bearing layers within the cohesive soils should be expected. In addition to effective pumping from filtered sumps and perimeter ditches, other measures of groundwater control including the use of well points at localized areas may be required in order to maintain a reasonably dry subgrade for construction. Surface runoff and precipitation must be diverted away from the excavations. All footings must be constructed in the dry as per OPSS.PROV 904.

The design of a dewatering system that will be required is the responsibility of the Contractor, and the Contract Documents must alert him to this responsibility. Filtered sumps must be properly designed to control loss of fines and ground loss.

Dewatering of all excavations should be carried out in accordance with OPSS.PROV 517, SP 517F01 Amendment to OPSS 517, November 2016 (issued July 2017), and OPSS.PROV 902 and NSSP FOUN0003. A design engineer with a minimum five years of relevant experience will be required to design and implement a dewatering system.

Selection of the method of excavation is the responsibility of the contractor and must be based on his equipment, experience and interpretation of the site conditions. It is recommended that the excavations be inspected periodically to confirm stability at all stages. Provision must be made for the handling of potential obstructions in the existing fills and native tills. Suggested wording for an NSSP in this regard is included in Appendix I.

Construction will need to be carried out in conjunction with temporary protection (shoring) which is discussed in more detail in the section below.

Hydrogeological aspects of this project are addressed by Egis separately. Egis advised that an assessment of water taking quantities has been completed and that an EASR will be required for this site.

16. TEMPORARY PROTECTION SYSTEMS

Temporary protection systems (TPS), or shoring, will be required as part of the staged construction for the new longer and wider bridges, widening of the highway and lowering of the road profile.

An item titled "Temporary Protection System" as per OPSS.PROV 539 and SP105S09 should be included in the contract documents. It is recommended that Performance Level 2 as per Clause 539.04.01.01 and the alignment of the temporary protection be specified on the contract drawings.

The selection and design of the temporary protection systems is the responsibility of the contractor. The design of such systems must incorporate traffic loading and surcharge loading due to the construction equipment and operations. It is anticipated that the protection system will need to be extended through the existing embankment fill, the underlying native compact to very dense sand and silt till, and into the cohesive stiff to hard clayey silt till. Installation of roadway protection should consider that the existing embankment fill and native tills contain oversize obstructions such as cobbles and boulders.

For conceptual planning and costing purposes, an augered soldier pile and wood lagging wall may be considered for temporary protection at this site. There may be difficulties in installing sheetpile walls due to the presence of very dense and hard soils containing cobbles and boulders. These shoring walls may be designed using the geotechnical parameters given below:

γ	=	20 kN/m ³
γ_w	=	10 kN/m ³
K_a	=	0.33 (approach fills)
	=	0.31 (native sand and silt till)
	=	0.32 (native clayey silt till, clayey silt/silty clay)
K_p	=	3.0 (approach fills)
	=	3.2 (native sand and silt till)
	=	3.1 (native clayey silt till)

It is recommended that lateral earth pressures acting on the wall be computed in accordance with the CHBDC 2019. The surcharge should include soil loadings above the top of the pile and other loadings adjacent to the wall. A properly designed and constructed soldier pile and lagging wall will be permeable and therefore water pressure acting on the retained height may be set to zero. Filter fabric should be placed behind the lagging boards to minimize migration of fines. Dewatering will need to be considered as discussed in the section above.

The actual pressure distribution acting on the shoring system is a function of the construction sequence and the relative flexibility of the wall, and these factors must be considered when designing the shoring system. Given the retained height and the permissible ground movements, the TPS will likely have to be supported by struts, internal bracings and anchors where space permits. All shoring systems should be designed by a Professional Engineer experienced in such designs.

Consideration should be given to leaving in place the TPS that is immediately adjacent to the bridge footings. It is recommended that the TPS be decommissioned by cutting to at least 1.2 m below the final grade as per OPSS.PROV 539 requirements. Decommissioning procedures should be to minimize the risks of disturbance and damage to the finished works and the bridge. Any other TPS installed in close proximity to permanent works including buried utilities should also be left in place and similarly decommissioned. Suggested wording for an NSSP is included in Appendix I.

17. ADJACENT STRUCTURES AND BURIED UTILITIES

An existing watermain running below Park Road South will be realigned prior to the construction of the new bridges, widening of the highway and lowering of the road profile.

Based on the latest design information, the new 400 mm CPP watermain will be located in the



order of 15 m east of the new east abutment, near Station 11+800, and generally parallel to Park Road South. The casing for housing the watermain will be a 1,200 mm diameter reinforced concrete pipe designed to be jacked in place in conjunction with MTBM trenchless operation. The casing will run in a north-south orientation under the Highway 401 EBL and WBL embankments with an invert level at about Elevation 100. Launch and exit shafts will be located off the highway embankment at the northerly and southerly limits of the trenchless section of the watermain. Beyond the shafts, the remainder of the new watermain will be installed in trenched excavations using cut and cover methods. The crown cover above the casing will be about 10 m below the Highway 401 grade.

This watermain realignment will be carried out as an advanced contract prior to other aspects of this Park Road South project. It is understood that designers of the new bridge, highways/roadways and the watermain have co-ordinated to confirm that the anticipated staged construction will safely and effectively accommodate all aspects of the proposed works without adversely impacting each other. In particular, construction of the new east abutment footings will involve a relatively large excavation under Highway 401 and the staging has taken into consideration the presence of the new watermain. It is also our understanding that the design of the casing/watermain has taken into consideration the additional loading that will be imposed by the RSS wall and the highway widening fill at the southeast quadrant.

The potential presence of other underground utilities at the site should be confirmed prior to construction. It is recommended that the exact locations and elevations of any utilities be established by the designer, and compared with the extent of the potential work zones related to the proposed replacement bridges, retaining walls, road cuts, new fills and associated works.

Protection and/or relocation of utilities, if necessary, should be implemented. Underground utilities should not be undermined or damaged during new foundation construction and fill placement.

18. SOIL CORROSION POTENTIAL

The results of corrosivity and sulphate analytical tests conducted on selected soil samples are included in Appendix C. Based on the test results, the following statements can be made:

- The degree of exposure for sulphate attack on concrete from the native sand and silt till is considered low to moderate given the measured water soluble sulphate contents in the soil samples and the slightly alkaline pH values.
- The potential for corrosion on metal ranged from mildly to very corrosive for the sand and silt till at shallow depths.

- The effects of road de-icing salts should also be considered when selecting the class of concrete and corrosion mitigation measures.

19. CONSTRUCTION CONCERNS

Potential construction concerns include, but are not necessarily limited to, the following:

- It is anticipated that a new watermain will be installed under Highway 401 and parallel to Park Road South prior to commencement of the bridge construction, road lowering and highway widening. Its alignment may be close to the work zones and should be well documented to avoid adverse impacts during the main construction.
- Staged construction of the new bridge footings requires deep excavations in the centre portion of the Highway 401 embankment with supported temporary protection (shoring) and associated dewatering. Construction activities must avoid undermining the highway and adversely affecting its live traffic during construction.
- Staged construction of the replacement bridges, RSS walls, road cut and highway widening must be carried out in a manner that would avoid undermining the existing bridge foundations while they remain operational.
- Foundation excavations and earth cuts will be extended below the groundwater level. Seepage from perched groundwater and water-bearing layers within the glacial tills will also occur. Sump pumping, diversion of surface runoff, precipitation and other forms of temporary dewatering including localized well points may be required to maintain a reasonably dry excavation during construction. A dewatering specialist should be consulted to provide input on the required dewatering system.
- Settlement monitoring of the existing bridge foundations during new bridge construction, displacement monitoring of the temporary protection systems and adjacent highway travel lanes adjacent to the work areas are recommended during construction.
- Daily visual inspection of the highway pavement surface must be carried out in the vicinity of the construction works. If cracks form in the pavement or settlement occurs, these observations must immediately be brought to the attention of the CA for determining if further action is required.
- Confirmation that the backfill to the abutments are adequately placed and compacted to specifications.
- The forward and side approach slopes and permanent cut slopes should be inspected

after construction for surficial disturbance. Where necessary, remedial measures such as re-vegetation and/or placement of gravel sheeting may be required.

20. SIGNATURES/CLOSURE

Engineering analysis and preparation of the foundation design report were carried out by Sydney Pang, P.Eng. with the assistance of Rocio Reyna, P.Eng. This report was reviewed by P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

This report was issued before any final design or construction details had been prepared or issued. Therefore, differences may exist between the report recommendations and the final design, the contract documents, or conditions during construction. In such instances, Thurber Engineering Ltd. should be contacted immediately to address these differences. Designers and contractors undertaking or bidding the work should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for design and construction, and make their own interpretation of the data as it may affect their proposed scope of work, cost, schedules, safety, and equipment capabilities.

We trust this information meets your present needs. If you have any questions, please contact the undersigned at your convenience.

Thurber Engineering Ltd.



Rocio Reyna, P. Eng.
Associate, Senior Geotechnical Engineer



Sydney Pang, P. Eng.
Senior Associate, Senior Foundation Engineer



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

Date: **December 10, 2024**
File: **30915**

STATEMENT OF LIMITATIONS AND CONDITIONS

1. STANDARD OF CARE

This Report has been prepared in accordance with generally accepted engineering or environmental consulting practices in the applicable jurisdiction. No other warranty, expressed or implied, is intended or made.

2. COMPLETE REPORT

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

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

3. BASIS OF REPORT

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

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5. INTERPRETATION OF THE REPORT

- a) Nature and Exactness of Soil and Contaminant Description: Classification and identification of soils, rocks, geological units, contaminant materials and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgmental in nature. Comprehensive sampling and testing programs implemented with the appropriate equipment by experienced personnel may fail to locate some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and the Client and all other persons making use of such documents or records with our express written consent should be aware of this risk and the Report is delivered subject to the express condition that such risk is accepted by the Client and such other persons. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. If special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.
- b) Reliance on Provided Information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to Thurber. Thurber has relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, Thurber does not accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of misstatements, omissions, misrepresentations, or fraudulent acts of the Client or other persons providing information relied on by Thurber. Thurber is entitled to rely on such representations, information and instructions and is not required to carry out investigations to determine the truth or accuracy of such representations, information and instructions.
- c) Design Services: The Report may form part of design and construction documents for information purposes even though it may have been issued prior to final design being completed. Thurber should be retained to review final design, project plans and related documents prior to construction to confirm that they are consistent with the intent of the Report. Any differences that may exist between the Report's recommendations and the final design detailed in the contract documents should be reported to Thurber immediately so that Thurber can address potential conflicts.
- d) Construction Services: During construction Thurber should be retained to provide field reviews. Field reviews consist of performing sufficient and timely observations of encountered conditions in order to confirm and document that the site conditions do not materially differ from those interpreted conditions considered in the preparation of the report. Adequate field reviews are necessary for Thurber to provide letters of assurance, in accordance with the requirements of many regulatory authorities.

6. RELEASE OF POLLUTANTS OR HAZARDOUS SUBSTANCES

Geotechnical engineering and environmental consulting projects often have the potential to encounter pollutants or hazardous substances and the potential to cause the escape, release or dispersal of those substances. Thurber shall have no liability to the Client under any circumstances, for the escape, release or dispersal of pollutants or hazardous substances, unless such pollutants or hazardous substances have been specifically and accurately identified to Thurber by the Client prior to the commencement of Thurber's professional services.

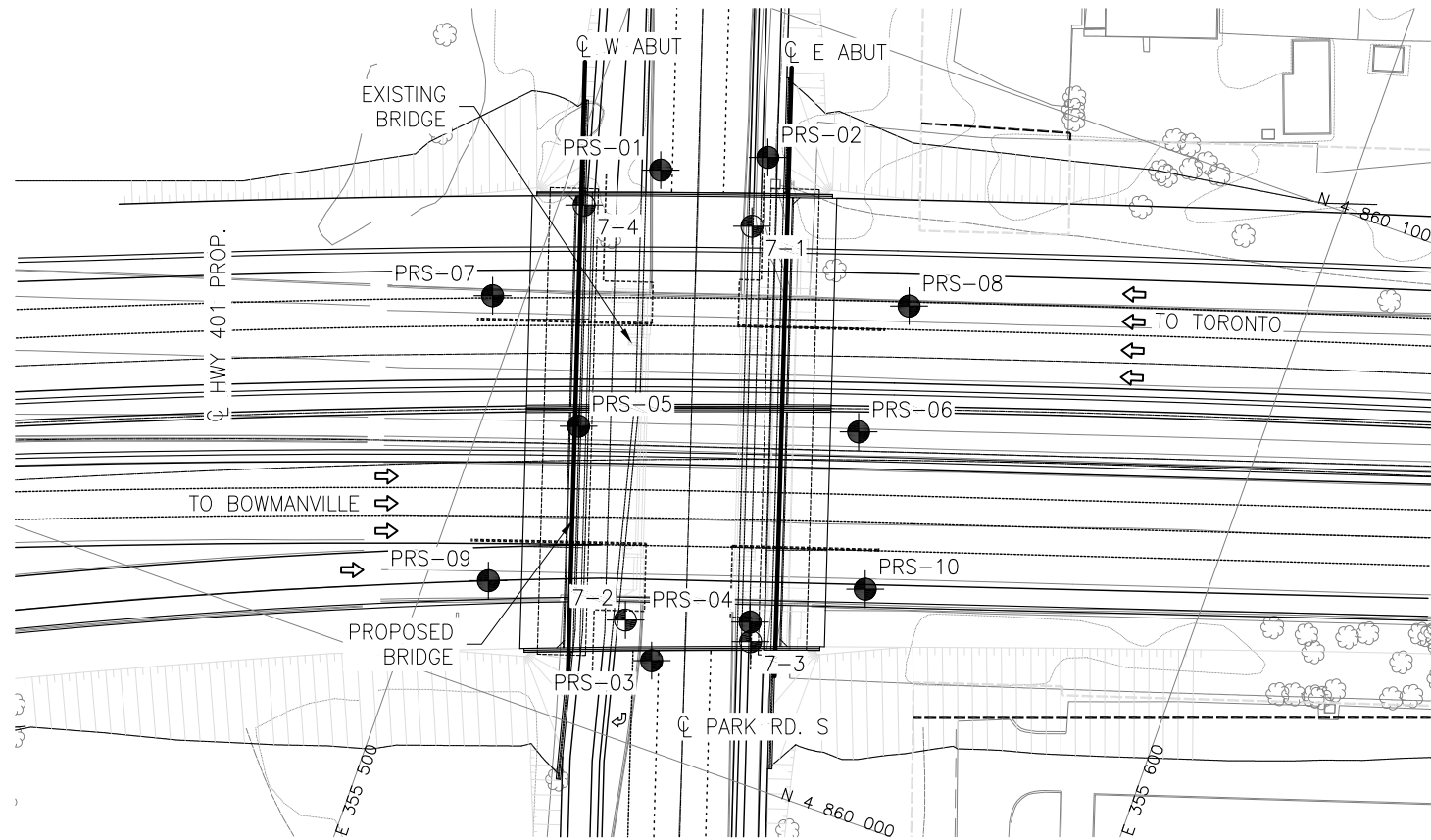
7. INDEPENDENT JUDGEMENTS OF CLIENT

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

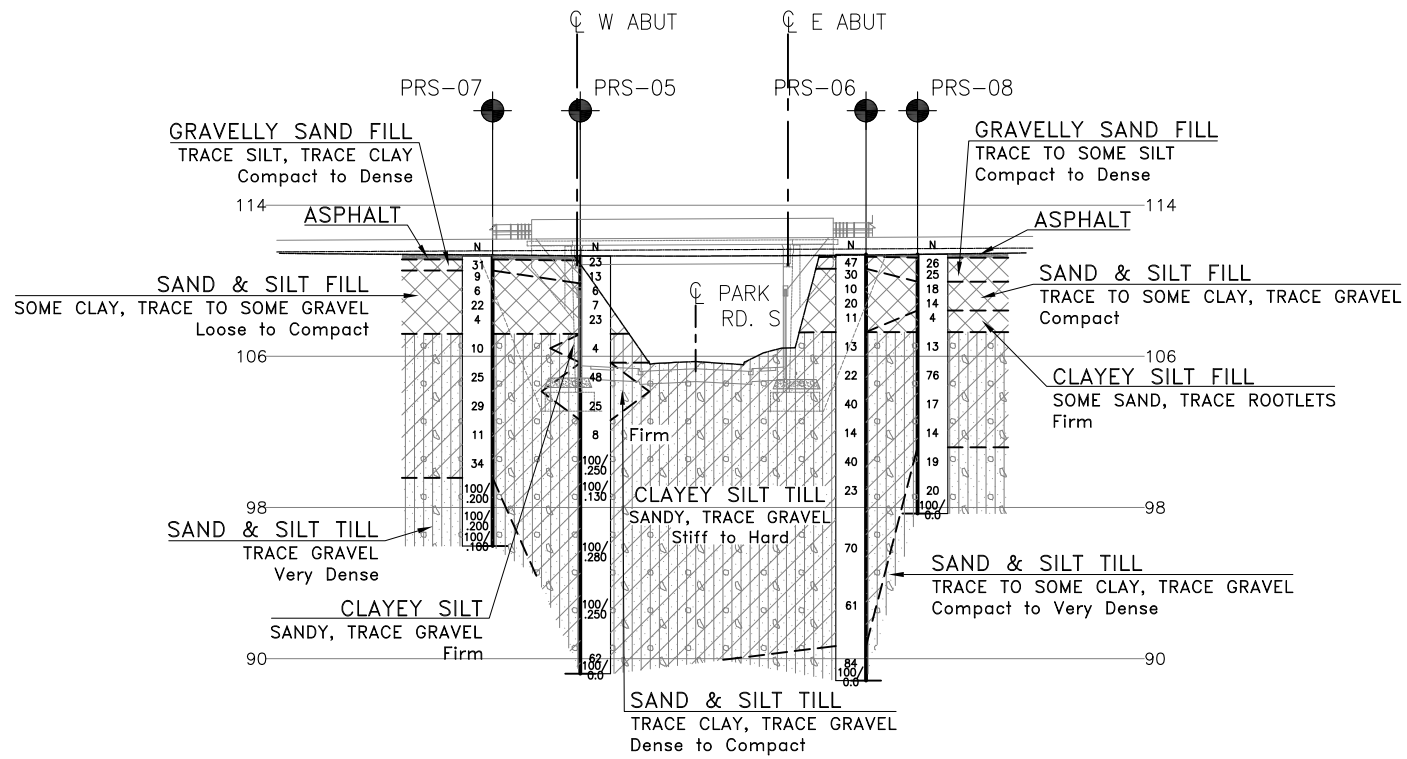
APPENDIX A

Drawings 1 and 2

Borehole Locations and Soil Strata



PLAN
SCALE 1:1000



PROFILE ALONG CL HWY 401 PROP.

SCALE 1:1000
SCALE 1:400

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



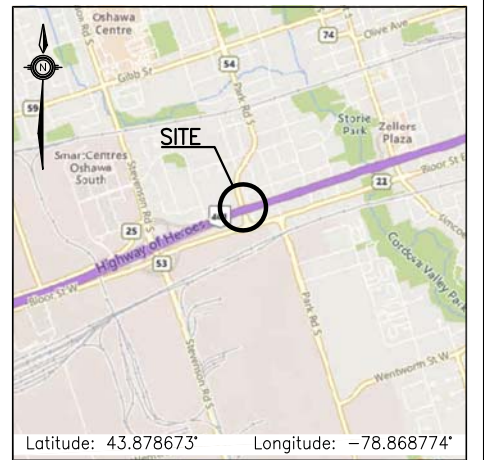
CONT No 2025-2001
GWP No 2555-17-00

HIGHWAY 401
PARK ROAD SOUTH
OVERPASS REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

egis



THURBER ENGINEERING LTD.



KEYPLAN

LEGEND

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

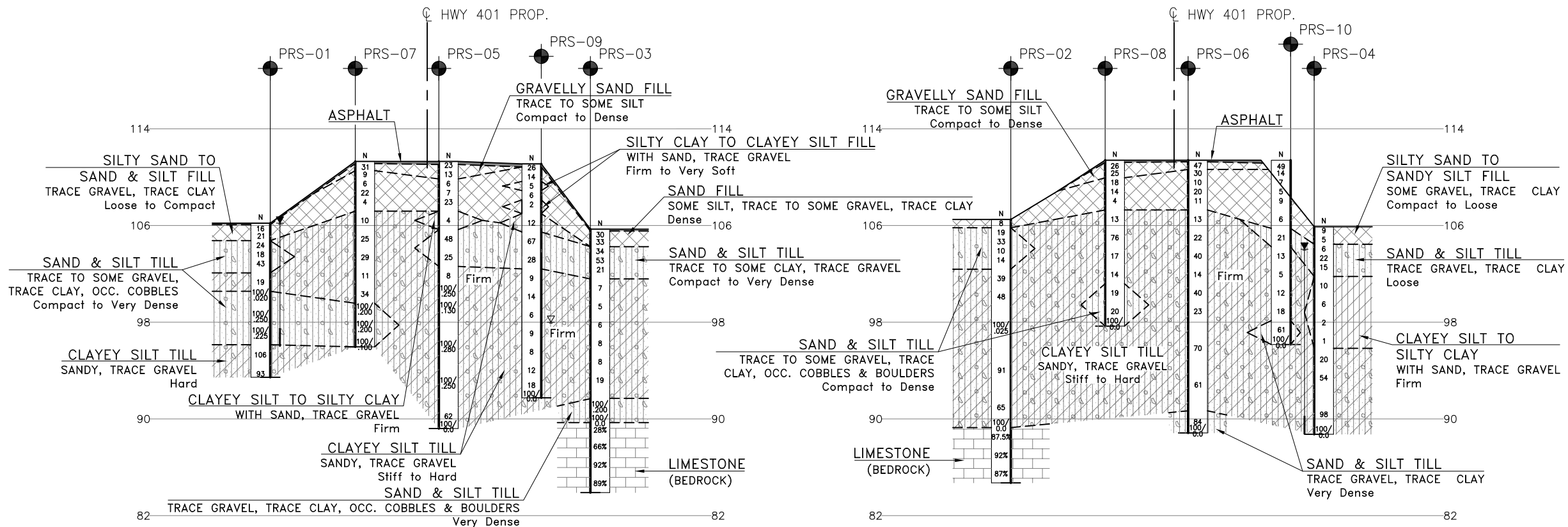
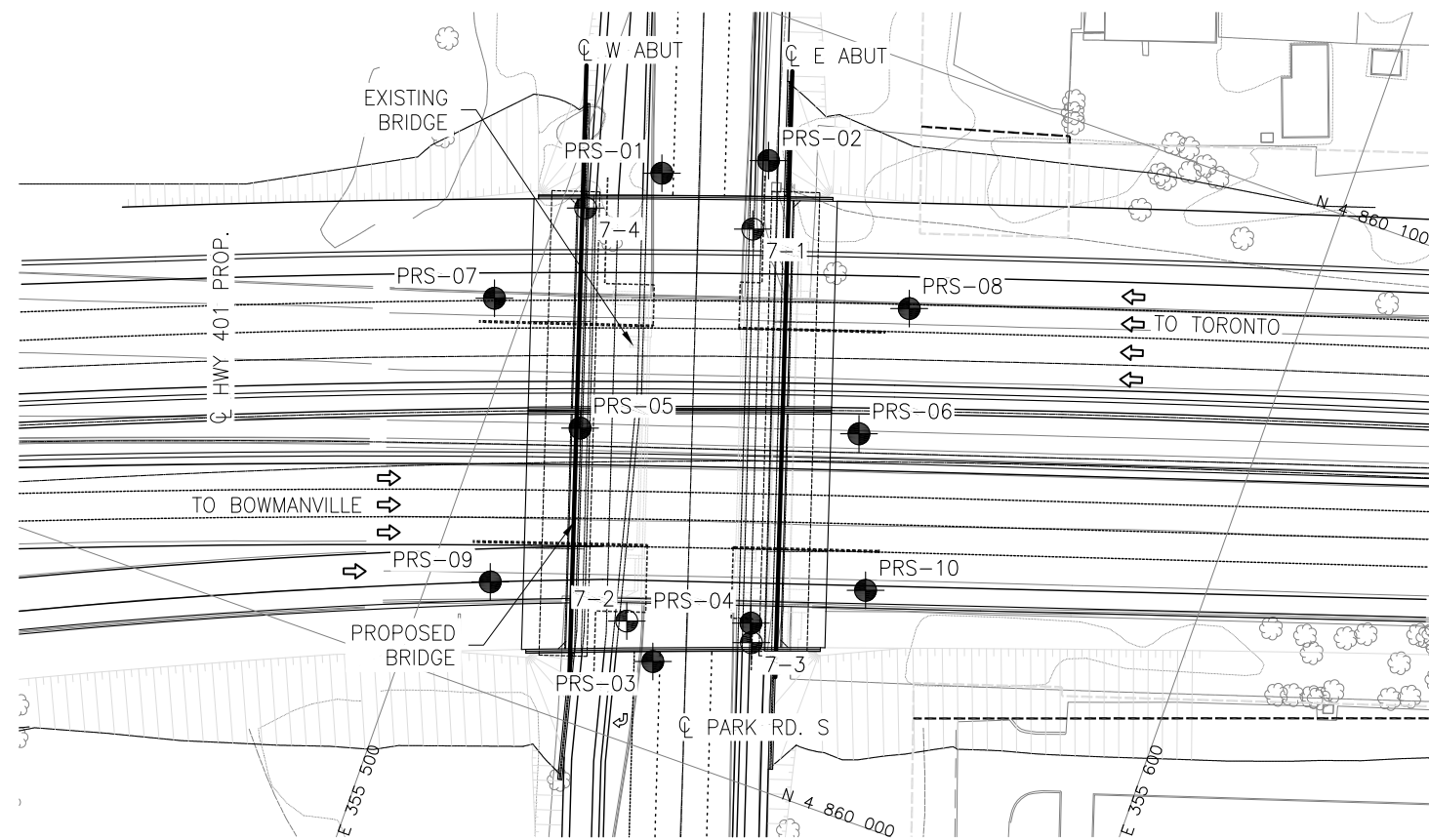
NO	ELEVATION	NORTHING	EASTING
7-1	106.4	4 860 071.4	355 525.6
7-2	106.0	4 860 015.6	355 527.3
7-3	106.0	4 860 018.5	355 544.2
7-4	107.8	4 860 066.5	355 503.3
PRS-01	106.2	4 860 074.4	355 511.4
PRS-02	106.5	4 860 080.9	355 524.5
PRS-03	105.7	4 860 011.6	355 532.5
PRS-04	105.9	4 860 021.0	355 543.2
PRS-05	111.3	4 860 038.1	355 512.6
PRS-06	111.4	4 860 050.1	355 548.4
PRS-07	111.3	4 860 050.8	355 495.7
PRS-08	111.4	4 860 068.3	355 549.2
PRS-09	111.1	4 860 014.4	355 508.1
PRS-10	111.4	4 860 030.4	355 556.4

-NOTES-

- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
- Coordinate system is MTM NAD 83 Zone 10.

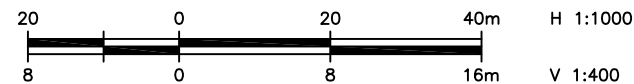
GEOCRES No. 30M15-348

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	RPR	CHK	SKP
DRAWN	AN	CHK	RPR
CODE	LOAD	DATE	DEC 2024
SITE	STRUCT	DWG	1



SECTION ALONG W ABUT

SECTION ALONG E ABUT



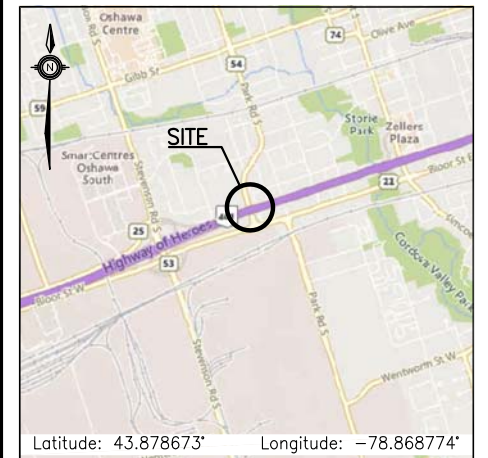
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



CONT No 2025-2001
GWP No 2555-17-00

HIGHWAY 401
PARK ROAD SOUTH
OVERPASS REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

eegis



KEYPLAN

LEGEND

●	Borehole
○	Borehole (Previous Investigation)
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
▽	Water Level Upon Completion of Drilling
┆	Water Level in Monitoring Well/Piezometer
┆	Monitoring Well/Piezometer Screen
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
7-1	106.4	4 860 071.4	355 525.6
7-2	106.0	4 860 015.6	355 527.3
7-3	106.0	4 860 018.5	355 544.2
7-4	107.8	4 860 066.5	355 503.3
PRS-01	106.2	4 860 074.4	355 511.4
PRS-02	106.5	4 860 080.9	355 524.5
PRS-03	105.7	4 860 011.6	355 532.5
PRS-04	105.9	4 860 021.0	355 543.2
PRS-05	111.3	4 860 038.1	355 512.6
PRS-06	111.4	4 860 050.1	355 548.4
PRS-07	111.3	4 860 050.8	355 495.7
PRS-08	111.4	4 860 068.3	355 549.2
PRS-09	111.1	4 860 014.4	355 508.1
PRS-10	111.4	4 860 030.4	355 556.4

-NOTES-

- The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- This drawing is for subsurface information only. Surface details and features are for conceptual illustration.
- Coordinate system is MTM NAD 83 Zone 10.

GEOCRES No. 30M15-348

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	RPR	CHK SKP	CODE
DRAWN	AN	CHK RPR	SITE
LOAD			
STRUCT			
DATE	DEC 2024		
DWG	2		

APPENDIX B

Records of Borehole Sheets
Current Investigation

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

NOTE: Hierarchy of Soil Strength Prediction

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



4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

5. LEGEND FOR RECORDS OF BOREHOLES

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

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


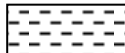



 Water Level
 Shear Strength Determination by Pocket Penetrometer

- (1) SPT 'N' Value Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.
- (2) DCPT Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.

UNIFIED SOILS CLASSIFICATION

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

EXPLANATION OF ROCK LOGGING TERMS

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

<u>DISCONTINUITY SPACING</u>		<u>STRENGTH CLASSIFICATION</u>			
Bedding	Bedding Plane Spacing	Rock Strength	Approximate Uniaxial Compressive Strength		Field Estimation of Hardness*
			(MPa)	(psi)	
Very thickly bedded	Greater than 2m	Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Thickly bedded	0.6 to 2m				
Medium bedded	0.2 to 0.6m	Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Thinly bedded	60mm to 0.2m	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Very thinly bedded	20 to 60mm				
Laminated	6 to 20mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
Thinly Laminated	Less than 6mm				

<u>TERMS</u>					
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.	Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.	Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.	Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen				
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.				

RECORD OF BOREHOLE No PRS-01

1 OF 2

METRIC

WP# 2555-17-00 LOCATION Park Road South Bridge: MTM83-10; N 4 860 074.4 E 355 511.4 ORIGINATED BY SG
DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers/PW Casing/Tri-Coring COMPILED BY AN
DATUM Geodetic DATE 2022.11.16 - 2022.11.16 LATITUDE 43.878900 LONGITUDE -78.868956 CHECKED BY CZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
106.2	GROUND SURFACE													
0.0	ASPHALT: (130mm)													
0.1	SAND and SILT, trace gravel, trace clay Compact Brown Moist (FILL)		1	SS	16		106							
			2	SS	21		105							7 46 39 8
104.8														
1.4	SAND and SILT, trace to some gravel, trace clay, occasional cobbles Compact To Dense Grey Moist (TILL)		3	SS	24		104							
			4	SS	18		103							
	Some gravel below 3.0 m		5	SS	43		102							
102.1														
4.1	Clayey SILT, with sand, trace gravel Very Stiff Brown Moist (TILL)		6	SS	19		101							8 30 39 23
100.6														
5.6	SAND and SILT, trace gravel, occasional cobbles and boulders Very Dense Grey Wet (TILL)		7	SS	100/.020		100							Split spoon damaged on possible cobbles or boulder
			8	SS	100/.250		99							
							98							1 62 37 0
			9	SS	100/.225		97							
96.2														

Continued Next Page

+³, ×³: Numbers refer to Sensitivity
20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PRS-01

2 OF 2

METRIC

WP# 2555-17-00 LOCATION Park Road South Bridge: MTM83-10; N 4 860 074.4 E 355 511.4 ORIGINATED BY SG
DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers/PW Casing/Tri-Coring COMPILED BY AN
DATUM Geodetic DATE 2022.11.16 - 2022.11.16 LATITUDE 43.878900 LONGITUDE -78.868956 CHECKED BY CZ

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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
10.0	Continued From Previous Page Clayey SILT , with sand, trace gravel Hard Grey Moist (TILL)		10	SS	106		96										
							95										
93.5			11	SS	93		94										2 39 44 15
12.7	END OF BOREHOLE AT 12.7m. WATER WAS ADDED TO THE BOREHOLE DURING DRILLING. THEREFORE WATER LEVEL WAS NOT ESTABLISHED UPON COMPLETION OF DRILLING. Monitoring Well installation consists of 50mm diameter Schedule 40 PVC pipe with a 1.5m slotted screen. WATER LEVEL READINGS DATE DEPTH(m) ELEV.(m) 2023.04.14 0.3 105.9 2023.05.18 0.0 106.2																

RECORD OF BOREHOLE No PRS-02

1 OF 3

METRIC

WP# 2555-17-00 LOCATION Park Road South Bridge: MTM83-10; N 4 860 080.9 E 355 524.5 ORIGINATED BY SG
DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers/PW and HW Casing/HQ Coring COMPILED BY AN
DATUM Geodetic DATE 2022.11.17 - 2022.11.18 LATITUDE 43.878958 LONGITUDE -78.868793 CHECKED BY CZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					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						
								20 40 60 80 100						
106.5	GROUND SURFACE													
0.0	Silty SAND , trace gravel, trace clay, occasional rootlets Loose Brown Moist (FILL)		1	SS	8									
105.8														
0.7	SAND and SILT , trace gravel, trace to some clay, occasional cobbles and boulders Compact to Dense Brown Wet (TILL)		2	SS	19									7 40 44 9
			3	SS	33									
			4	SS	10									
			5	SS	14									
102.4	Clayey SILT , with sand, trace gravel, occasional cobbles and boulders Hard Grey Wet (TILL)		6	SS	39									2 35 48 15
4.1														
			7	SS	48									High resistance to borehole advance between 6.0m and 12.0m due to possible cobbles and boulders
	Auger grinding at 7.6m													
	No recovery. Possible cobbles and boulders		8	SS	100/ .025									

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

METRIC

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No PRS-02

3 OF 3

METRIC

WP# 2555-17-00 LOCATION Park Road South Bridge: MTM83-10; N 4 860 080.9 E 355 524.5 ORIGINATED BY SG
DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers/PW and HW Casing/HQ Coring COMPILED BY AN
DATUM Geodetic DATE 2022.11.17 - 2022.11.18 LATITUDE 43.878958 LONGITUDE -78.868793 CHECKED BY CZ

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 (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)					
	Continued From Previous Page							20	40	60	80	100		W _P	W	W _L		

RECORD OF BOREHOLE No PRS-03

1 OF 3

METRIC

WP# 2555-17-00 LOCATION Park Road South Bridge: MTM83-10; N 4 860 011.6 E 355 532.5 ORIGINATED BY SG
DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers/PW and HW Casing/HQ Coring/Tri-Coring COMPILED BY AN
DATUM Geodetic DATE 2022.11.14 - 2022.11.15 LATITUDE 43.878334 LONGITUDE -78.868700 CHECKED BY CZ

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			SHEAR STRENGTH kPa				W _P	W	W _L			WATER CONTENT (%)
105.7	GROUND SURFACE							20	40	60	80	100					GR SA SI CL
0.0	ASPHALT: (150mm)							20	40	60	80	100					
0.2	SAND, some silt, trace to some gravel, trace clay Dense Brown Wet (FILL)		1	SS	30		105							o			0 79 19 2
			2	SS	33									o			
104.3																	
1.4	SAND and SILT, trace to some clay, trace gravel Compact to Very Dense Grey Moist		3	SS	34		104							o			
			4	SS	53		103							o			3 49 35 13
			5	SS	21		102							o			
							101										
101.6																	
4.1	ClayeySILT, with sand, trace gravel Firm Grey Wet (TILL)		6	SS	7		100										8 39 40 13
							99										
			7	SS	5		98							o			1 31 46 22
			8	SS	6		97										
							96										
	No recovery		9	SS	8												

Continued Next Page

+³, ×³: Numbers refer to Sensitivity
20
15
10
(%) STRAIN AT FAILURE

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

METRIC


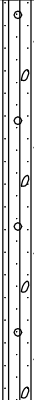




[illegible]

RECORD OF BOREHOLE No PRS-04

1 OF 2

METRIC

WP# 2555-17-00 LOCATION Park Road South Bridge: MTM83-10; N 4 860 021.0 E 355 543.2 ORIGINATED BY SG
DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers/PW Casing/Tri-Coring COMPILED BY AN
DATUM Geodetic DATE 2022.11.22 - 2022.11.22 LATITUDE 43.878417 LONGITUDE -78.868565 CHECKED BY CZ

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 (%)					GR	SA	SI	CL
								○ UNCONFINED + FIELD VANE		● QUICK TRIAXIAL × LAB VANE		w _p w w _L								
105.9	GROUND SURFACE						20	40	60	80	100									
0.0	Silty SAND , some gravel, trace clay Loose Brown Moist (FILL)		1	SS	9									○						
			2	SS	5									○					15 51 29 5	
104.5																				
1.4	SAND and SILT , trace clay, trace gravel Loose to Compact Brown Moist		3	SS	6									○						
			4	SS	22									○						
			5	SS	15									○					1 49 42 8	
101.8																				
4.1	Clayey SILT to Silty CLAY , with sand, trace gravel Stiff to Firm Grey Wet		6	SS	10									○						
			7	SS	6														5 49 38 8	
			8	SS	2														5 25 40 30	
			9	SS	1														5 41 42 12	
95.9																				
				</																

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

METRIC

SOIL PROFILE					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES	GROUND WATER CONDITIONS	ELEVATION SCALE
			NUMBER	TYPE	"N" VALUES
	Continued From Previous Page				
10.0	Clayey SILT, with sand, trace gravel Very Stiff to Hard Grey Wet (TILL)	[Strat Plot]	10	SS	20
			11	SS	54
			12	SS	98
88.7					
17.2	END OF BOREHOLE AT 17.2m UPON REFUSAL. WATER WAS ADDED TO THE BOREHOLE DURING DRILLING. THEREFORE WATER LEVEL WAS NOT ESTABLISHED UPON COMPLETION OF DRILLING. Monitoring Well installation consists of 50mm diameter Schedule 40 PVC pipe with a 1.5m slotted screen.		13	SS	100/.0

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+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No PRS-05

1 OF 3

METRIC

WP# 2555-17-00 LOCATION Park Road South Bridge: MTM83-10; N 4 860 038.1 E 355 512.6 ORIGINATED BY SG
DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers/PW Casing/Tri-Coring COMPILED BY AN
DATUM Geodetic DATE 2022.11.09 - 2022.11.11 LATITUDE 43.878574 LONGITUDE -78.868945 CHECKED BY CZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
111.3	GROUND SURFACE												
0.0	ASPHALT: (250mm)												
111.0													
0.3	Gravelly SAND , trace silt Compact Brown Moist (FILL)		1	SS	23		111						
			2	SS	13		110						
109.9													
1.4	SAND and SILT , trace gravel, trace clay Loose to Compact Brown Moist (FILL)		3	SS	6		109						
			4	SS	7		108						0 35 59 6
			5	SS	23		107						
107.2													
4.1	Clayey SILT to Silty CLAY , with sand Firm Grey Wet		6	SS	4		106						0 37 52 11
105.7													
5.6	SAND and SILT , trace clay, trace gravel Dense to Compact Grey Wet (TILL)		7	SS	48		105						
			8	SS	25		104						
							103						
102.6													
8.7	Clayey SILT , with sand, trace gravel Stiff Grey Wet (TILL)		9	SS	8		102						1 38 46 15

Continued Next Page

+³, ×³: Numbers refer to Sensitivity 20 15 10 5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PRS-05

2 OF 3

METRIC

WP# 2555-17-00 LOCATION Park Road South Bridge: MTM83-10; N 4 860 038.1 E 355 512.6 ORIGINATED BY SG
DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers/PW Casing/Tri-Coring COMPILED BY AN
DATUM Geodetic DATE 2022.11.09 - 2022.11.11 LATITUDE 43.878574 LONGITUDE -78.868945 CHECKED BY CZ

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								
	Continued From Previous Page															
	Clayey SILT , with sand, trace gravel Hard Grey Wet (TILL)		10	SS	100/ 250		101									
			11	SS	100/ .130		100									
			12	SS	100/ 280		99									
			13	SS	100/ 250		98									
							97									
							96									
							95									
							94									
							93									
							92									

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PRS-05

3 OF 3

METRIC

WP# 2555-17-00 LOCATION Park Road South Bridge: MTM83-10; N 4 860 038.1 E 355 512.6 ORIGINATED BY SG
DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers/PW Casing/Tri-Coring COMPILED BY AN
DATUM Geodetic DATE 2022.11.09 - 2022.11.11 LATITUDE 43.878574 LONGITUDE -78.868945 CHECKED BY CZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					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						
	Continued From Previous Page							20 40 60 80 100						
								○ UNCONFINED + FIELD VANE						
								● QUICK TRIAXIAL × LAB VANE						
								WATER CONTENT (%)						
								20 40 60						
													</	

RECORD OF BOREHOLE No PRS-06

1 OF 3

METRIC

WP# 2555-17-00 LOCATION Park Road South Bridge: MTM83-10; N 4 860 050.1 E 355 548.4 ORIGINATED BY SG
DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers/PW Casing/Tri-Coring COMPILED BY AN
DATUM Geodetic DATE 2022.11.01 - 2022.11.02 LATITUDE 43.878679 LONGITUDE -78.868498 CHECKED BY CZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				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 (%) W _P W W _L				
111.4	GROUND SURFACE							20	40	60	80	100						
0.0	ASPHALT: (130mm)																	
0.1	Gravelly SAND, some silt		1	SS	47		111											
110.6	Dense																	
	Brown																	
	Moist																	
0.8	(FILL)																	
	SAND and SILT, trace to some clay, trace gravel		2	SS	30		110											
	Compact																	
	Brown																	
	Moist																	
	(FILL)		3	SS	10													
			4	SS	20		109											
			5	SS	11		108											
107.3	Clayey SILT, with sand, trace gravel																	
4.1	Stiff to Hard						107											
	Grey		6	SS	13													
	Wet																	
	(TILL)																	

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

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RECORD OF BOREHOLE No PRS-06

3 OF 3

METRIC

WP# 2555-17-00 LOCATION Park Road South Bridge: MTM83-10; N 4 860 050.1 E 355 548.4 ORIGINATED BY SG
DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers/PW Casing/Tri-Coring COMPILED BY AN
DATUM Geodetic DATE 2022.11.01 - 2022.11.02 LATITUDE 43.878679 LONGITUDE -78.868498 CHECKED BY CZ

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								
	Continued From Previous Page							20	40	60	80	100				
90.7	Clayey SILT , with sand, trace gravel Hard Grey Moist (TILL)						91									
20.7	SAND and SILT , trace gravel, trace clay Very Dense Grey Moist (TILL)		14	SS	84		90						o			
88.8																
22.6	END OF BOREHOLE AT 22.6m UPON REFUSAL. WATER WAS ADDED TO THE BOREHOLE DURING DRILLING. THEREFORE WATER LEVEL WAS NOT ESTABLISHED UPON COMPLETION OF DRILLING. BACKFILLED WITH BENTONITE HOLEPLUG TO 0.3m, THEN CONCRETE TO 0.2m, THEN ASPHALT TO GROUND SURFACE.		15	SS	100/ .0											

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No PRS-07

2 OF 2

METRIC

WP# 2555-17-00 LOCATION Park Road South Bridge: MTM83-10; N 4 860 050.8 E 355 495.7 ORIGINATED BY SG
DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
DATUM Geodetic DATE 2022.10.28 - 2022.10.28 LATITUDE 43.878689 LONGITUDE -78.869154 CHECKED BY CZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				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					
								20 40 60 80 100					
Continued From Previous Page							<div><div></div><div></div><div></div></div> <div><div></div><div></div><div></div></div> <div><div></div><div></div><div></div></div> <div><div></div><div></div><div></div></div> <div><div></div><div></div><div></div></div> <div><div></div><div></div><div></div></div> <div><div></div><div></div><div></div></div> <div><div></div><div></div><div></div></div> <div><div></div><div></div><div></div></div> <div><div></div><div></div><div></div></div> <div><div></div><div></div><div></div></div> <div><div></div><div></div><div></div></div> <div><div></div><div></div><div></div></div> <div><div></div><div></div><div></div></div> <div><div></div><div></div><div></div></div> <div><div></div><div></div><div></div></div> <div><div></div><div></div><div></div></div> <div><div></div><div></div><div></div></div> <div><div></div><div></div><div></div></div> <div><div></div><div></div><div></div></div> <div><div></div><div></div><div></div></div> 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RECORD OF BOREHOLE No PRS-08

1 OF 2

METRIC

WP# 2555-17-00 LOCATION Park Road South Bridge: MTM83-10; N 4 860 068.3 E 355 549.2 ORIGINATED BY SG
DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
DATUM Geodetic DATE 2022.10.27 - 2022.10.28 LATITUDE 43.878843 LONGITUDE -78.868487 CHECKED BY CZ

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									
111.4	GROUND SURFACE							20	40	60	80	100					
0.0	ASPHALT: (178mm)							20	40	60	80	100					
0.2	GravellySAND, trace to some silt Compact Brown Moist (FILL)		1	SS	26	▽	111										
			2	SS	25		110										
110.0																	
1.4	SAND and SILT, trace to some clay, trace gravel Compact Brown Moist (FILL)	3	SS	18	109												
		4	SS	14													
108.4																	
3.0	ClayeySILT, some sand, occasional rootlets Firm Dark Brown Wet (FILL)	5	SS	4	108												
107.3	ClayeySILT, with sand, trace gravel Stiff to Very Stiff Grey Wet (TILL)		6	SS	13		107										
105.8							106										
5.6																	
	Hard		7	SS	76	105											
104.2			8	SS	17	104											
7.2			9	SS	14	103											
							102										

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PRS-08

2 OF 2

METRIC

WP# 2555-17-00 LOCATION Park Road South Bridge: MTM83-10; N 4 860 068.3 E 355 549.2 ORIGINATED BY SG
 DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2022.10.27 - 2022.10.28 LATITUDE 43.878843 LONGITUDE -78.868487 CHECKED BY CZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				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						
								20 40 60 80 100						
Continued From Previous Page							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				PLASTIC LIMIT W P NATURAL MOISTURE CONTENT W LIQUID LIMIT W L			
101.2														
10.2	SAND and SILT, trace clay, trace gravel Compact Grey Wet (TILL)		10	SS	19		101							
							100							
				11	SS	20		99						
								98						
97.7			12	SS	100/.0									
13.7	END OF BOREHOLE AT 13.7m UPON REFUSAL ON POSSIBLE COBBLES, BOULDERS OR BEDROCK. WATER LEVEL AT 3.0m UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.3m, THEN CONCRETE TO 0.2m, THEN ASPHALT TO GROUND SURFACE.													

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RECORD OF BOREHOLE No PRS-09

1 OF 3

METRIC

WP# 2555-17-00 LOCATION Park Road South Bridge; MTM83-10: N 4 860 014.4 E 355 508.1 ORIGINATED BY SG
DIST Central HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SN
DATUM Geodetic DATE 2023.11.19 - 2023.11.20 LATITUDE 43.878360 LONGITUDE -78.869004 CHECKED BY RPR

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 (%) 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 (%) W _P W W _L				
111.1	GROUND SURFACE							20	40	60	80	100				
0.0	ASPHALT: (250mm)						111									
110.8																
0.3	SAND and SILT, trace to some gravel, trace clay Compact Brown Moist (FILL)		1	SS	26									○		
			2	SS	14		110							○		
109.7																
1.4	Silty CLAY, with sand, trace gravel Firm Brown to Grey Moist (FILL)		3	SS	5									○		1 47 40 12
108.9							109									
2.2	SAND and SILT, trace to some gravel, trace clay Compact Brown Moist (FILL)		4	SS	6									○		
108.1																
3.0	Clayey SILT, with sand, trace gravel Very Soft Brown to Grey Wet (FILL)		5	SS	2		108							○		
107.0							107									
4.1	Clayey SILT, with sand, trace gravel Stiff Brown to Grey Moist (TILL)		6	SS	12									○		1 45 40 14
							106									
105.5																
5.6	SAND and SILT, trace gravel, trace clay Very Dense to Compact Brown to Grey Moist (TILL)		7	SS	67		105							○		
							104									
	Occasional oxidized staining		8	SS	28		103							○		
102.4																
8.7	Clayey SILT, with sand, trace gravel Stiff Grey Wet (TILL)		9	SS	9		102							○		

Continued Next Page

+³, ×³: Numbers refer to Sensitivity
20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PRS-09

2 OF 3

METRIC

WP# 2555-17-00 LOCATION Park Road South Bridge; MTM83-10: N 4 860 014.4 E 355 508.1 ORIGINATED BY SG
DIST Central HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SN
DATUM Geodetic DATE 2023.11.19 - 2023.11.20 LATITUDE 43.878360 LONGITUDE -78.869004 CHECKED BY RPR

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 (%)			GR	SA	SI	CL
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE											
	Continued From Previous Page							20	40	60	80	100										
	Clayey SILT , with sand, trace gravel Stiff Grey Wet (TILL)						101															
			10	SS	14		100											4	36 41 19			
99.4																						
11.7	Firm		11	SS	6		99											2	25 44 29			
97.8							98				4.4 +											
13.3	Stiff		12	SS	9		97															
							96															
			13	SS	8		95											3	42 41 14			
							94															
			14	SS	12		93															
							92															
	Very Stiff		15	SS	18													9	51 32 8			
	Hard																					
91.7			16	SS	100/0.0																	
19.4	END OF BOREHOLE AT 19.4m UPON REFUSAL. WATER LEVEL IN OPEN BOREHOLE AT 13.1m UPON																					

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PRS-09

3 OF 3

METRIC

WP# 2555-17-00 LOCATION Park Road South Bridge; MTM83-10: N 4 860 014.4 E 355 508.1 ORIGINATED BY SG
DIST Central HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SN
DATUM Geodetic DATE 2023.11.19 - 2023.11.20 LATITUDE 43.878360 LONGITUDE -78.869004 CHECKED BY RPR

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									
	Continued From Previous Page																
	COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND AUGER CUTTINGS TO 0.3m, THEN ASPHALT PATCH TO SURFACE.																

RECORD OF BOREHOLE No PRS-10

1 OF 2

METRIC

WP# 2555-17-00 LOCATION Park Road South Bridge: MTM83-10; N 4 860 030.4 E 355 556.4 ORIGINATED BY SG
DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
DATUM Geodetic DATE 2022.11.08 - 2022.11.09 LATITUDE 43.878501 LONGITUDE -78.868401 CHECKED BY CZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				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					
111.4	GROUND SURFACE												
0.0	ASPHALT: (250mm)												
111.1													
0.3	Gravelly SAND , trace silt Dense Brown Moist (FILL)		1	SS	49		111						
110.6													
0.8	Silty SAND , some gravel, trace clay Compact to Loose Brown Moist (FILL)		2	SS	14		110						10 53 29 8
			3	SS	7								
109.2													
2.2	Sandy SILT , trace gravel, trace clay Loose Brown Moist (FILL)		4	SS	5		109						
			5	SS	9		108						
107.3													
4.1	Clayey SILT , some sand, trace gravel, occasional organics Firm Dark Brown Wet (FILL)		6	SS	6		107						
							106						
105.8													
5.6	SAND and SILT , trace clay, trace gravel Compact Grey Wet (TILL)		7	SS	21		105						2 48 44 6
104.2													
7.2	Clayey SILT , with sand, trace to some gravel Stiff to Firm Grey Wet (TILL)		8	SS	13		104						
							103						
			9	SS	5		102						1 35 45 19

Continued Next Page

+³, ×³: Numbers refer to Sensitivity 20 15 10 5 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No PRS-10

2 OF 2

METRIC

WP# 2555-17-00 LOCATION Park Road South Bridge: MTM83-10; N 4 860 030.4 E 355 556.4 ORIGINATED BY SG
DIST HWY 401 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
DATUM Geodetic DATE 2022.11.08 - 2022.11.09 LATITUDE 43.878501 LONGITUDE -78.868401 CHECKED BY CZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE LIMIT LIQUID CONTENT LIMIT			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				WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								
	Continued From Previous Page						20	40	60	80	100	W _P	W	W _L		
98.1	Clayey SILT , with sand, trace to some gravel Stiff to Very Stiff Grey Wet (TILL)		10	SS	12											13 38 39 10
13.3	SAND and SILT , trace clay, trace gravel Very Dense Grey Moist (TILL)		12	SS	61											
96.2			13	SS	100/.0											
15.2	END OF BOREHOLE AT 15.2m. WATER LEVEL AT 2.7m UPON COMPLETION. BACKFILLED WITH BENTONITE HOLEPLUG TO 0.3m, THEN CONCRETE TO 0.2m, THEN ASPHALT TO GROUND SURFACE.															

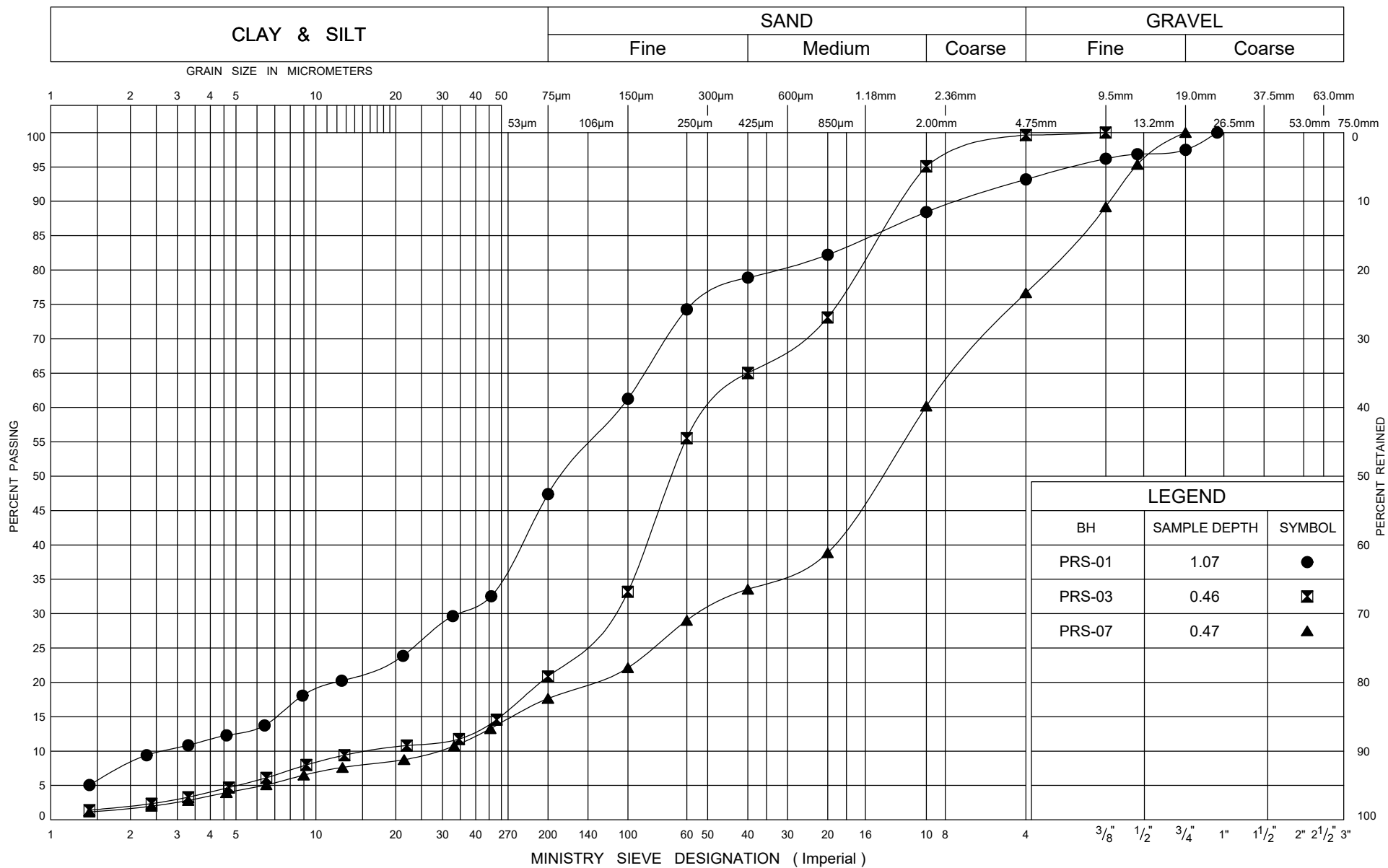
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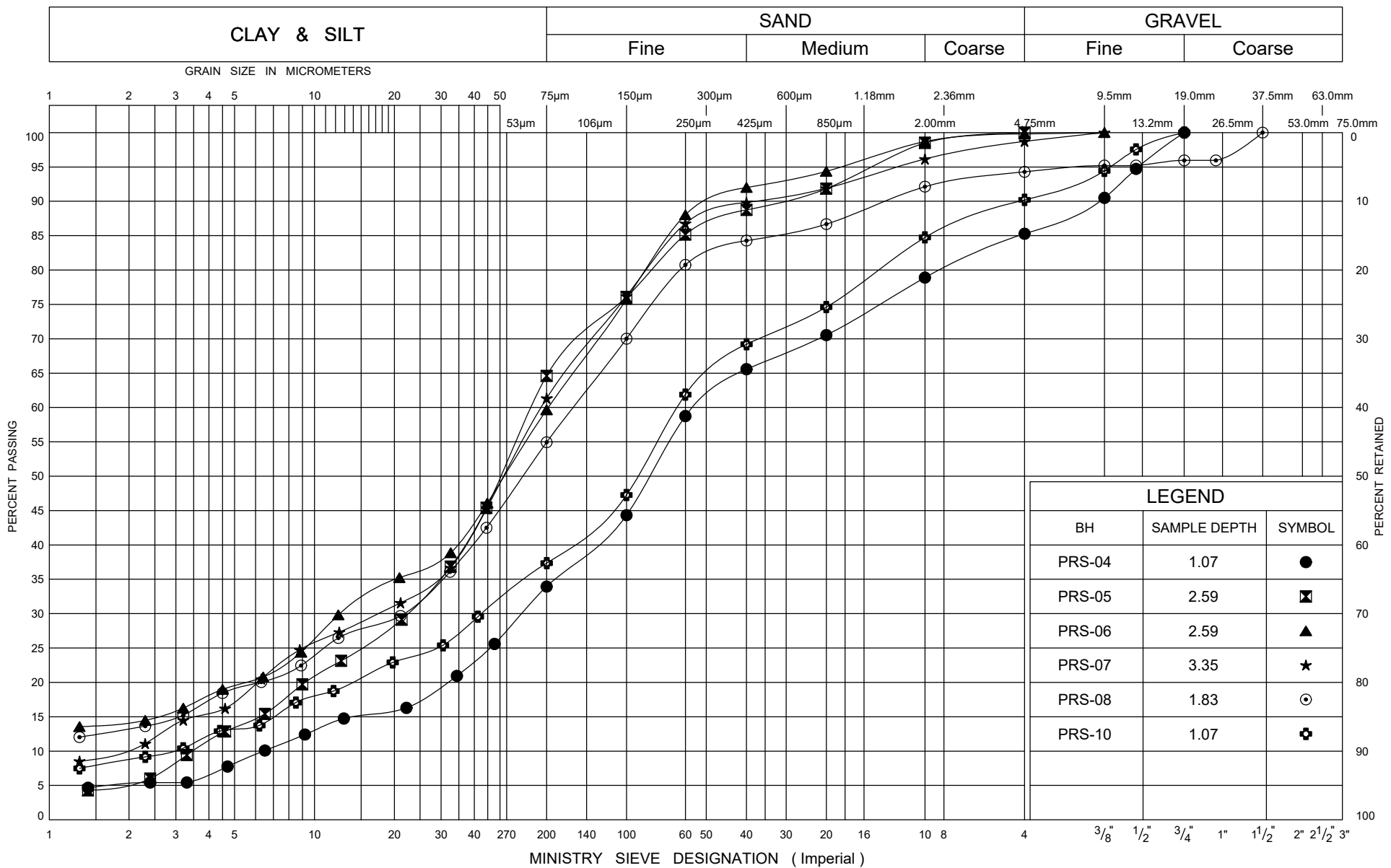
APPENDIX C

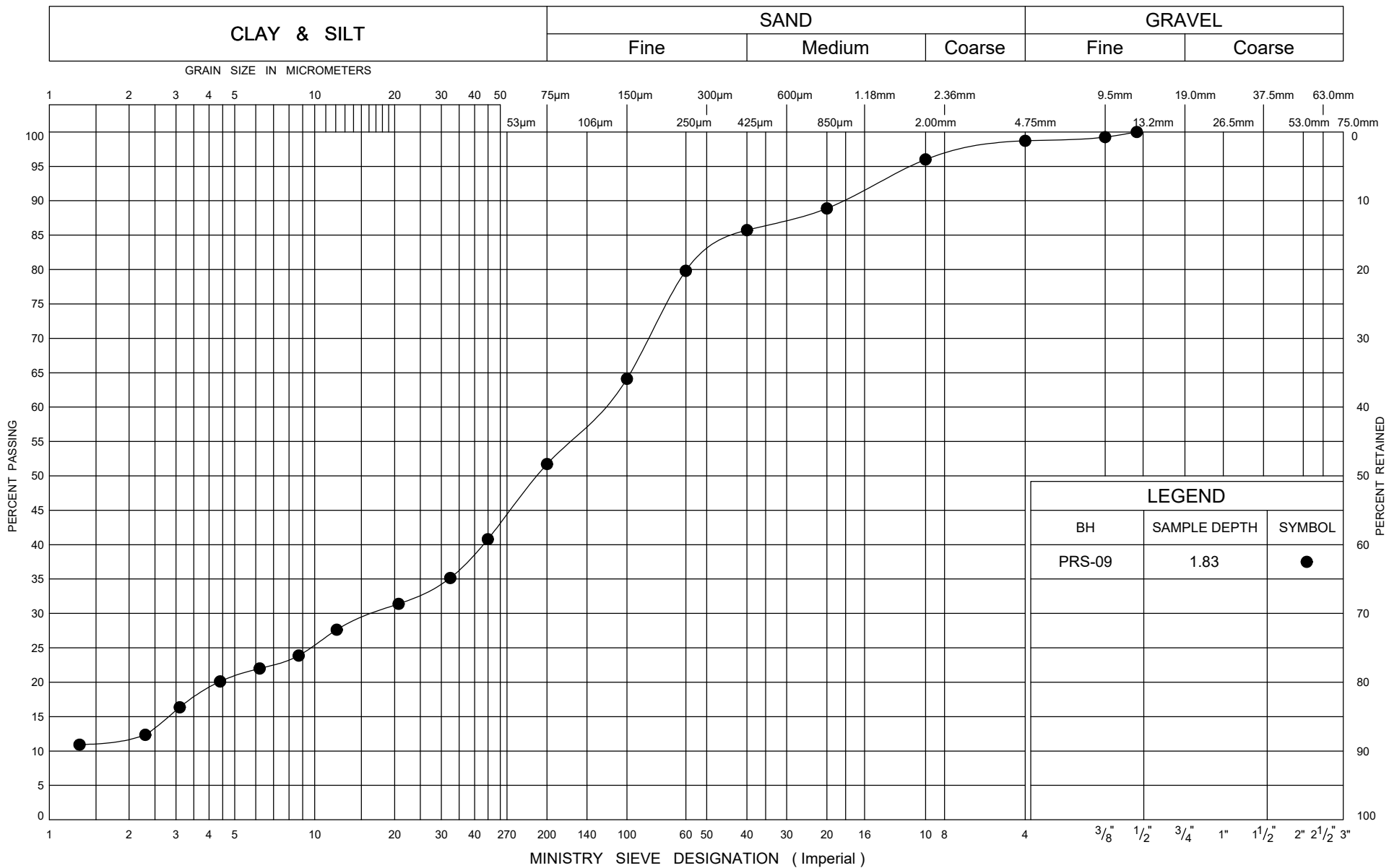
Geotechnical and Analytical Laboratory Test Results

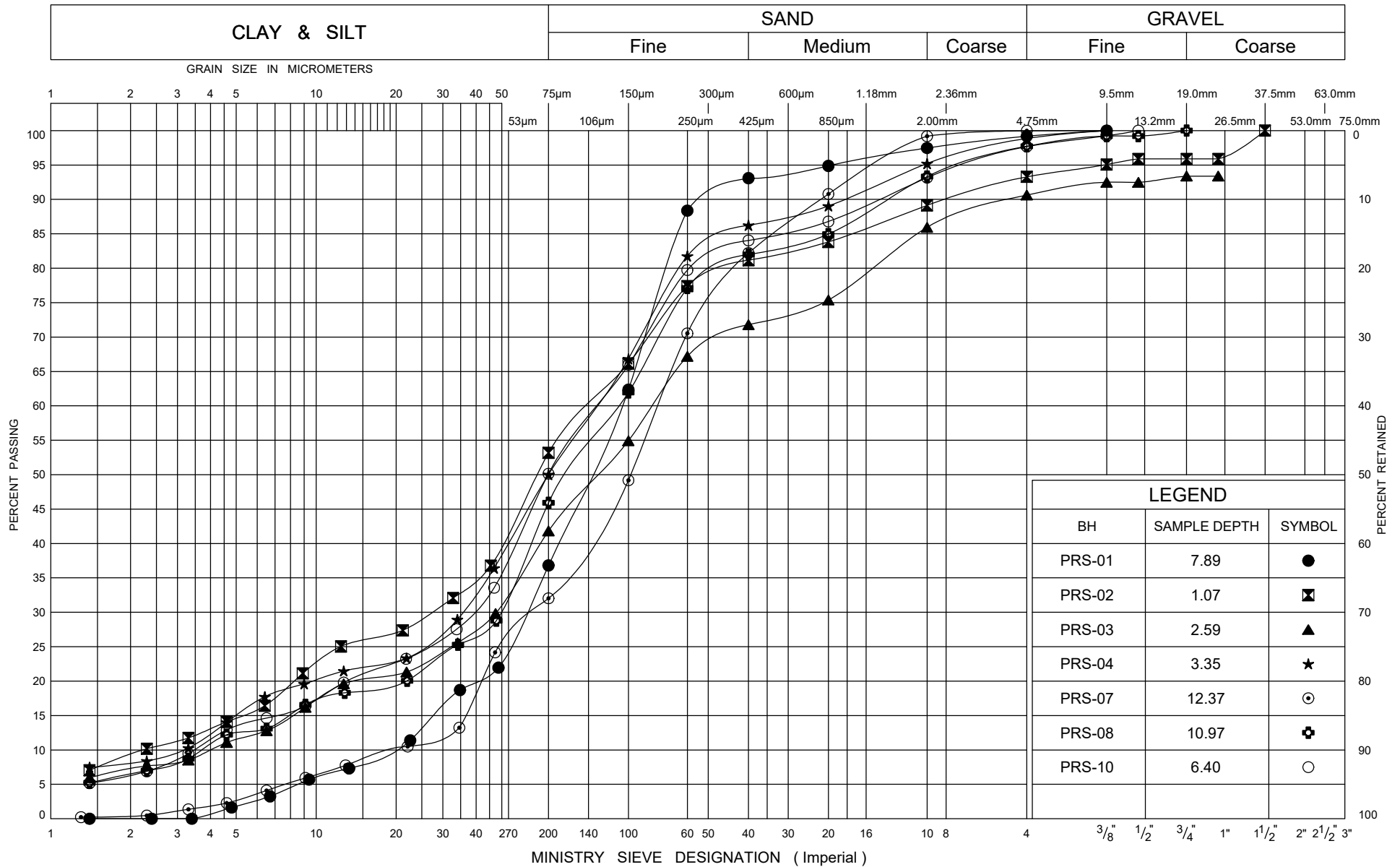
Point Load Test Results

Rock Core Photographs









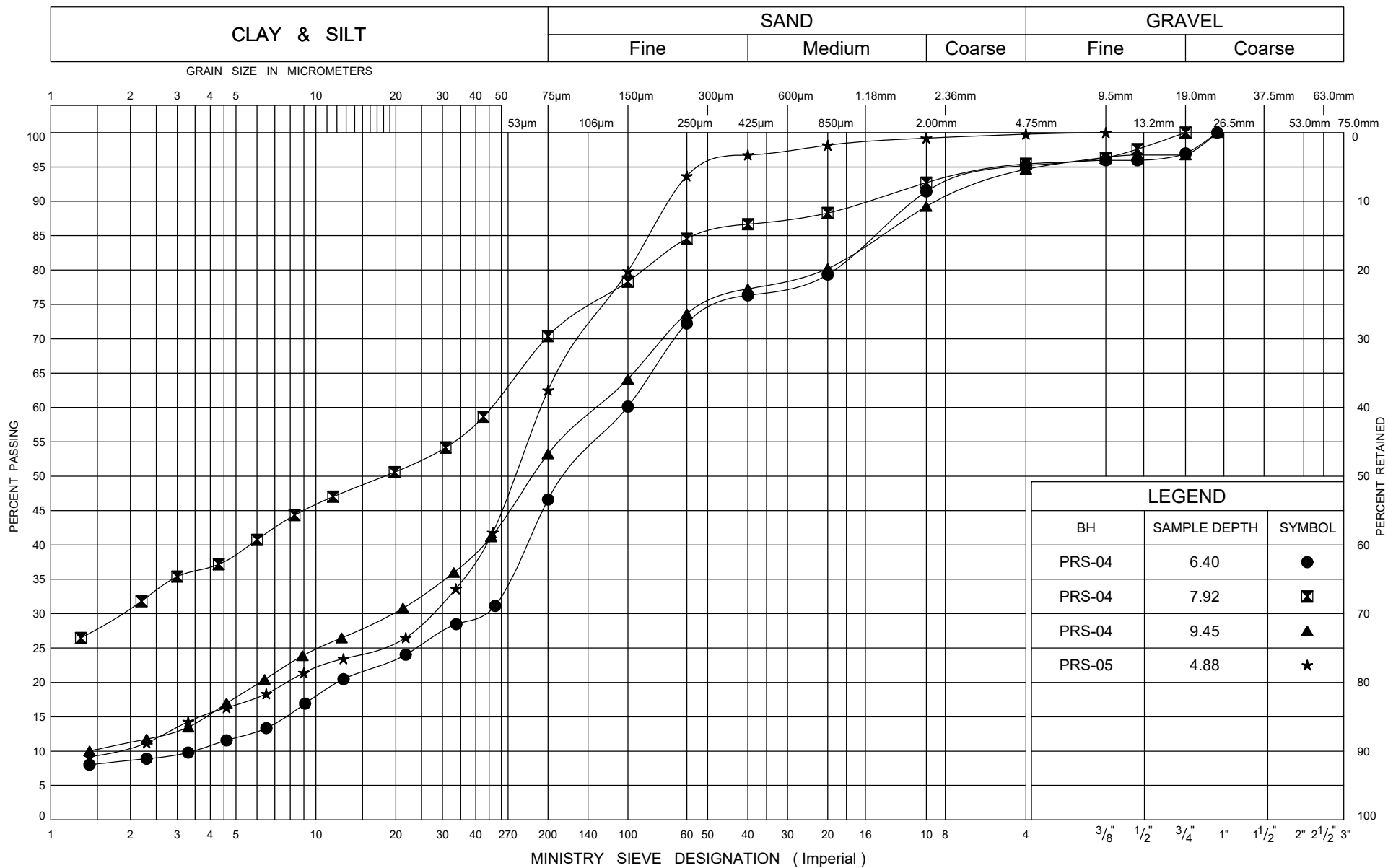
Ministry of
Transportation

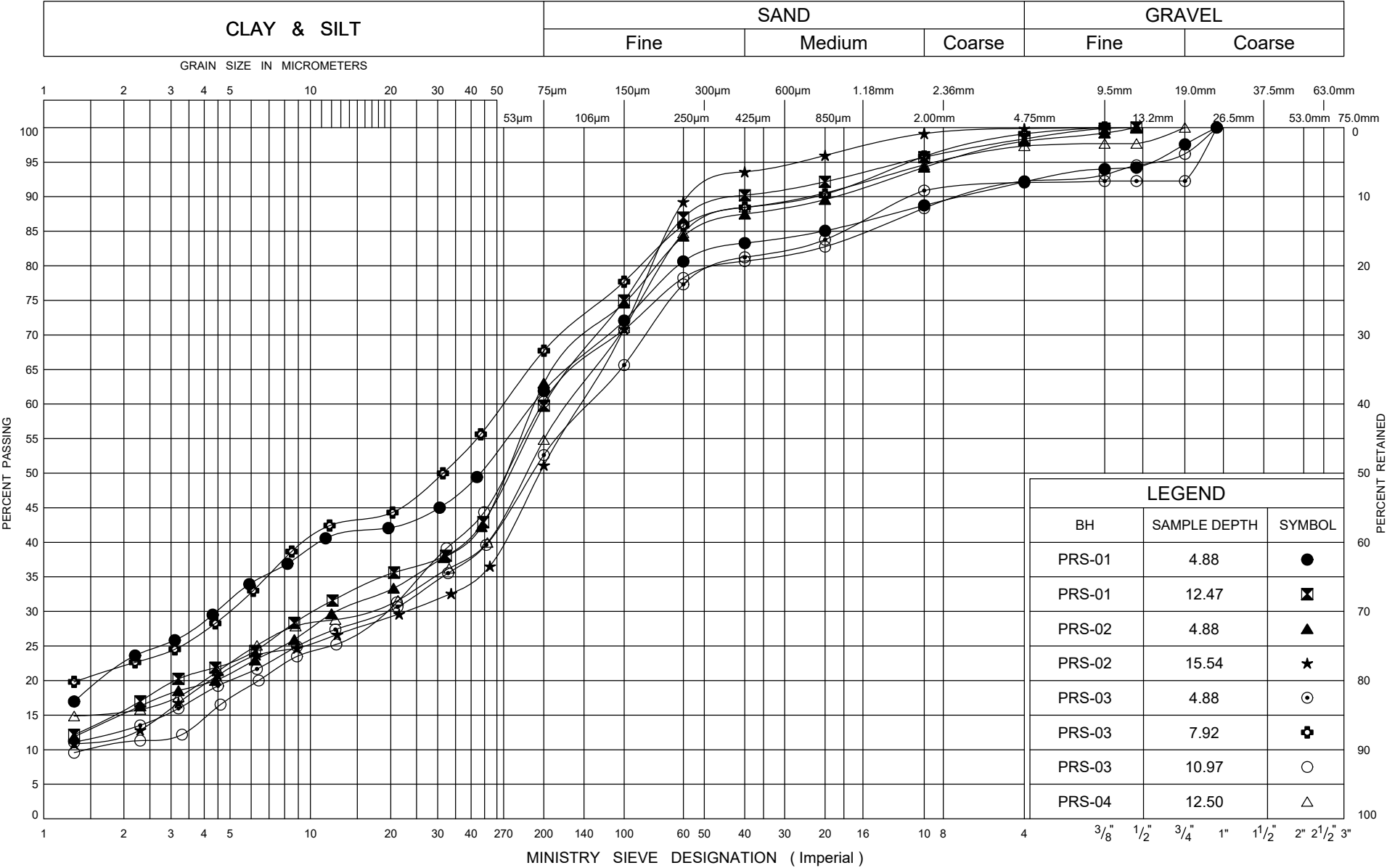
GRAIN SIZE DISTRIBUTION SAND and SILT TILL

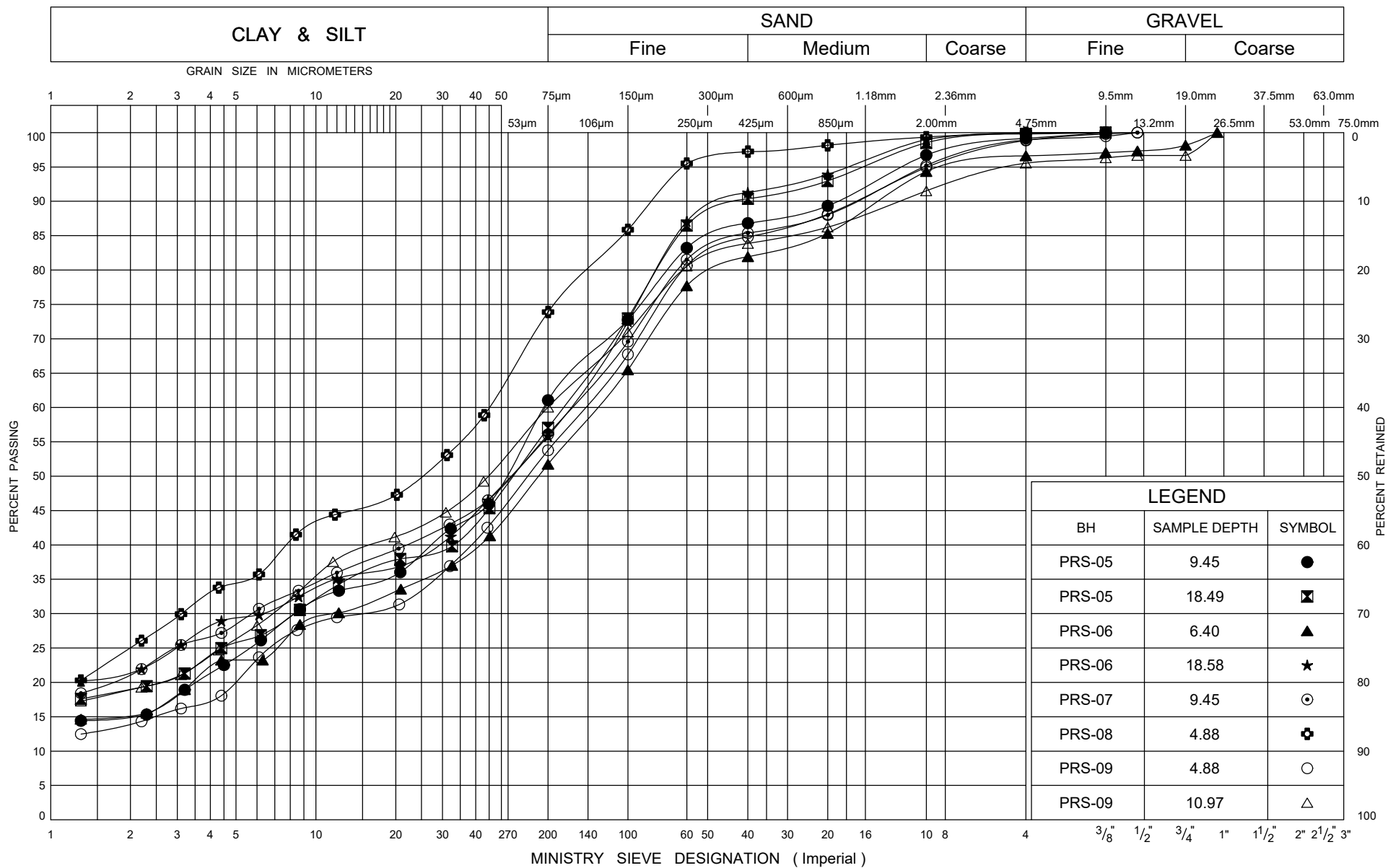
FIG No C4

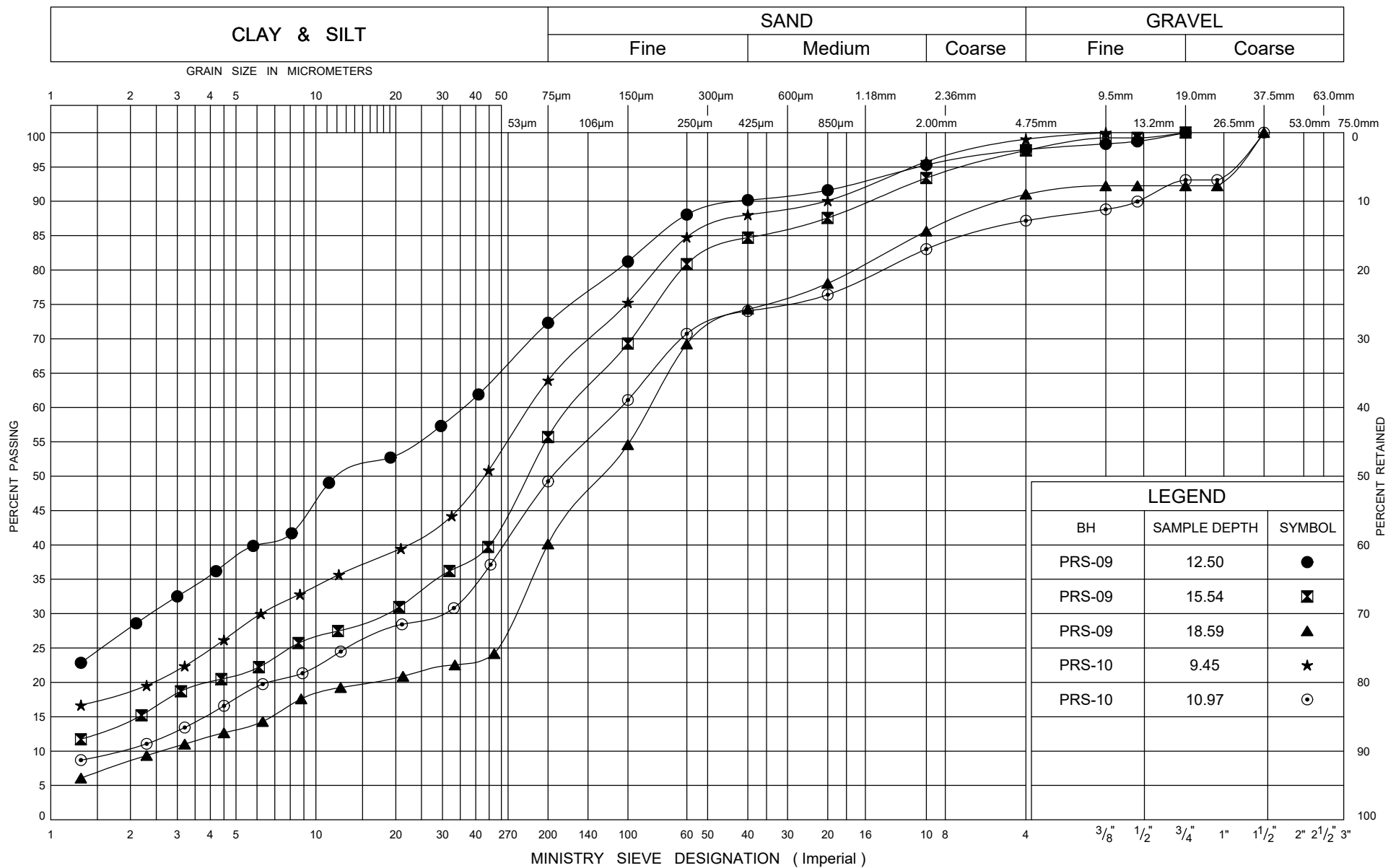
WP# 2555-17-00

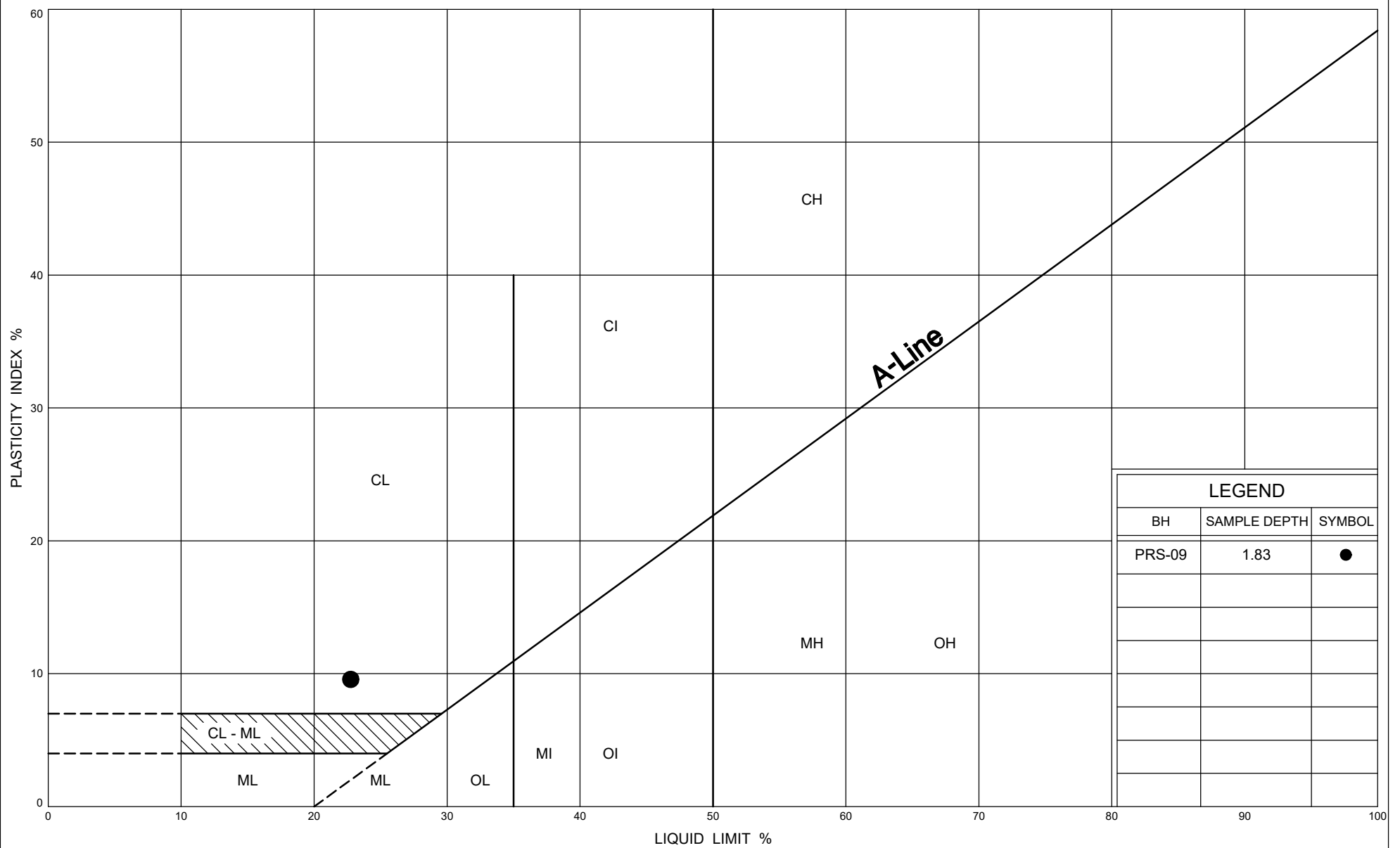
Park Road South Bridge











Ministry of
Transportation

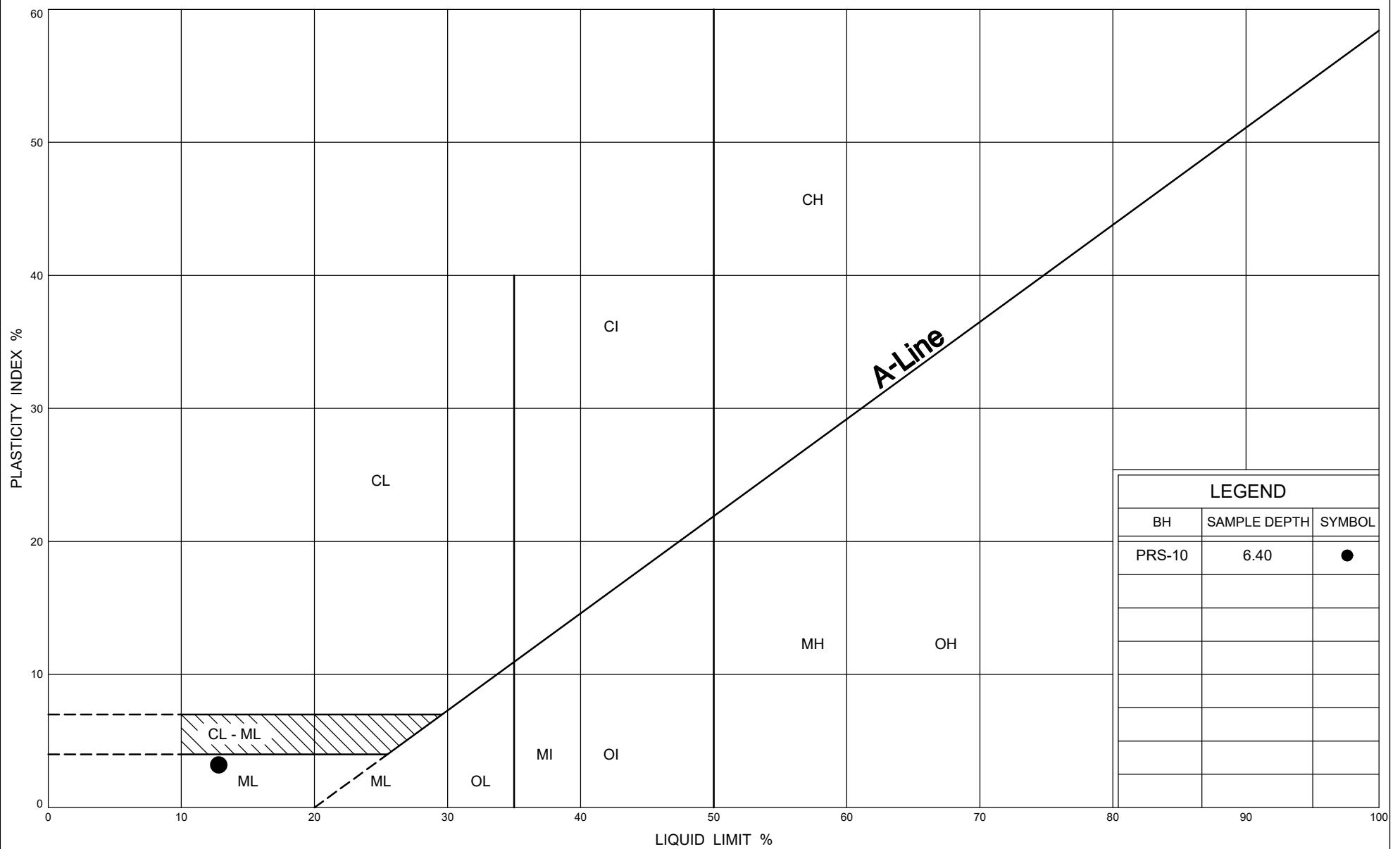
PLASTICITY CHART

Silty CLAY FILL

FIG No C9

WP# 2555-17-00

Park Road South Bridge



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Transportation

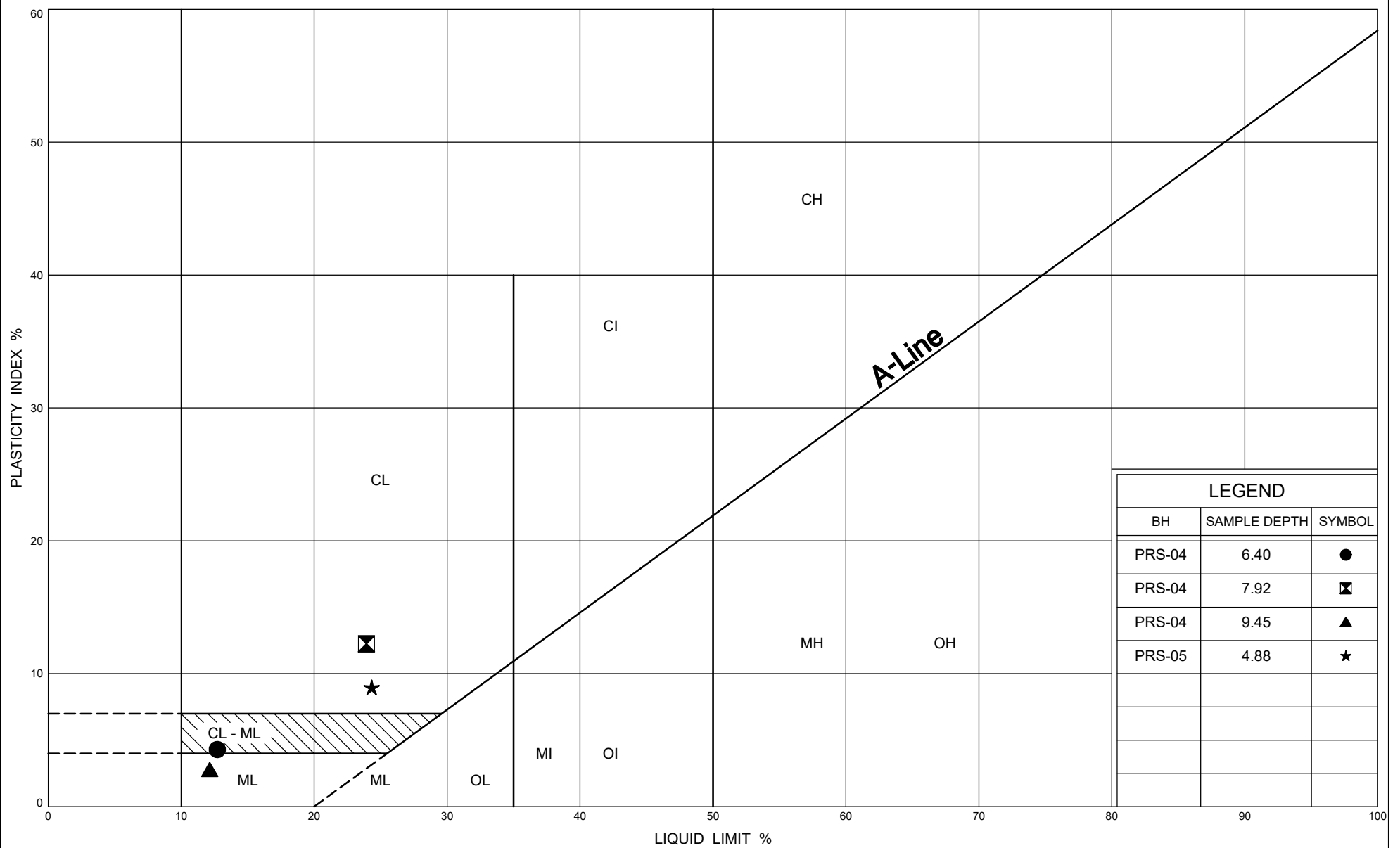
PLASTICITY CHART

SAND and SILT TILL

FIG No C10

WP# 2555-17-00

Park Road South Bridge



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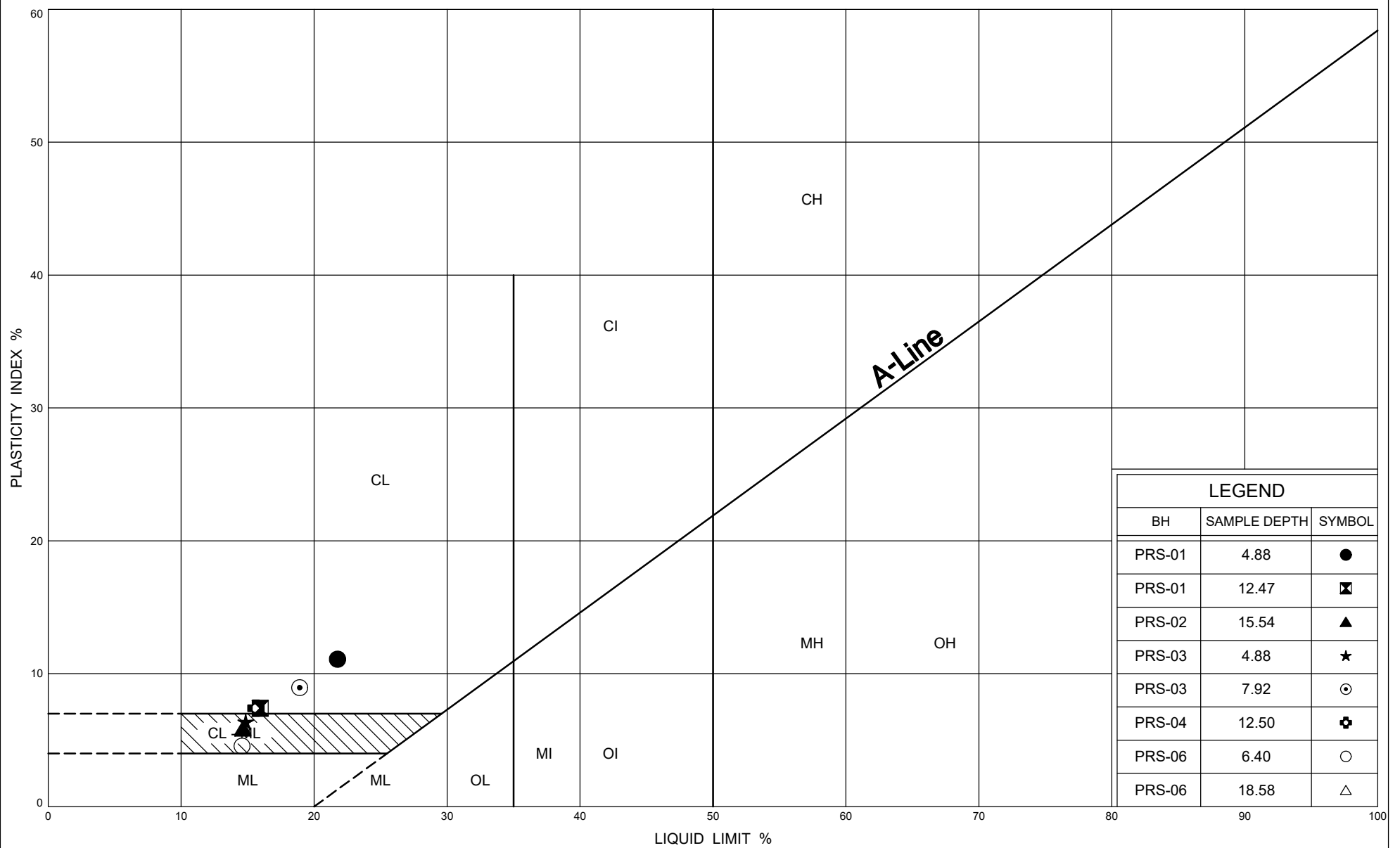
PLASTICITY CHART

Clayey SILT to Silty CLAY

FIG No C11

WP# 2555-17-00

Park Road South Bridge



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Transportation

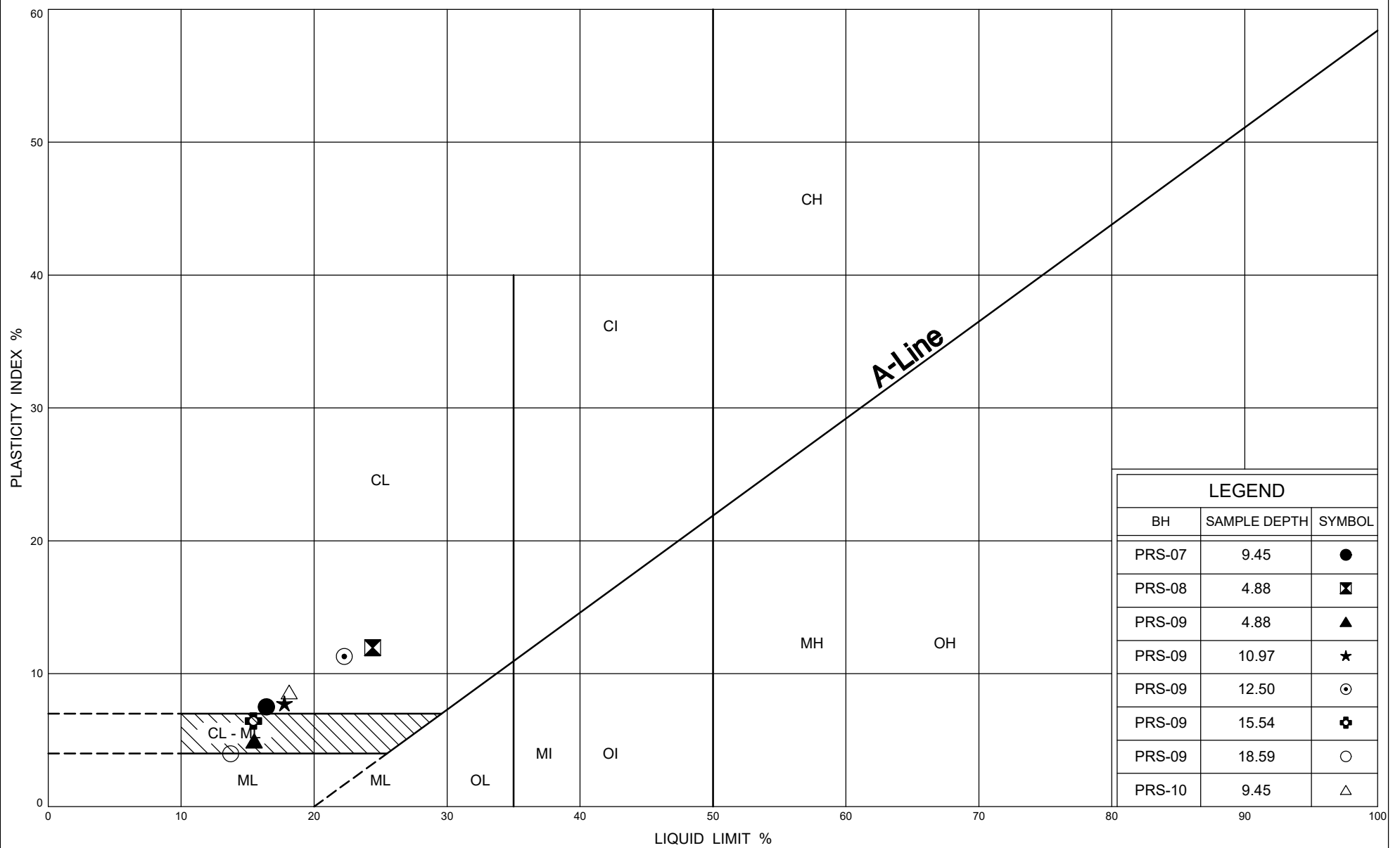
PLASTICITY CHART

Clayey SILT TILL

FIG No C12

WP# 2555-17-00

Park Road South Bridge



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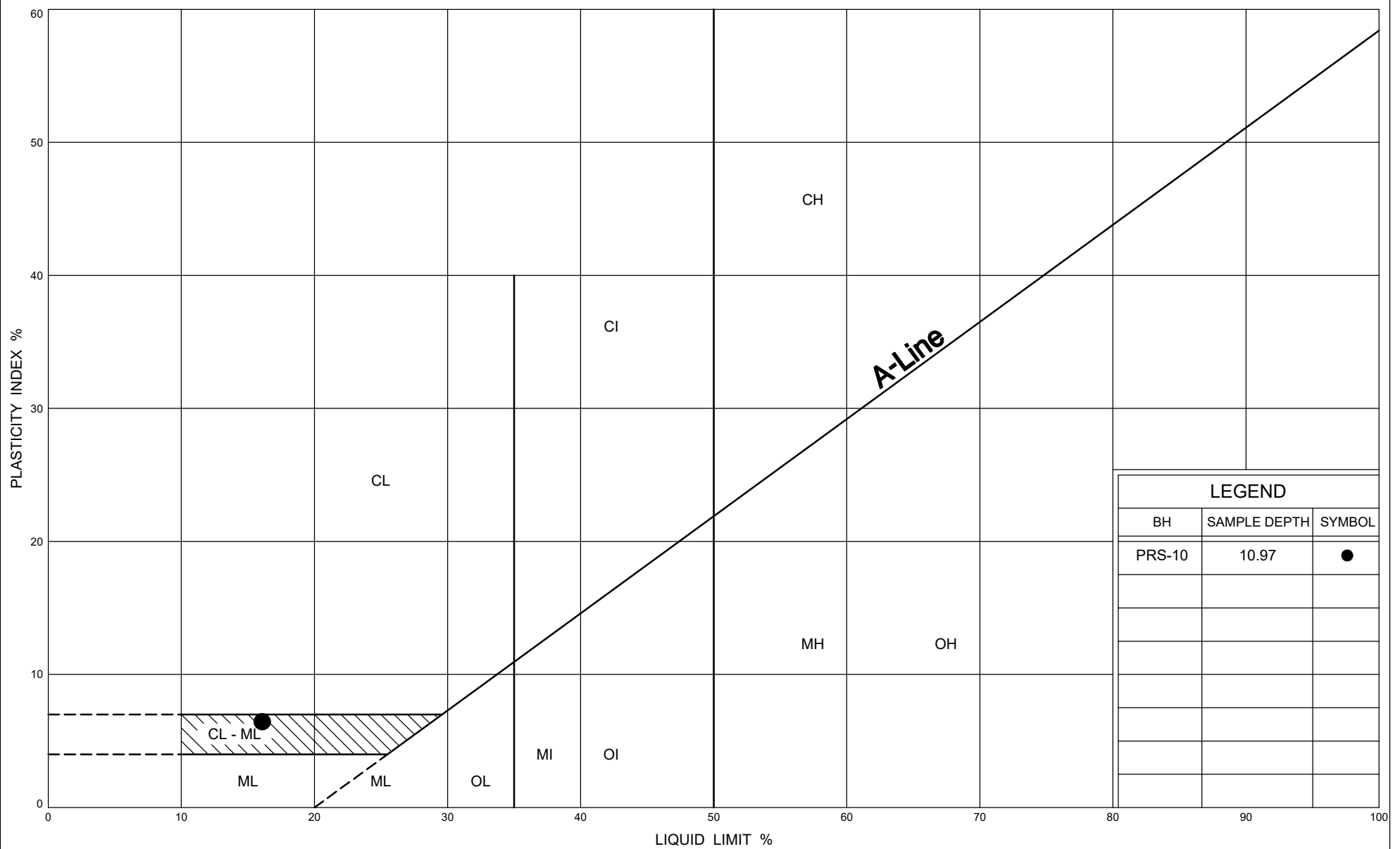
PLASTICITY CHART

Clayey SILT TILL

FIG No C13

WP# 2555-17-00

Park Road South Bridge



LEGEND		
BH	SAMPLE DEPTH	SYMBOL
PRS-10	10.97	●



Ministry of
Transportation

PLASTICITY CHART

Clayey SILT TILL

FIG No C14

WP# 2555-17-00

Park Road South Bridge



FINAL REPORT

CA40202-JUN23 R1

30915

Prepared for

Thurber Engineering Ltd.



FINAL REPORT

CA40202-JUN23 R1

First Page

CLIENT DETAILS		LABORATORY DETAILS	
Client	Thurber Engineering Ltd.	Project Specialist	Maarit Wolfe, Hon.B.Sc
Address	103, 2010 Winston Park Drive	Laboratory	SGS Canada Inc.
	Oakville, ON	Address	185 Concession St., Lakefield ON, K0L 2H0
	L6H 5R7, Canada		
Contact	Rod de Castro	Telephone	705-652-2000
Telephone		Facsimile	705-652-6365
Facsimile		Email	Maarit.Wolfe@sgs.com
Email	rdecastro@thurber.ca	SGS Reference	CA40202-JUN23
Project	30915	Received	06/20/2023
Order Number		Approved	06/29/2023
Samples	Soil (8)	Report Number	CA40202-JUN23 R1
		Date Reported	06/29/2023

COMMENTS
Temperature of Sample upon Receipt: 12 degrees C
Cooling Agent Present: Yes
Custody Seal Present: Yes
Chain of Custody Number: n/a
Corrosivity Index is based on the American Water Works Corrosivity Scale according to AWWA C-105. An index greater than 10 indicates the soil matrix may be corrosive to cast iron alloys.


SIGNATORIES
Maarit Wolfe, Hon.B.Sc 



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Annexes..... 8



FINAL REPORT

CA40202-JUN23 R1

Client: Thurber Engineering Ltd.

Project: 30915

Project Manager: Rod de Castro

Samplers: Abdul Basit

MATRIX: SOIL			Sample Number	5	6	7	8	9	10	11	12
			Sample Name	PRS-01 SS5	PRS-02 SS5	PRS-03 SS5	PRS-04 SS4	CS-01 SS5	CS-02 SS5	CS-03 SS3	CS-04 SS4
			Sample Matrix	Soil	Soil	Soil	Soil	Soil	Soil	Soil	Soil
			Sample Date	16/11/2022	17/11/2022	14/11/2022	22/11/2022	23/11/2022	12/01/2023	12/12/2022	25/11/2022
Parameter	Units	RL		Result	Result	Result	Result	Result	Result	Result	Result
Corrosivity Index											
Corrosivity Index	none	1		6	6	6	18	8	4	6	6
Soil Redox Potential	mV	no		254	210	241	204	269	314	314	291
Sulphide (Na2CO3)	%	0.04		0.06	0.05	0.06	0.06	0.11	0.06	0.07	0.05
pH	pH Units	0.05		8.63	8.52	8.39	8.61	8.69	8.44	8.57	8.54
Resistivity (calculated)	ohms.cm	-9999		5590	7300	2480	972	2650	5050	3150	5460
General Chemistry											
Conductivity	uS/cm	2		179	137	403	1030	378	198	317	183
Metals and Inorganics											
Moisture Content	%	0.1		0.9	4.2	3.6	5.7	0.7	1.6	0.6	0.4
Sulphate	µg/g	0.4		100	120	160	160	350	250	200	210
Other (ORP)											
Chloride	µg/g	0.4		47	40	2400	3400	18	18	47	14



FINAL REPORT

CA40202-JUN23 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	DIO0625-JUN23	µg/g	0.4	<0.4	4	35	95	80	120	100	75	125
Sulphate	DIO0625-JUN23	µg/g	0.4	<0.4	15	35	96	80	120	99	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 (Na2CO3)	ECS0057-JUN23	%	0.04	< 0.04	ND	20	109	80	120			
Sulphide (Na2CO3)	ECS0066-JUN23	%	0.04	< 0.04	ND	20	115	80	120			



FINAL REPORT

CA40202-JUN23 R1

QC SUMMARY

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	EWL0556-JUN23	uS/cm	2	< 2	0	20	98	90	110	NA		

pH

Method: SM 4500 | Internal ref.: ME-CA-IENVIEWL-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	EWL0556-JUN23	pH Units	0.05	NA	0		100			NA		



FINAL REPORT

CA40202-JUN23 R1

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

Results relate only to the sample tested.

Data reported represent the sample as submitted to SGS. 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 and Excess Soil Quality" 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.

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

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-- End of Analytical Report --

Laboratory Information Section - Lab use only

Received By: K. Kinkina
Received Date (mm/dd/yy): 06-20-23
Received Time: 14:40Received By (signature): [Signature]
Custody Seal Present: ☒
Custody Seal Intact: ☒Cooling Agent Present: ☒
Temperature Upon Receipt (°C): 12.3LAB LIMS # CA40202-2023

REPORT INFORMATION		INVOICE INFORMATION		PROJECT INFORMATION	
Company: <u>Thurber Engineering Ltd.</u>	<input checked="" type="checkbox"/> (same as Report Information)	Quotation #: <u>30915</u>	P.O. #: <u></u>	Site Location/ID: <u></u>	
Contact: <u>Rod de Castro</u>	Company: <u></u>	Project #: <u>30915</u>	TURNAROUND TIME (TAT) REQUIRED		
Address: <u>103-2010 Winston Park Drive</u>	Contact: <u></u>	TAT's are quoted in business days (exclude statutory holidays & weekends).			
<u>Oakville, Ontario</u>	Address: <u></u>	Samples received after 6pm or on weekends: TAT begins next business day			
Phone: <u>905 829 8666 x 5244</u>	Phone: <u></u>	Regular TAT (5-7days) <input checked="" type="checkbox"/> Regular TAT (Additional Charges May Apply): <input type="checkbox"/> 1 Day <input type="checkbox"/> 2 Days <input type="checkbox"/> 3 Days <input type="checkbox"/> 4 Days			
Email: <u>rdcastro@thurber.ca</u>	Email: <u></u>	RUSH TAT (Additional Charges May Apply): <input type="checkbox"/> 1 Day <input type="checkbox"/> 2 Days <input type="checkbox"/> 3 Days <input type="checkbox"/> 4 Days			
Email: <u></u>	Email: <u></u>	PLEASE CONFIRM RUSH FEASIBILITY WITH SGS REPRESENTATIVE PRIOR TO SUBMISSION			
Specify Due Date: <u></u>		Rush Confirmation ID: <u></u>			
NOTE: DRINKING (POTABLE) WATER SAMPLES FOR HUMAN CONSUMPTION MUST BE SUBMITTED WITH SGS DRINKING WATER CHAIN OF CUSTODY					
REGULATIONS		ANALYSIS REQUESTED		COMMENTS:	
Regulation 153/04: Table 1: <input type="checkbox"/> R/P/I <input type="checkbox"/> Soil Texture: <input type="checkbox"/> Coarse <input type="checkbox"/> Medium <input type="checkbox"/> Fine Table 2: <input type="checkbox"/> J/C/C <input type="checkbox"/> AVO Table 3: <input type="checkbox"/> AVO Table 4: <input type="checkbox"/> AVO		Other Regulations: <input type="checkbox"/> Reg 347/658 (3 Day min TAT) <input type="checkbox"/> PWQO <input type="checkbox"/> MMER <input type="checkbox"/> Other: <input type="checkbox"/> CCME <input type="checkbox"/> MISA		Sewer By-Law: <input type="checkbox"/> Sanitary <input type="checkbox"/> Storm <input type="checkbox"/> Municipality:	
RECORD OF SITE CONDITION (RSC)		DATE SAMPLED		TIME SAMPLED	
SAMPLE IDENTIFICATION		DATE SAMPLED		TIME SAMPLED	
MATRIX		# OF BOTTLES		DATE SAMPLED	
1 PRS-01 SS5	11/16/22	AM	1	SOIL	
2 PRS-02 SS5	11/17/22	AM	1	SOIL	
3 PRS-03 SS5	11/14/22	AM	1	SOIL	
4 PRS-04 SS4	11/22/22	AM	1	SOIL	
5 CS-01 SS5	11/23/22	AM	1	SOIL	
6 CS-02 SS5	11/23/22	AM	1	SOIL	
7 CS-03 SS3	12/5/22	AM	1	SOIL	
8 CS-04 SS4	11/25/22	AM	1	SOIL	
9					
10					
11					
12					
Observations/Comments/Special Instructions					

Sampled By (NAME): Abdul BasitSignature: ABDate: 06/19/23

(mm/dd/yy)

Pink Copy - Client

Relinquished by (NAME): Czarlene PontejosSignature: C-PDate: 06/20/23

(mm/dd/yy)

Yellow & White Copy - SGS



THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET
ASTM D5731-08

Job No: 30915
Project Name: Hwy 401 & Park St. S Overpass
Core Size: HQ BH No : PRS-02

Date Drilled: 18-Nov-22
Date Tested: 17-May-23
Tester: AK
Client:

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	$I_{s(50)}$ (MPa)	UCS (MPa)	Rock Type	Rock Strength (after Hoek & Brown, 1997)
1	1	17.2	D	9.2	60.7	86.5	2.6	62.0	Limestone	Strong
2	1	17.3	D	6.9	60.7	87.4	1.9	46.5	Limestone	Medium Strong
3	1	17.6	D	7.4	60.7	80.9	2.1	49.9	Limestone	Medium Strong
4	2	18.7	A	15.2	60.7	64.0	3.4	81.5	Limestone	Strong
5	2	18.9	D	12.2	60.7	73.0	3.4	82.2	Limestone	Strong
6	2	19.3	D	19.1	60.7	82.0	5.4	128.7	Limestone	Very Strong
7	3	20.3	D	5.9	60.7	80.0	1.7	39.8	Limestone	Medium Strong
8	3	20.5	D	6.2	60.7	88.2	1.7	41.8	Limestone	Medium Strong
9	3	21.0	D	9.4	60.7	79.3	2.6	63.3	Limestone	Strong
10										
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35										

- * It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing
- * Diametral Test should have $0.7 \times D$ on either side of test point.
- * Correlation factor to obtain UCS values is 24.



THURBER ENGINEERING LTD.

POINT LOAD TEST SHEET
ASTM D5731-08

Job No: 30915
Project Name: Hwy 401 & Park St. S Overpass
Core Size: HQ BH No : PRS-03

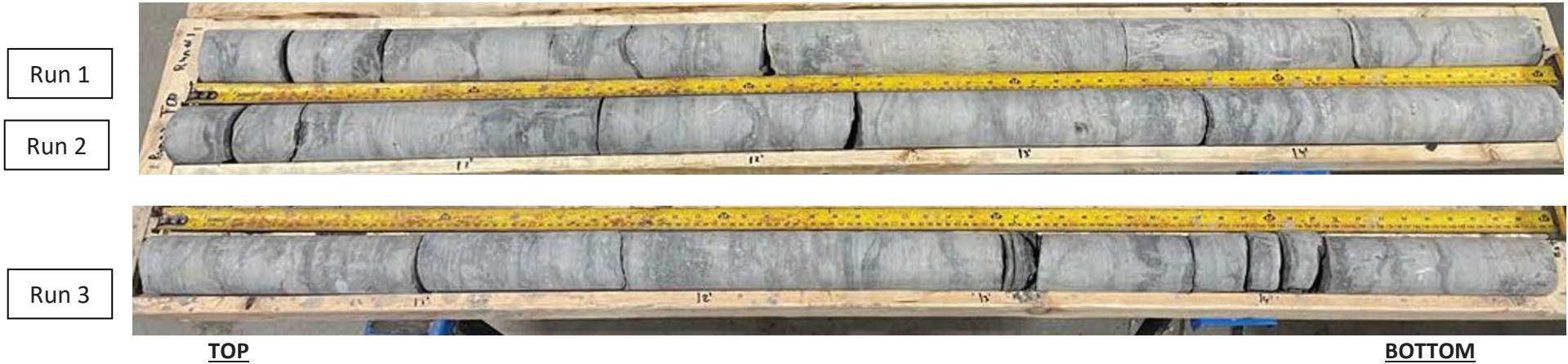
Date Drilled: 15-Nov-22
Date Tested: 16-May-23
Tester: AK
Client:

Test No.	Run No.	Depth (m)	Axial or Diametral	Gauge (MPa)	Diameter (mm)	Length (mm)	$I_{s(50)}$ (MPa)	UCS (MPa)	Rock Type	Rock Strength (after Hoek & Brown, 1997)
1	1	16.5	A	9.6	60.7	70.9	2.0	47.6	Limestone	Medium Strong
2	1	16.6	A	10.3	60.7	71.5	2.1	50.7	Limestone	Strong
3	1	16.9	A	13.3	60.7	70.1	2.8	66.5	Limestone	Strong
4	2	17.3	A	7.9	60.7	65.2	1.7	41.8	Limestone	Medium Strong
5	2	17.6	A	8.3	60.7	66.3	1.8	43.3	Limestone	Medium Strong
6	2	18.4	A	13.1	60.7	65.7	2.9	68.8	Limestone	Strong
7	3	18.7	D	13.8	60.7	74.3	3.9	93.0	Limestone	Strong
8	3	18.9	D	17.7	60.7	61.0	5.0	119.3	Limestone	Very Strong
9	3	19.6	D	10.0	60.7	69.0	2.8	67.4	Limestone	Strong
10	4	20.3	D	11.2	60.7	78.5	3.1	75.5	Limestone	Strong
11	4	21.5	D	9.7	60.7	81.8	2.7	65.4	Limestone	Strong
12	4	21.6	D	6.7	60.7	80.5	1.9	45.1	Limestone	Medium Strong
13										
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35										

- * It is ideal to perform axial test on core specimens with D/L ratio of 1.1 ± 0.1
Long pieces of core can be tested diametrically to produce suitable lengths for axial testing
- * Diametral Test should have $0.7 \times D$ on either side of test point.
- * Correlation factor to obtain UCS values is 24.

PHOTOGRAPHS OF ROCK CORES - BOREHOLE PRS-02

RUNS 1 to 3



Date Drilled: November 18, 2022

Run #	Depth (ft)	Depth (m)
1	(56'6" – 61'6")	(17.2 – 18.7)
2	(61'6" – 66'6")	(18.7 – 20.3)
3	(66'6" – 71'6")	(20.3 – 21.8)

PHOTOGRAPHS OF ROCK CORES - BOREHOLE PRS-03

RUNS 1 to 3

Run 1

Run 2

Run 3

Run 4



TOP

BOTTOM

Date Drilled: November 15, 2022

Run #	Depth (ft)	Depth (m)
1	(51'6" – 56'6")	(15.7 – 17.2)
2	(56'6" – 61'6")	(17.2 – 18.7)
3	(61'6" – 66'6")	(18.7 – 20.3)
4	(66'6" – 71'6")	(20.3 – 21.8)

APPENDIX D

Records of Borehole Sheets

Previous Investigation

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 1

JOB 72-11149

LOCATION Co-ordinates 944,345 N. - 166,376 E.

ORIGINATED BY J.B.

W.P. 44-71-06

BORING DATE January 22, 1973

COMPILED BY J.B.

DATUM Geodetic

BOREHOLE TYPE B.W. Casing

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT _____ PLASTIC LIMIT _____ WATER CONTENT _____		BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD V.A. * ● QUICK TRIAXIAL x LAB. V.A. *		WATER CONTENT % 5 10 15			
349.1	Ground Level											
0.0	Hetrogeneous mixture of Sand, Silt, Gravel and Clay		1	SS	87	340						349.0 10 LG 36 12
341.1	Brown to Grey		2	SS	100/4"							
8.0	Grey		3	SS	100/4"							
	Very Dense		4	SS	81							
			5	SS	100/4"							
	Glacial Till						330					
525.3			6	SS	100/4"							
		7	SS	100/4"								
23.8	End of Borehole					320						

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 2

JOB NO. 2-21109

LOCATION Cochrane, Ont. 185 N. - 100, 100 E.

ORIGINATED BY J.R.

DATE 11-11-73

BORING DATE January 29, 1973

COMPILED BY J.R.

DATUM Geodetic

BOREHOLE TYPE B.W. Casing

CHECKED BY J.R.

SOIL PROFILE			SAMPLES		ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT		LIQUID LIMIT — %		BULK DENSITY	REMARKS							
ELEV. DEPTH	DESCRIPTION	STRAT. NO.	NUMBER	TYPE		20	40	60	80			100	PLASTIC LIMIT — %	WATER CONTENT — %				
						SHEAR STRENGTH P.S.F.					UNCONFINED + FIELD VANE		QUICK TRIAXIAL + FIELD VANE		WATER CONTENT %			
347.6	Ground Level																	
0.0	Heterogeneous Mixture of Silt, Sand, Gravel and Clay.		1	SS	100/4"													Elev. 347.6
	Very Dense, Glacial Till		2	SS	100/4"	340												3 43 10 10
336.6	Down to Grey		3	SS	100/4"													
11.0	Grey		4	SS	100/4"													
			5	SS	100/4"	320												5 37 39 20
	Heterogeneous Mixture of Clayey Silt, Sand and Gravel.		6	SS	100/4"													
	Very Stiff to Hard		7	SS	100/4"	320												
	Glacial Till		8	SS	100/4"													24-36 30 10
	Remarked Zone Between Elev. 311-332		9	SS	100/4"	310												
			10	SS	100/4"													
			11a	SS	100/4"													
			11b	SS	100/4"													
			12	SS	100/4"	300												5 38 39 18
294.1			13	SS	100/4"													
53.3	End of borehole					290												

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE N^o3

JOB 72-11149

LOCATION Co-ordinates 944,194 E. - 166,437 E.

ORIGINATED BY J.B.

W.P. 44-7-06

BORING DATE January 25, 1973

COMPILED BY J.B.

DATUM Geodetic

BOREHOLE TYPE B.W. Casing

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT				LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W			BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE				WATER CONTENT % W_P W W_L				
347.9	Ground Level														GR.SA.SI.CL
0.0	Hetrogeneous Mixture of Silt, Sand, Gravel and Clay.		1	SS	31	340								Elev. 343.4	9 36 41 14
	Dense to Very Dense		2	SS	100/9"										
	Glacial Till		3	SS	89										
333.9	Brown to Grey		4	SS	60										4 44 38 14
14.0	Grey		5	SS	16	330									
	Hetrogeneous Mixture of Clayey Silt, Sand and Gravel		6	SS	19										
	Very Stiff to Hard		7	SS	39	320									14 41 35 10
	Re-worked Zone Between Elev. 308-334		8	SS	30										
	Glacial Till		9	SS	28	310									
			10	SS	100/9"										
			11	SS	100/6"	300									1 40 37 22
			12	SS	100/7"										
297.3	End of Borehole					290									
50.6															

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 4

JOB 72-11149

LOCATION Co-ordinates 944,351 N. - 166,303 E.

ORIGINATED BY J.B.

W.P. 44-76-01

BORING DATE January 26, 1973

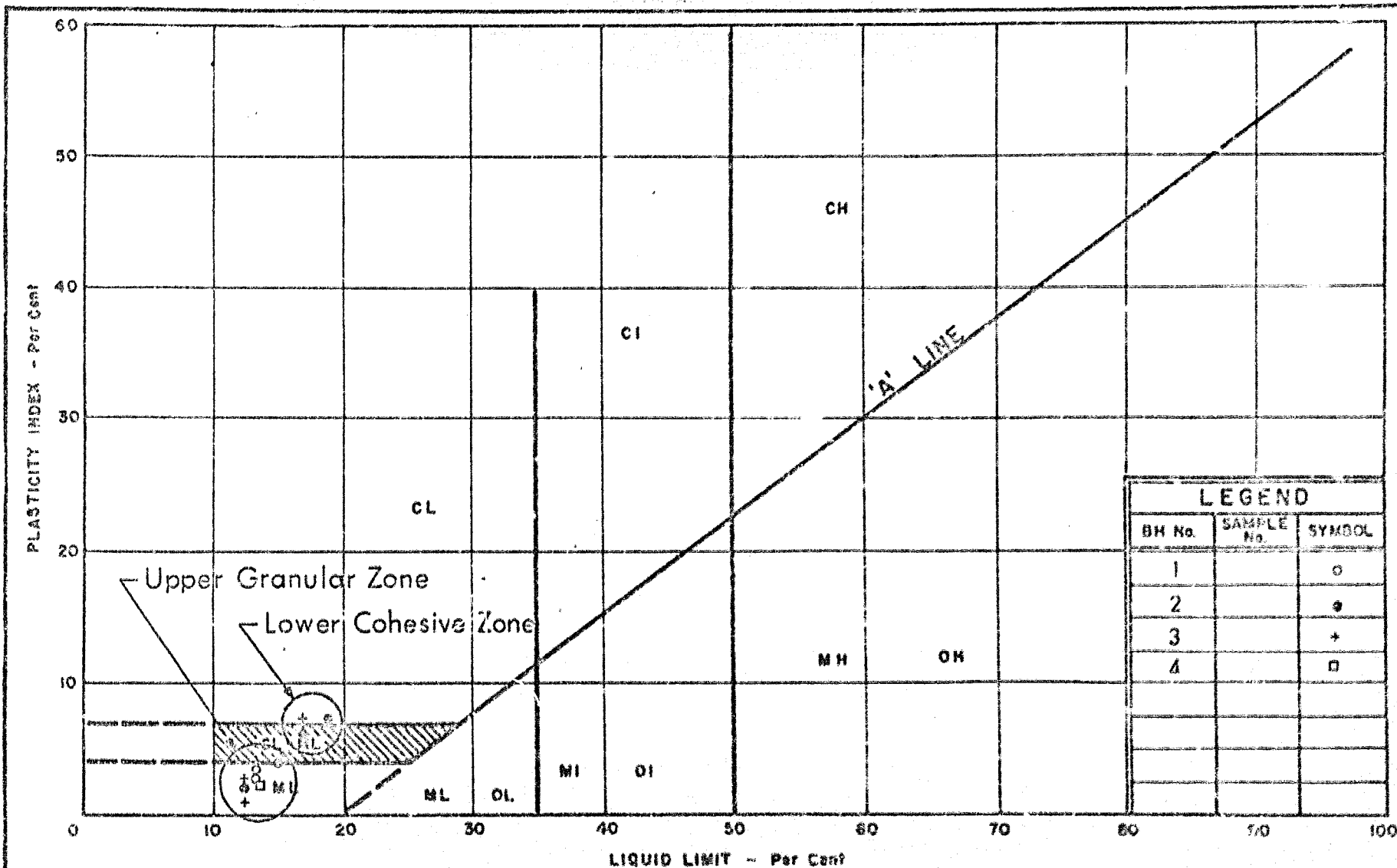
COMPILED BY J.B.

DATUM Geodetic

BOREHOLE TYPE B.W. Casing

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT			LIQUID LIMIT _____ w_L PLASTIC LIMIT _____ w_p WATER CONTENT _____ w w_p — w — w_L			BULK DENSITY γ P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE			WATER CONTENT %					
353.8	Ground Level										5	10	15		GR SA SI CL
0.0	Hetrogeneous Mixture of Sand, Silt, Gravel and Clay		1	SS	19	350									17 37 34 12 Elev. 346.2
	Compact to Very Dense		2	SS	100										
	Brown to Grey		3	SS	100		340								
	Grey		4	SS	75							0	1		
			5	SS	64										
	Glacial Till		6	SS	28	330									
332.0		7	SS	100											
21.8	Sand and Gravel														
323.3	Very Dense		8	SS	100	320									50 47 (3)
30.5	End of Borehole														



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

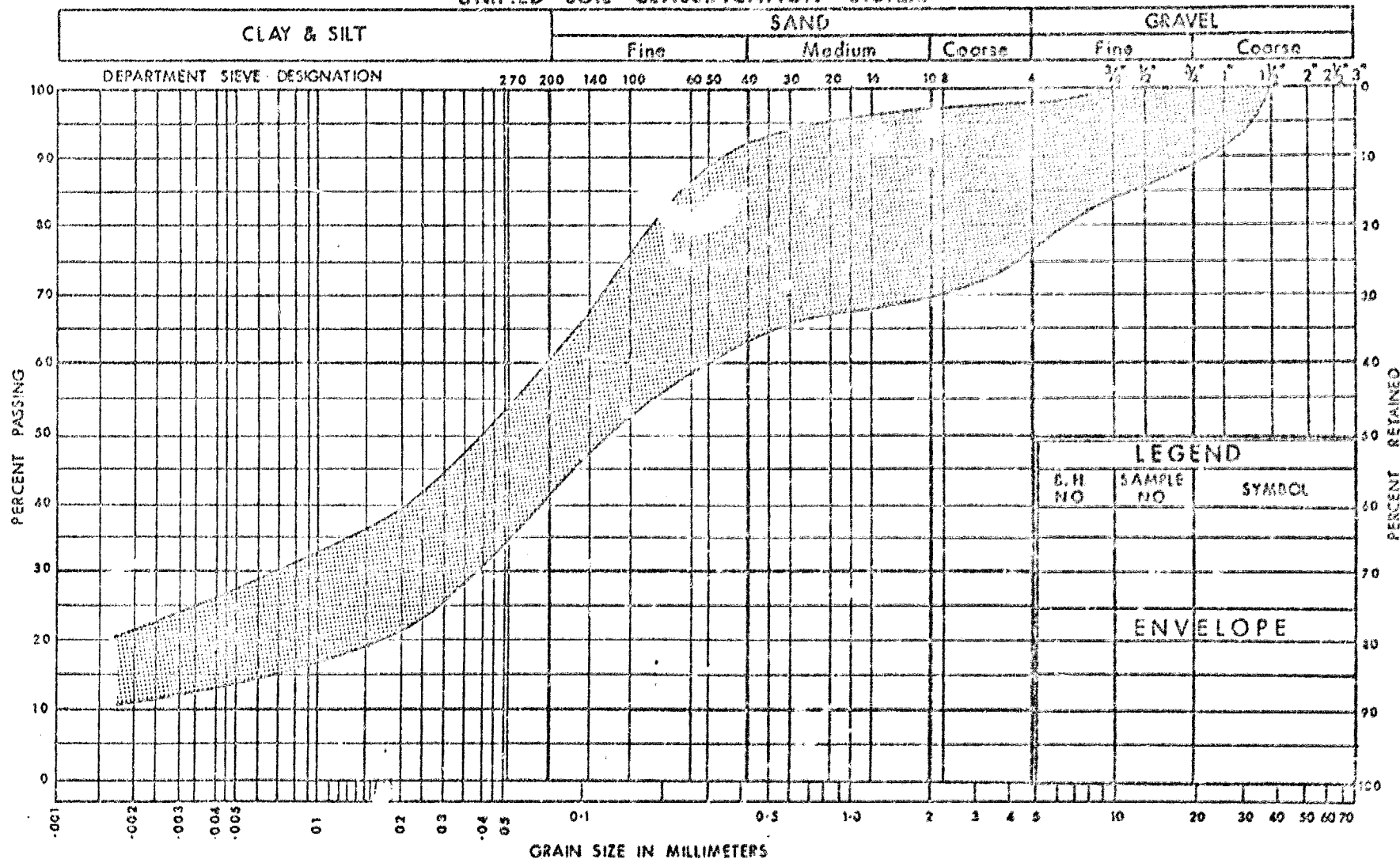
PLASTICITY CHART GLACIAL TILL

WT No. 44-71-06

JOB No. 72-11149

FIG. N^o 1

UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT
OF
TRANSPORTATION AND COMMUNICATIONS



DESIGN SERVICES
BRANCH

GRAIN SIZE DISTRIBUTION GLACIAL TILL

W.P. No. 44-71-06

JOB No. 72-11149

FIG. N° 2

FD-9a (Rev. Jan 73)

ABBREVIATIONS & SYMBOLS USED IN THIS REPORT

PENETRATION RESISTANCE

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

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

DESCRIPTION OF SOIL

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

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

TERMS TO BE USED IN DESCRIBING SOILS:-

TRACE < 10% , SOME 10-25% , WITH 25-40% , > 40% SILTY, SANDY, GRAVELLY, CLAYEY ETC.

TYPE OF SAMPLE

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

P.H. SAMPLE ADVANCED HYDRAULICALLY

P.M. SAMPLE ADVANCED MANUALLY

SOIL TESTS

U	UNCONFINED COMPRESSION	L.V.	LABORATORY VANE
UU	UNCONSOLIDATED UNDRAINED TRIAXIAL	F.V.	FIELD VANE
CU	CONSOLIDATED ISOTROPIC UNDRAINED TRIAXIAL	C	CONSOLIDATION
CID	" " DRAINED "	S	SENSITIVITY
CAU	" " ANISOTROPIC UNDRAINED "		
CAD	" " DRAINED "		

ABBREVIATIONS & SYMBOLS USED IN THIS REPORTSOIL PROPERTIES

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

GENERAL

π	= 3.1416
e	BASE OF NATURAL LOGARITHMS 2.7183
$\log_e a$ OR $\ln a$	NATURAL LOGARITHM OF a
$\log_{10} a$ OR $\log a$	LOGARITHM OF a TO BASE 10
t	TIME
g	ACCELERATION DUE TO GRAVITY
V	VOLUME
W	WEIGHT
M	MOMENT
F	FACTOR OF SAFETY

STRESS AND STRAIN

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

EARTH PRESSURE

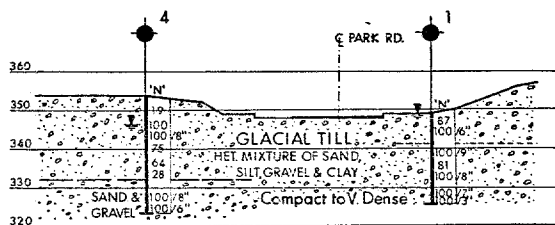
d	DISTANCE FROM TOP OF WALL TO POINT OF APPLICATION OF PRESSURE
δ	ANGLE OF WALL FRICTION
K	DIMENSIONLESS COEFFICIENT TO BE USED WITH VARIOUS SUFFIXES IN EXPRESSIONS REFERRING TO NORMAL STRESS ON WALLS
K_0	COEFFICIENT OF EARTH PRESSURE AT REST

FOUNDATIONS

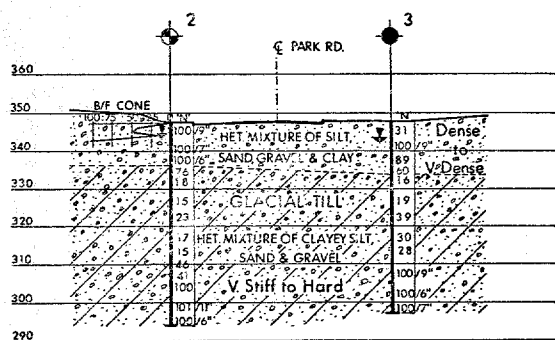
B	BREADTH OF FOUNDATION
L	LENGTH OF FOUNDATION
D	DEPTH OF FOUNDATION BENEATH GROUND
N	DIMENSIONLESS COEFFICIENT USED WITH A SUFFIX APPLYING TO SPECIFIC GRAVITY, DEPTH AND COHESION ETC. IN THE FORMULA FOR BEARING CAPACITY
k_s	MODULUS OF SUBGRADE REACTION

SLOPES

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

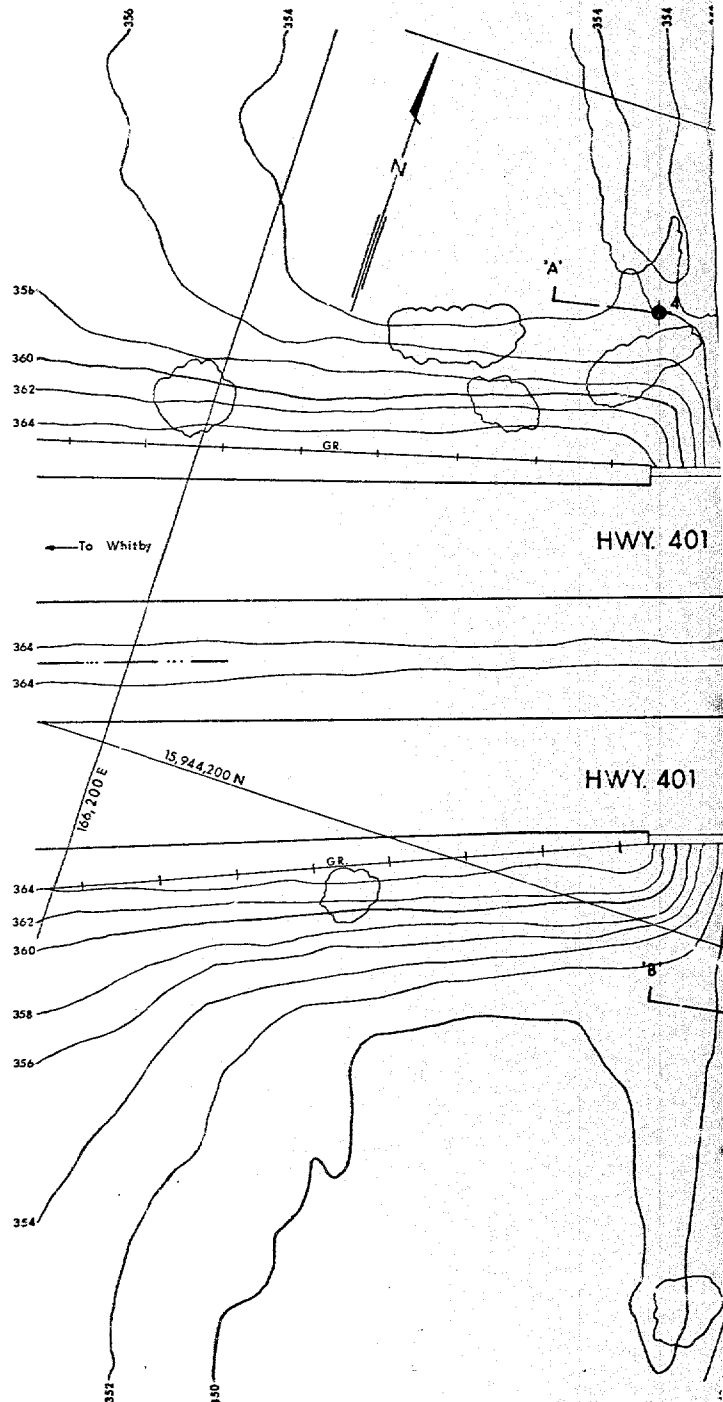


A-A

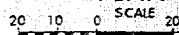


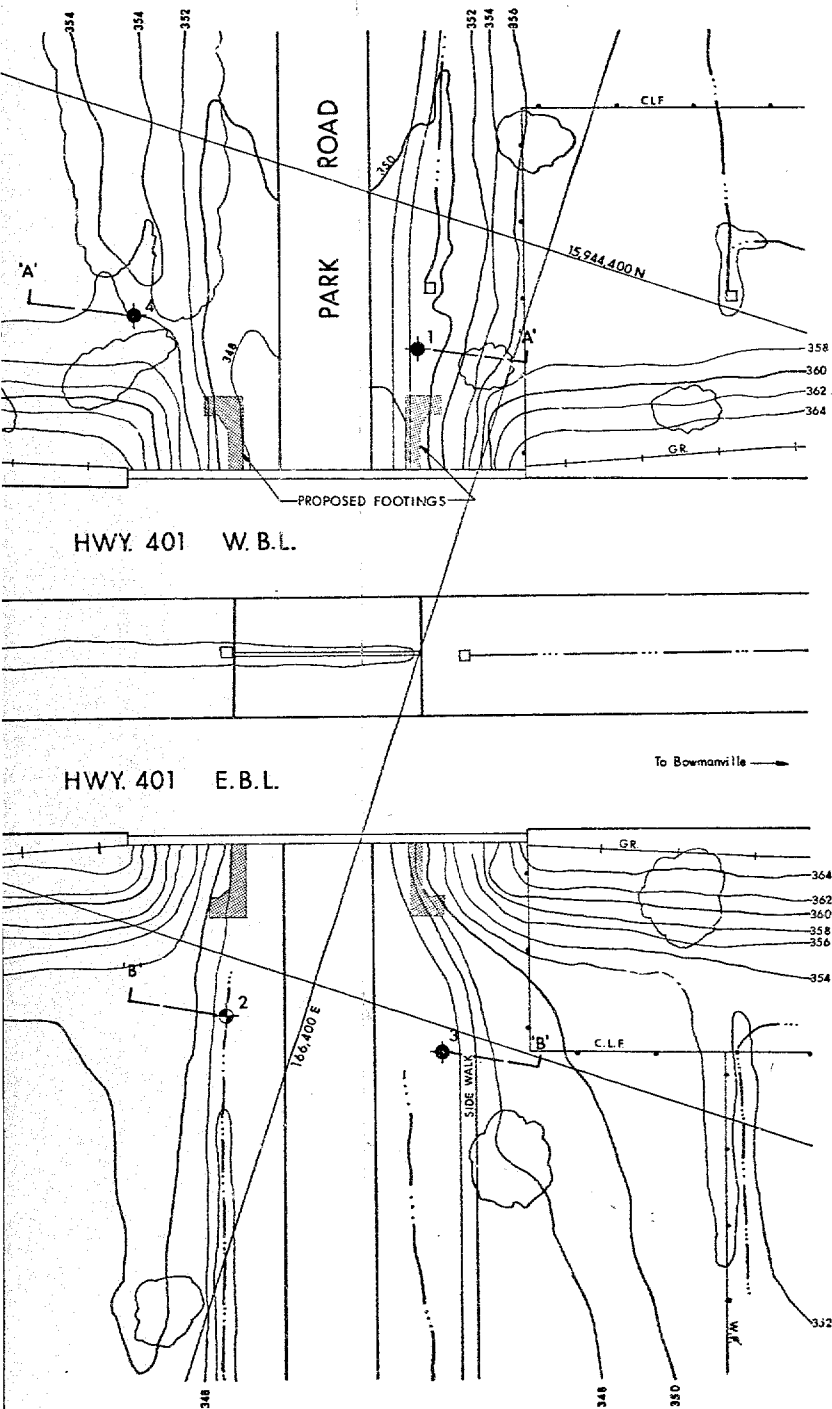
B-B

SECTIONS



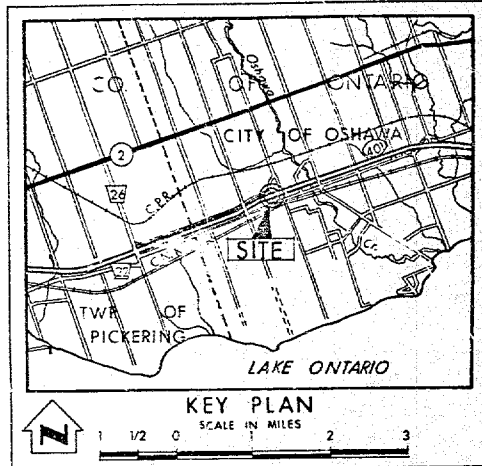
PLAN





PLAN
SCALE
20 10 0 20 40 FE.

REF. No. B-4-17



LEGEND

- Bore Hole
- ⊕ Cone Penetration Test
- ⊕ Bore Hole & Cone Test
- Water Levels established at time of field investigation, JAN. 1973

NO.	ELEVATION	CO-ORDINATES	
		NORTH	EAST
1	349.1	944,365	166,376
2	347.6	944,185	166,381
3	347.9	944,194	166,437
4	353.8	944,351	166,303

NOTE

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

REVISIONS	DATE	BY	DESCRIPTION

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS—ONTARIO
DESIGN SERVICES BRANCH—FOUNDATIONS OFFICE

PARK ROAD

HIGHWAY NO. 401 DIST. NO. 6
CO. ONTARIO CITY OF OSHAWA
TWP. LOT CON.

BORE HOLE LOCATIONS & SOIL STRATA

SUBMIT J.B.	CHECKED	WP NO. 44-71-06	DRAWING NO.
DRAWN F.L.	CHECKED	WD NO. 72-11149	72-11149A
DATE FEB 26, 1973	SITE NO.	BROGE DRAWING NO.	
APPROVED	PRINCIPAL FOUNDATION ENGINEER	CONT. NO.	

APPENDIX E

Selected Site Photographs



Photo 1- North Side of Highway 401 and Park St. S Overpass
Date: July 2023



Photo 2- North Side of Highway 401 and Park St. S Overpass
West abutment, north side (looking west)
Date: July 2023



Photo 3- South Side of Highway 401 and Park St. S Overpass
Date: July 2023



Photo 4- South Side of Highway 401 and Park St. S Overpass
East abutment, south side (looking east)
Date: July 2023



Photo 5- South Side of Highway 401 and Park St. S Overpass
West abutment, south side (looking east)
Date: July 2023

APPENDIX F

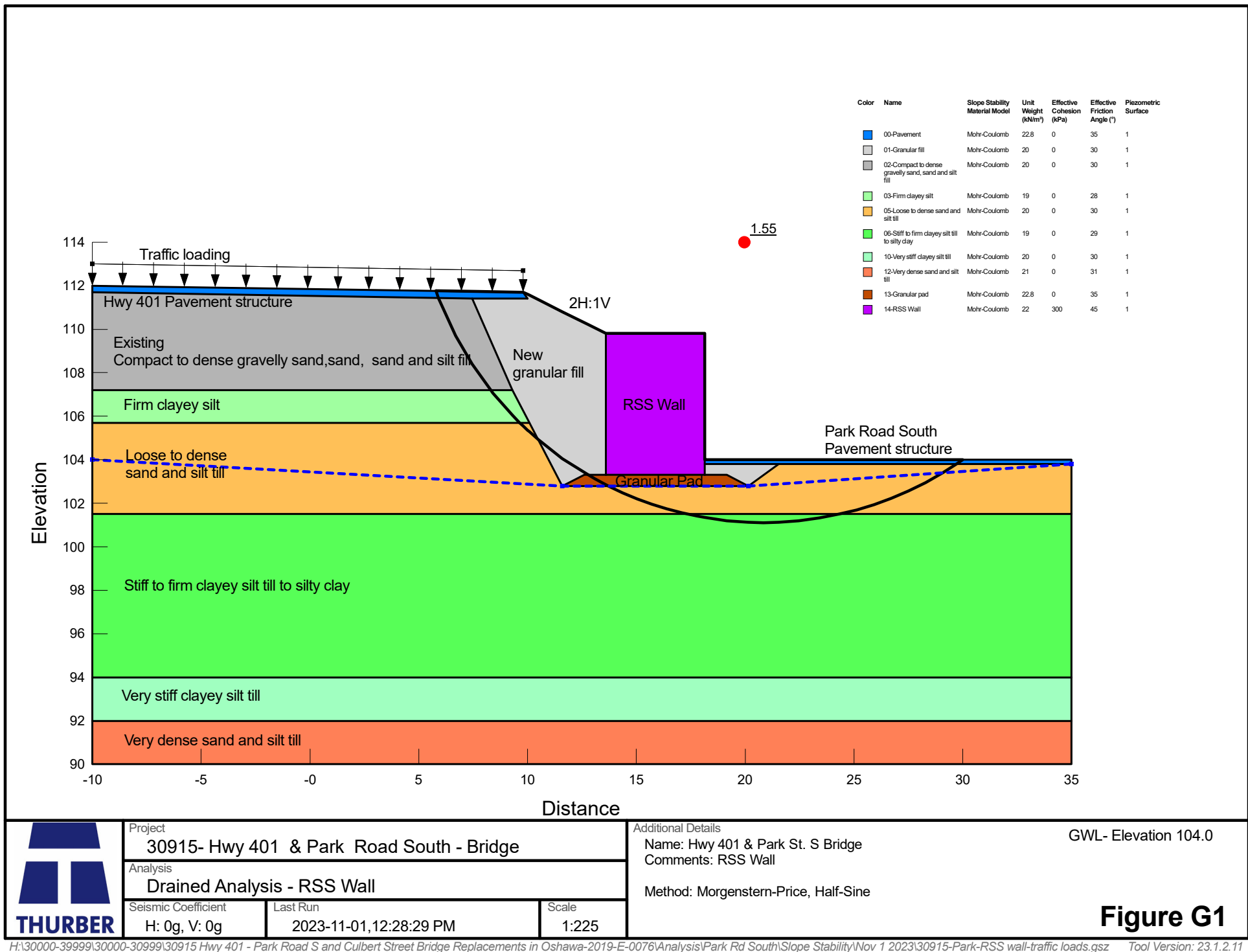
Foundation Comparison

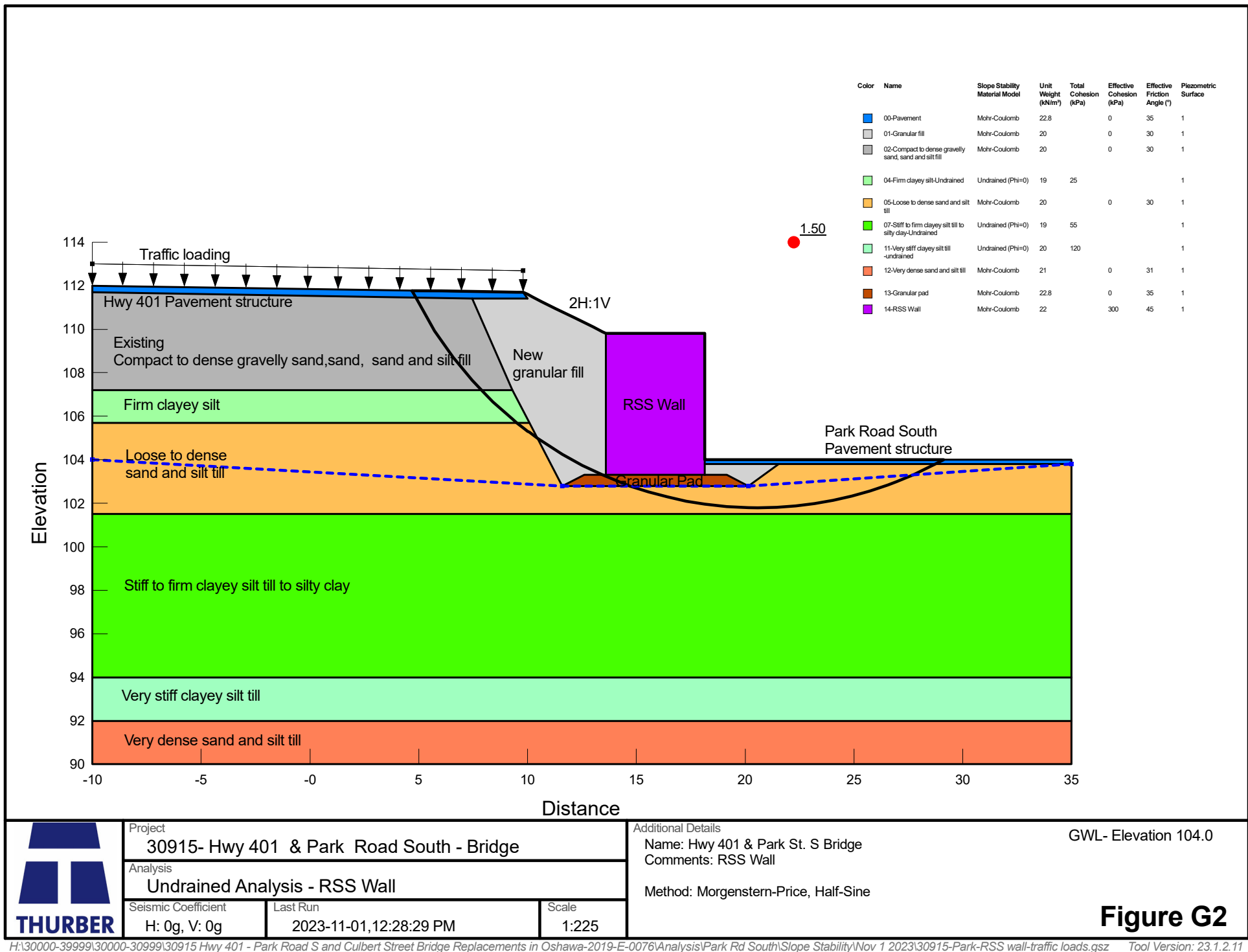
COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

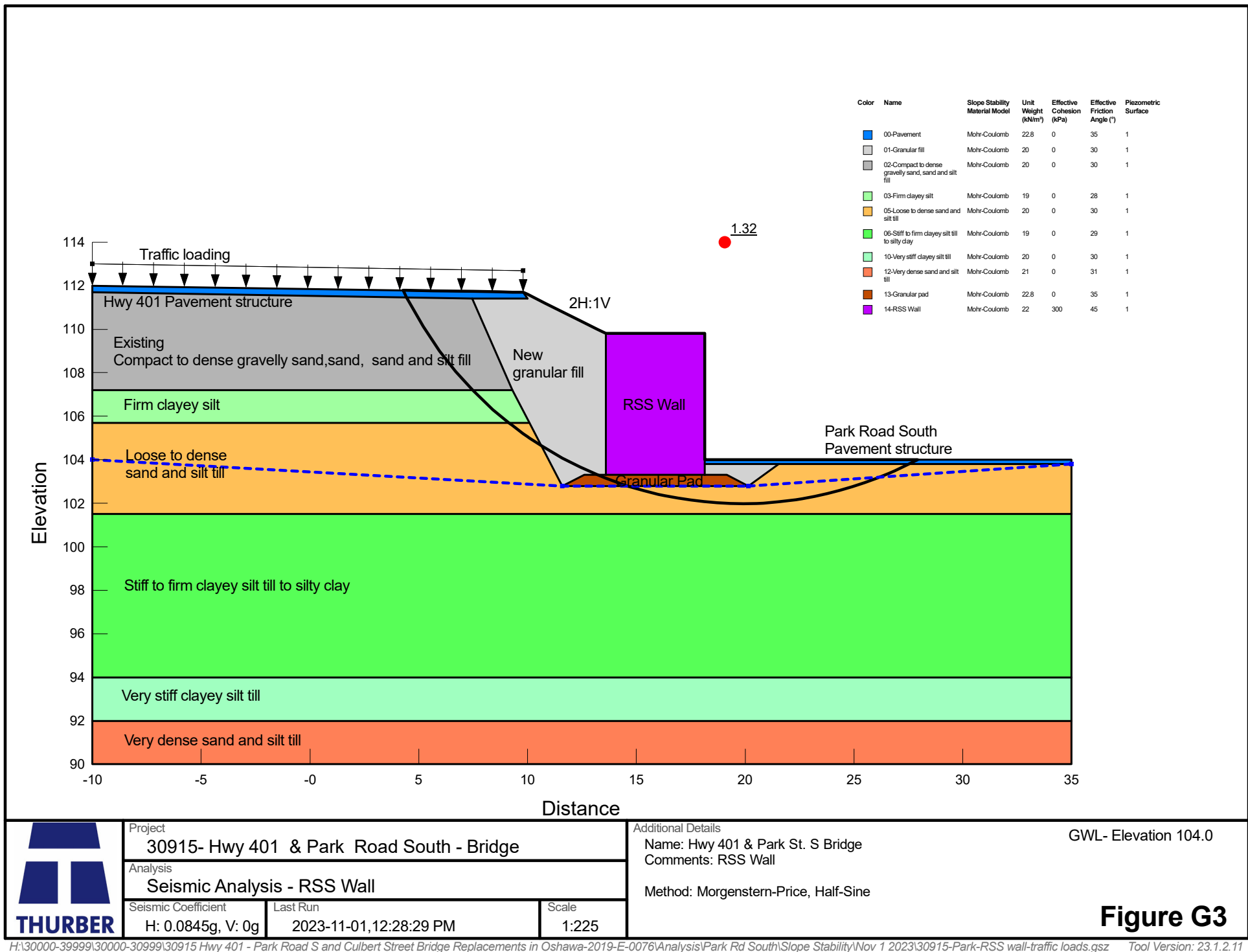
Foundation Element	Spread Footings	Caissons	Driven Piles
	<p>Advantages:</p> <ul style="list-style-type: none"> i. Ease of construction in areas where competent soils (e.g. 100-blow soils) are present at shallow depths. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. The presence of the less competent firm to stiff clays below the proposed founding elevation reduces the available geotechnical resistances, thus necessitating wider footing widths ii. Dewatering will be required to maintain the subgrade in the dry during construction iii. Will likely increase requirements for roadway protection. iv. At some locations along the footing alignments, the founding surface may need to be re-established using engineered fill or mass concrete. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Construction could continue in freezing weather. ii. Higher axial resistance than H-pile. iii. Higher lateral resistance is available due to larger diameter. iv. Favourable for founding the EBL bridge on more competent soils below the firm to stiff clays v. Less number of caissons is required for each foundation element than if steel piles were used. vi. Minimal disruption to traffic particularly at the pier since pile caps are not required. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Specialized installation measures such as temporary liners and drilling water/mud required to install caissons under the water table to minimize sidewall sloughing and water seepage. ii. Potential basal instability if water-bearing soils are exposed at the base. iii. Potential difficulty in cleaning and inspecting bases. iv. Not suitable for integral abutments. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Comparatively short abutment stem possible. ii. Permits integral abutment design. iii. Favourable for founding the EBL bridge on more competent soils below the firm to stiff clays iv. Ease of construction. v. Minimal excavation and dewatering required. vi. Construction could continue in freezing weather. vii. Readily installed. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Very dense soils at shallow depth may limit length of pile at some locations. ii. Cobbles and boulders may be encountered in glacially derived soils that could impede pile penetration to required depths. iii. Relatively lower lateral resistance than caissons is available given the pile dimension. iv. Larger number of piles will likely be required to resist foundation loads.
ABUTMENTS	<p style="text-align: center;">FEASIBLE (near the southerly portion of the EBL bridge, the spread footings may be founded on a pad of engineered fill)</p>	<p style="text-align: center;">FEASIBLE (If non-integral abutments are considered in the bridge design)</p>	<p style="text-align: center;">RECOMMENDED (If integral abutments are considered in the bridge design)</p>

APPENDIX G

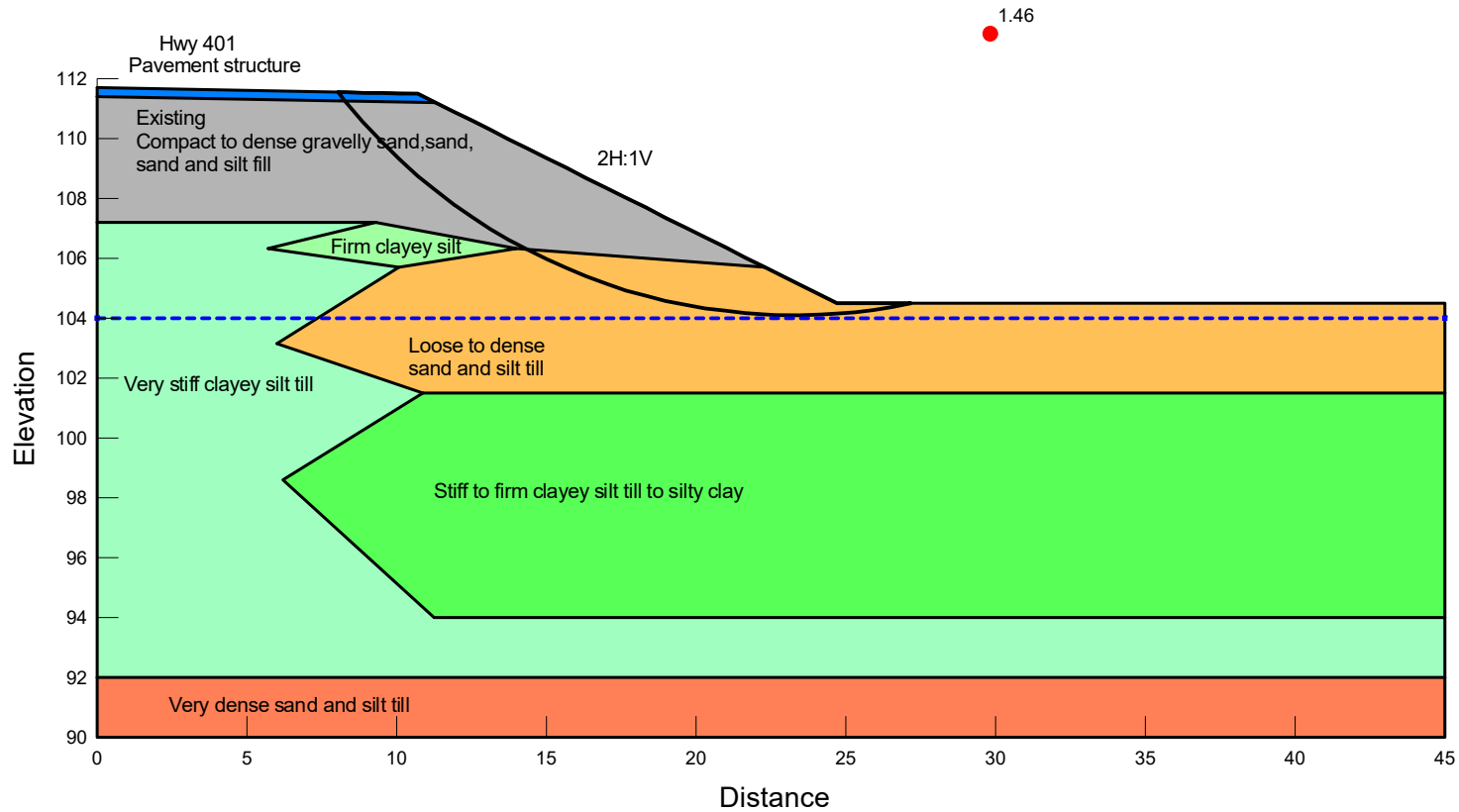
Selected Embankment Stability Output







Color	Name	Slope Stability Material Model	Unit Weight (kN/m ³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Surface
Blue	00-Pavement	Mohr-Coulomb	22.8	0	35	1
Grey	02-Compact to dense gravelly sand, sand and silt fill	Mohr-Coulomb	20	0	30	1
Light Green	03-Firm clayey silt	Mohr-Coulomb	19	0	28	1
Orange	05-Loose to dense sand and silt till	Mohr-Coulomb	20	0	30	1
Green	06-Stiff to firm clayey silt till to silty clay	Mohr-Coulomb	19	0	29	1
Light Green	10-Very stiff clayey silt till	Mohr-Coulomb	20	0	30	1
Red	12-Very dense sand and silt till	Mohr-Coulomb	21	0	31	1

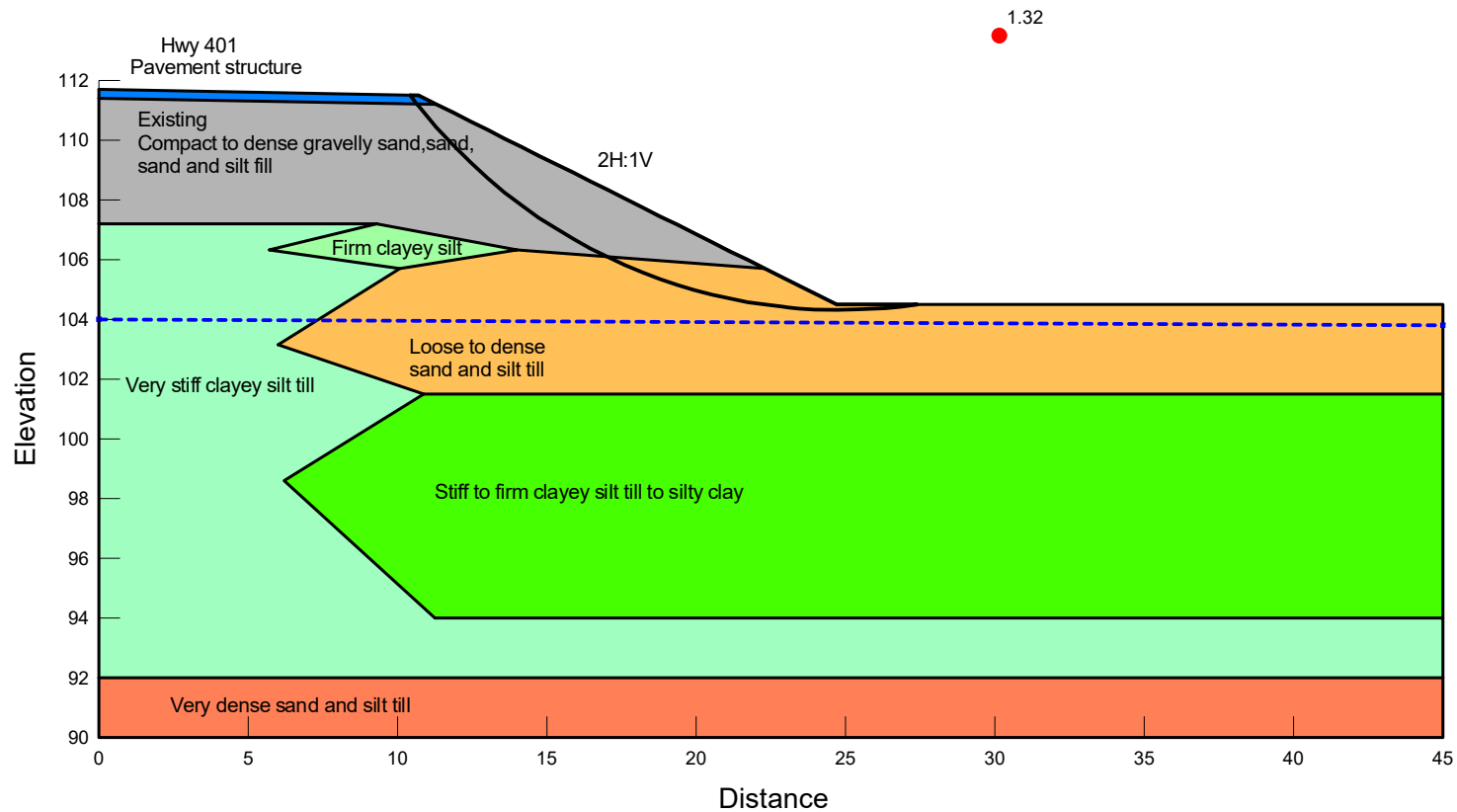


Project 30915- Hwy 401 & Park Road South - Bridge		
Analysis Drained Analysis - Highway 401 Widening - Earth Cut		
Seismic Coefficient H: 0g, V: 0g	Last Run 2024-12-10, 01:49:32 PM	Scale 1:250

Additional Details Name: Hwy 401 & Park Road South- Bridge	GWL- Elevation 104.0
Method: Morgenstern-Price, Half-Sine	

Figure G4

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Total Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Surface
Blue	00-Pavement	Mohr-Coulomb	22.8		0	35	1
Grey	02-Compact to dense gravelly sand, sand and silt fill	Mohr-Coulomb	20		0	30	1
Light Green	04-Firm clayey silt-Undrained	Undrained (Phi=0)	19	25			1
Orange	05-Loose to dense sand and silt till	Mohr-Coulomb	20		0	30	1
Bright Green	07-Stiff to firm clayey silt till to silty clay-Undrained	Undrained (Phi=0)	19	55			1
Light Green	11-Very stiff clayey silt till -undrained	Undrained (Phi=0)	20	120			1
Reddish Orange	12-Very dense sand and silt till	Mohr-Coulomb	21		0	31	1

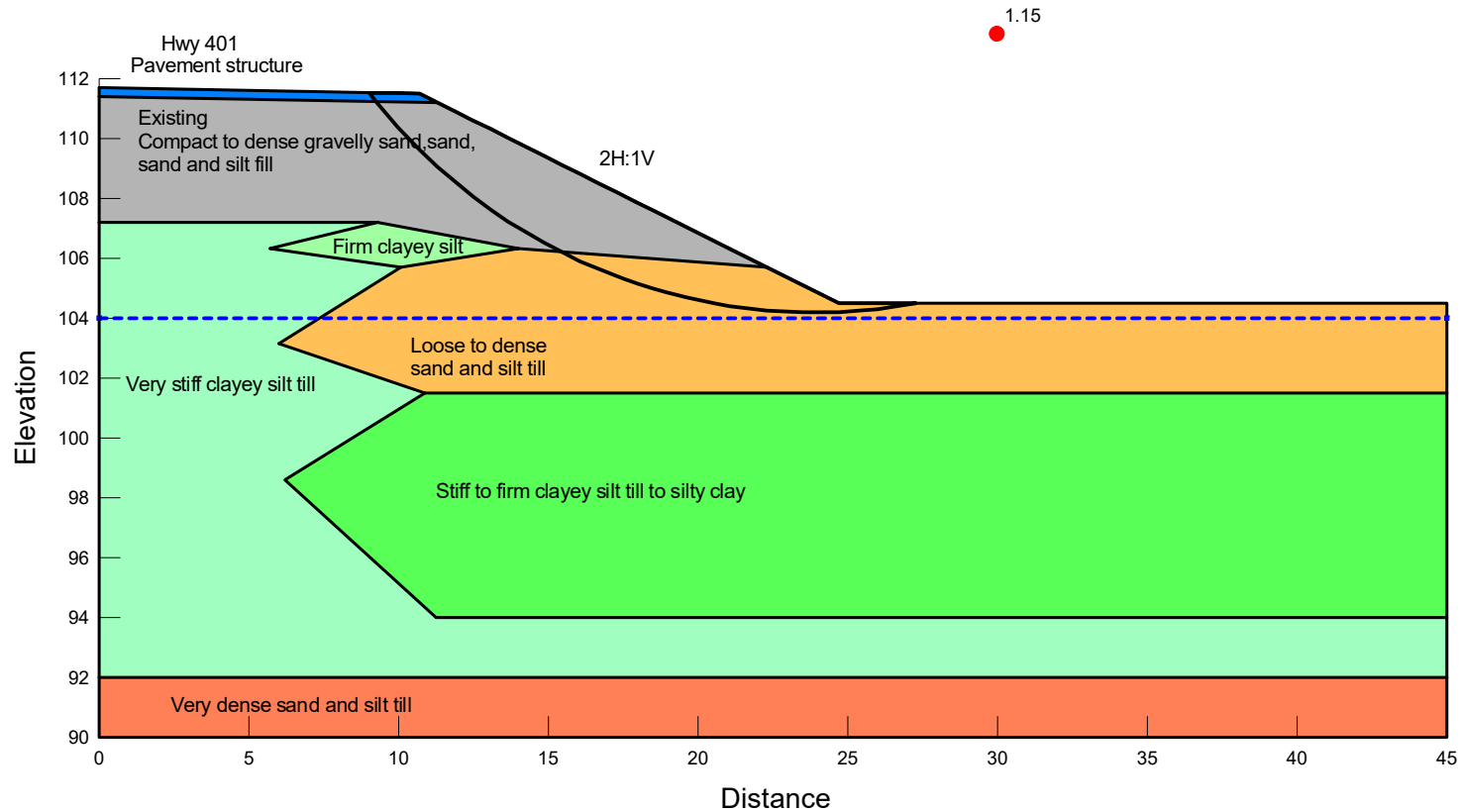


Project 30915- Hwy 401 & Park Road South - Bridge		
Analysis Undrained Analysis - Highway 401 Widening - Earth Cut		
Seismic Coefficient H: 0g, V: 0g	Last Run 2024-12-10,01:49:32 PM	Scale 1:250

Additional Details Name: Hwy 401 & Park Road South- Bridge	GWL- Elevation 104.0
Method: Morgenstern-Price, Half-Sine	

Figure G5

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Surface
Blue	00-Pavement	Mohr-Coulomb	22.8	0	35	1
Grey	02-Compact to dense gravelly sand, sand and silt fill	Mohr-Coulomb	20	0	30	1
Light Green	03-Firm clayey silt	Mohr-Coulomb	19	0	28	1
Orange	05-Loose to dense sand and silt till	Mohr-Coulomb	20	0	30	1
Green	06-Stiff to firm clayey silt till to silty clay	Mohr-Coulomb	19	0	29	1
Light Green	10-Very stiff clayey silt till	Mohr-Coulomb	20	0	30	1
Red	12-Very dense sand and silt till	Mohr-Coulomb	21	0	31	1

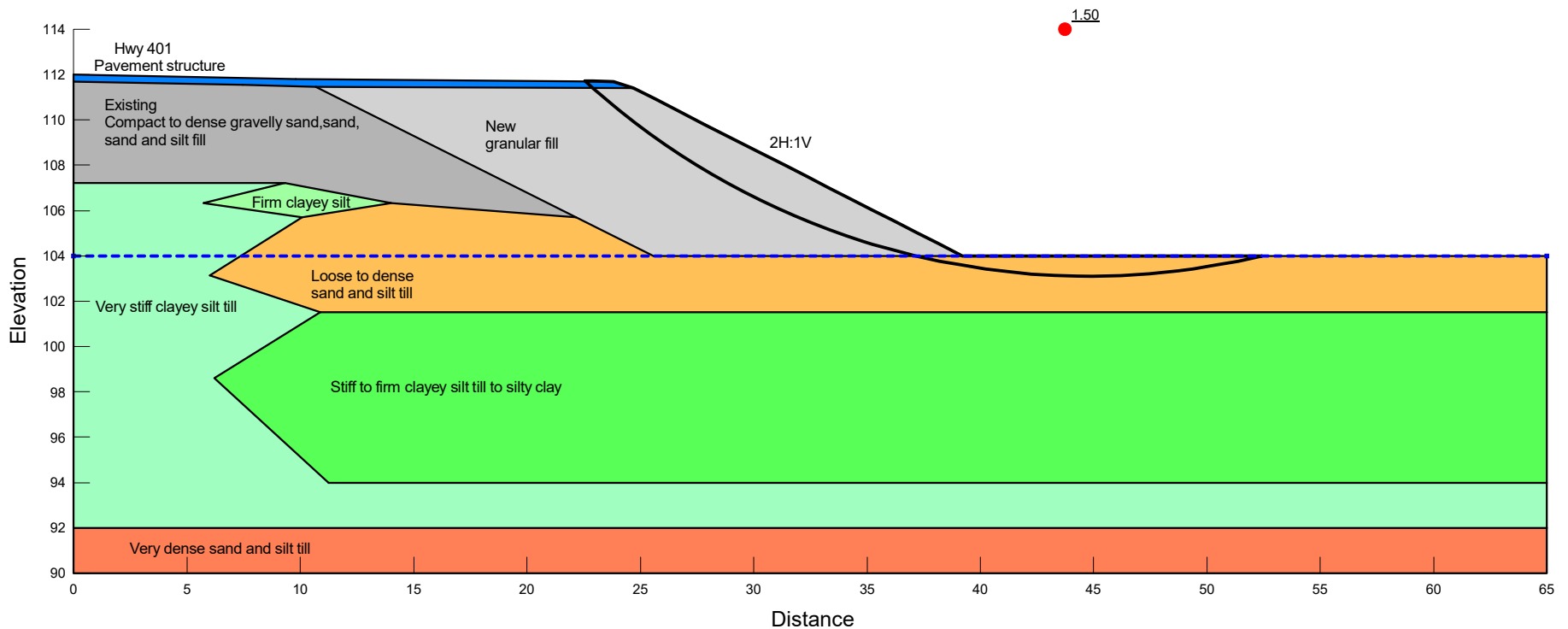


Project 30915- Hwy 401 & Park Road South - Bridge		
Analysis Seismic Analysis - Highway 401 Widening - Earth Cut		
Seismic Coefficient H: 0.0845g, V: 0g	Last Run 2024-12-10,01:49:32 PM	Scale 1:250

Additional Details Name: Hwy 401 & Park Road South- Bridge	GWL- Elevation 104.0
Method: Morgenstern-Price, Half-Sine	

Figure G6

Color	Name	Slope Stability Material Model	Unit Weight (kN/m ³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Surface
■	00-Pavement	Mohr-Coulomb	22.8	0	35	1
■	01-Granular fill	Mohr-Coulomb	20	0	30	1
■	02-Compact to dense gravelly sand, sand and silt fill	Mohr-Coulomb	20	0	30	1
■	03-Firm clayey silt	Mohr-Coulomb	19	0	28	1
■	05-Loose to dense sand and silt till	Mohr-Coulomb	20	0	30	1
■	06-Stiff to firm clayey silt till to silty clay	Mohr-Coulomb	19	0	29	1
■	10-Very stiff clayey silt till	Mohr-Coulomb	20	0	30	1
■	12-Very dense sand and silt till	Mohr-Coulomb	21	0	31	1

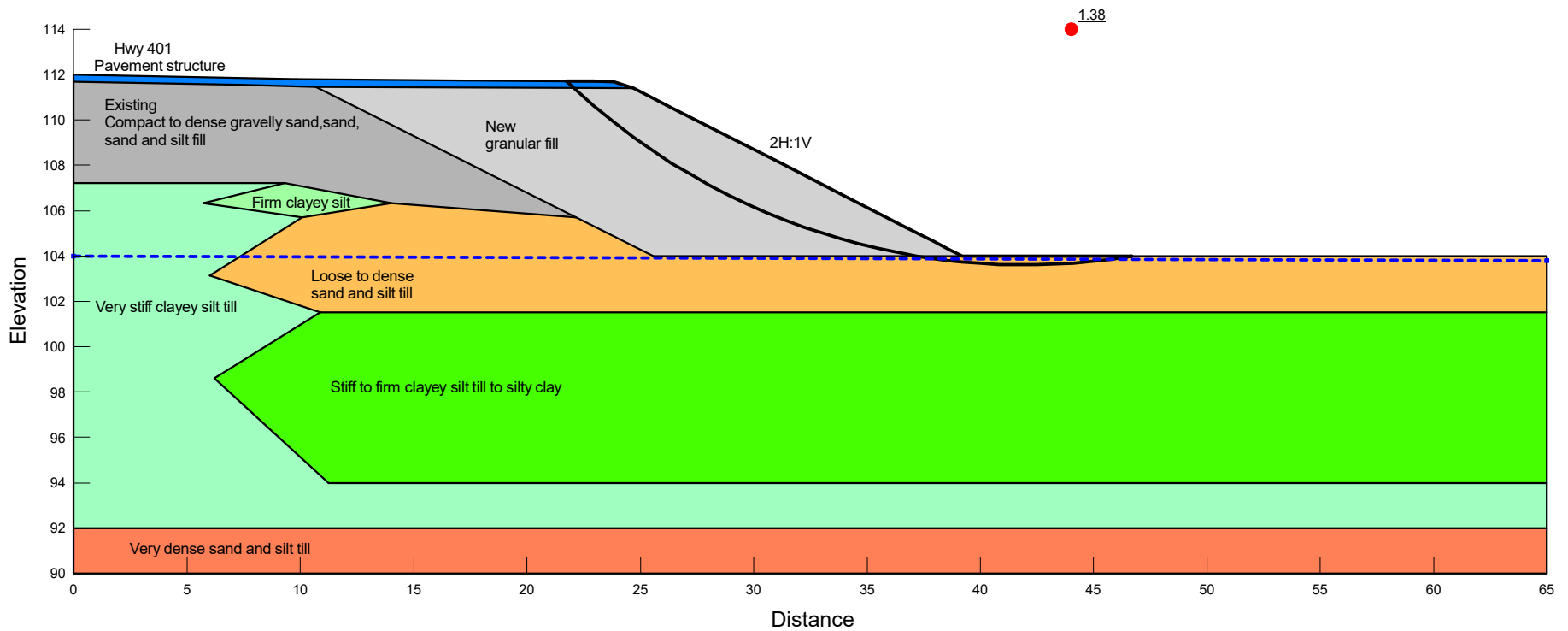


Project 30915- Hwy 401 & Park Road South - Bridge		
Analysis Drained Analysis - Highway 401 Widening		
Seismic Coefficient H: 0g, V: 0g	Last Run 2023-11-01,01:15:00 PM	Scale 1:285

Additional Details Name: Hwy 401 & Park Road South- Bridge	GWL- Elevation 104.0
Method: Morgenstern-Price, Half-Sine	

Figure G7

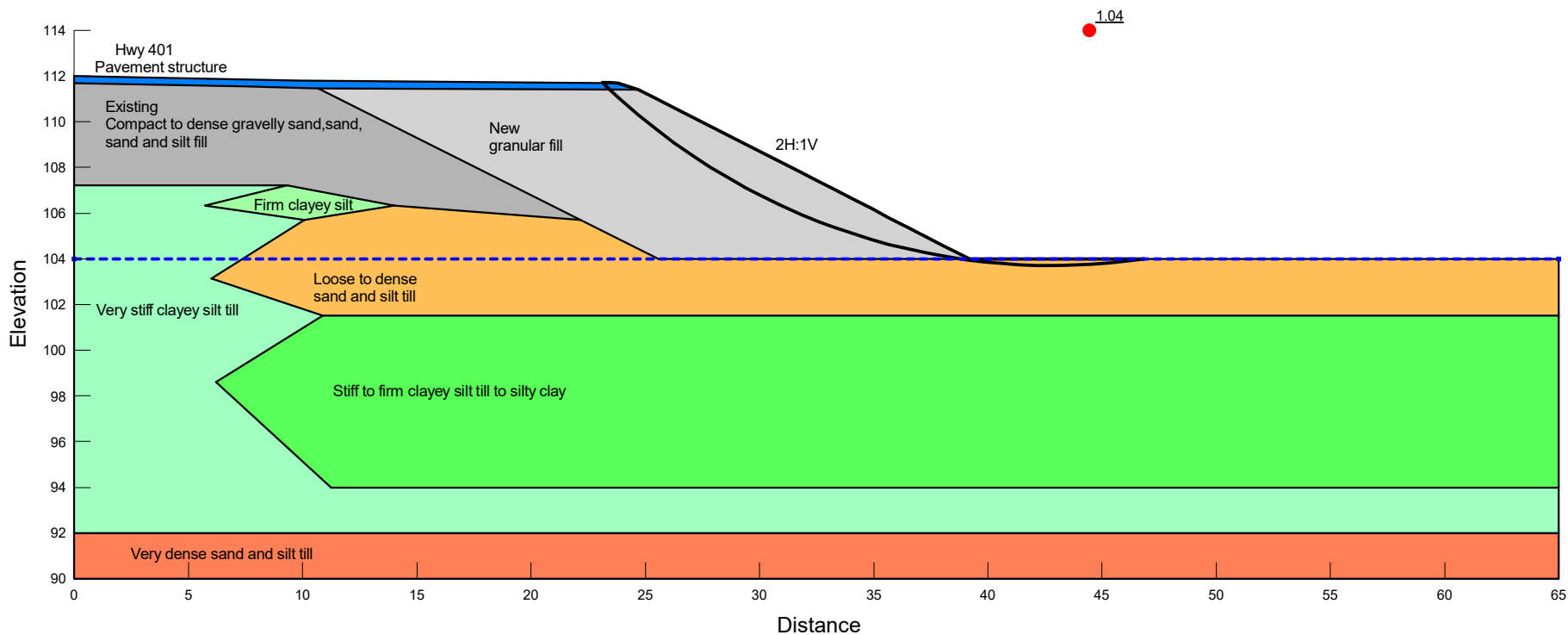
Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Total Cohesion (kPa)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Surface
Blue	00-Pavement	Mohr-Coulomb	22.8		0	35	1
Grey	01-Granular fill	Mohr-Coulomb	20		0	30	1
Grey	02-Compact to dense gravelly sand, sand and silt fill	Mohr-Coulomb	20		0	30	1
Light Green	04-Firm clayey silt-Undrained	Undrained (Phi=0)	19	25			1
Orange	05-Loose to dense sand and silt till	Mohr-Coulomb	20		0	30	1
Green	07-Stiff to firm clayey silt till to silty clay-Undrained	Undrained (Phi=0)	19	55			1
Light Green	11-Very stiff clayey silt till -undrained	Undrained (Phi=0)	20	120			1
Red	12-Very dense sand and silt till	Mohr-Coulomb	21		0	31	1



Project		
30915- Hwy 401 & Park Road South - Bridge		
Analysis		
Undrained Analysis - Highway 401 Widening		
Seismic Coefficient	Last Run	Scale
H: 0g, V: 0g	2023-11-01,01:15:00 PM	1:285

Additional Details		GWL- Elevation 104.0
Name: Hwy 401 & Park Road South- Bridge		
Method: Morgenstern-Price, Half-Sine		
Figure G8		

Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Surface
■	00-Pavement	Mohr-Coulomb	22.8	0	35	1
■	01-Granular fill	Mohr-Coulomb	20	0	30	1
■	02-Compact to dense gravelly sand, sand and silt fill	Mohr-Coulomb	20	0	30	1
■	03-Firm clayey silt	Mohr-Coulomb	19	0	28	1
■	05-Loose to dense sand and silt till	Mohr-Coulomb	20	0	30	1
■	06-Stiff to firm clayey silt till to silty clay	Mohr-Coulomb	19	0	29	1
■	10-Very stiff clayey silt till	Mohr-Coulomb	20	0	30	1
■	12-Very dense sand and silt till	Mohr-Coulomb	21	0	31	1

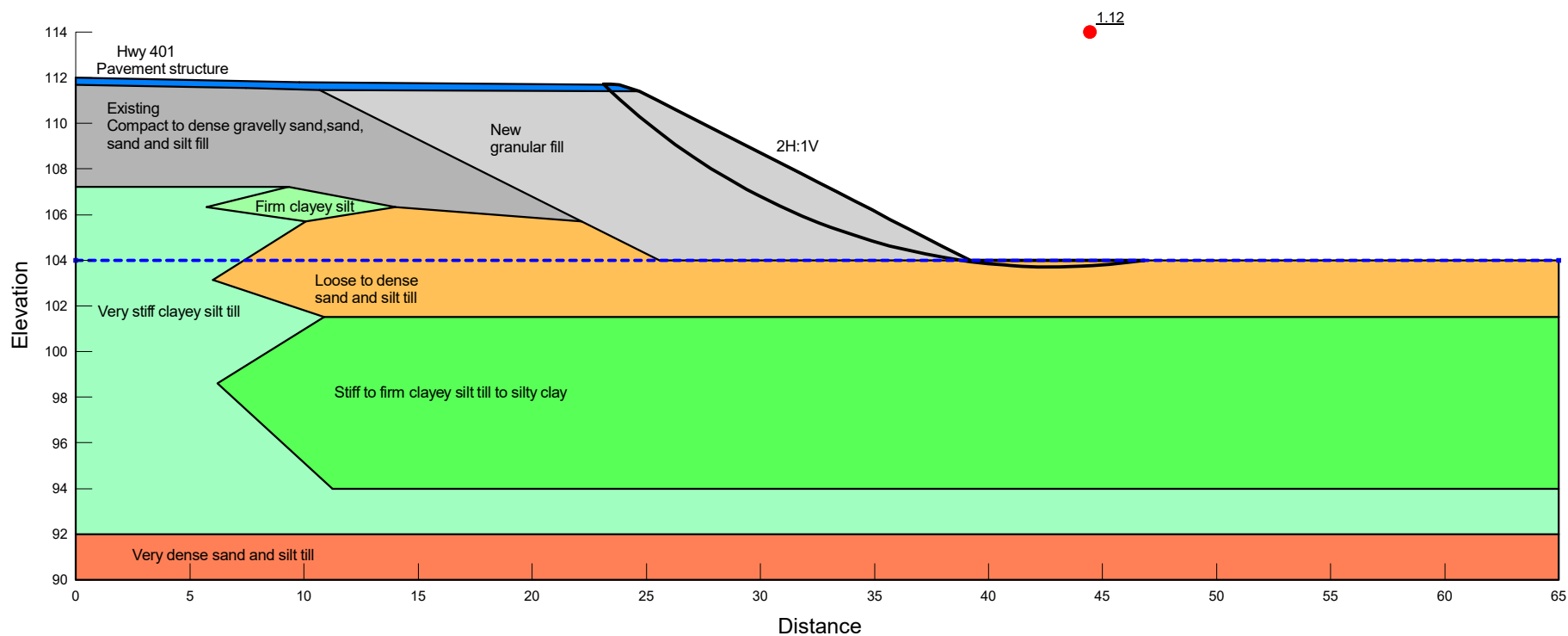


Project		
30915- Hwy 401 & Park Road South - Bridge		
Analysis		
Seismic Analysis - Highway 401 Widening		
Seismic Coefficient	Last Run	Scale
H: 0.0845g, V: 0g	2023-11-01,01:15:00 PM	1:285

Additional Details
Name: Hwy 401 & Park Road South- Bridge
GWL- Elevation 104.0
Method: Morgenstern-Price, Half-Sine

Figure G9

Color	Name	Slope Stability Material Model	Unit Weight (kN/m ³)	Effective Cohesion (kPa)	Effective Friction Angle (°)	Piezometric Surface
■	00-Pavement	Mohr-Coulomb	22.8	0	35	1
■	01-Granular fill	Mohr-Coulomb	21	0	32	1
■	02-Compact to dense gravelly sand, sand and silt fill	Mohr-Coulomb	20	0	30	1
■	03-Firm clayey silt	Mohr-Coulomb	19	0	28	1
■	05-Loose to dense sand and silt till	Mohr-Coulomb	20	0	30	1
■	06-Stiff to firm clayey silt till to silty clay	Mohr-Coulomb	19	0	29	1
■	10-Very stiff clayey silt till	Mohr-Coulomb	20	0	30	1
■	12-Very dense sand and silt till	Mohr-Coulomb	21	0	31	1



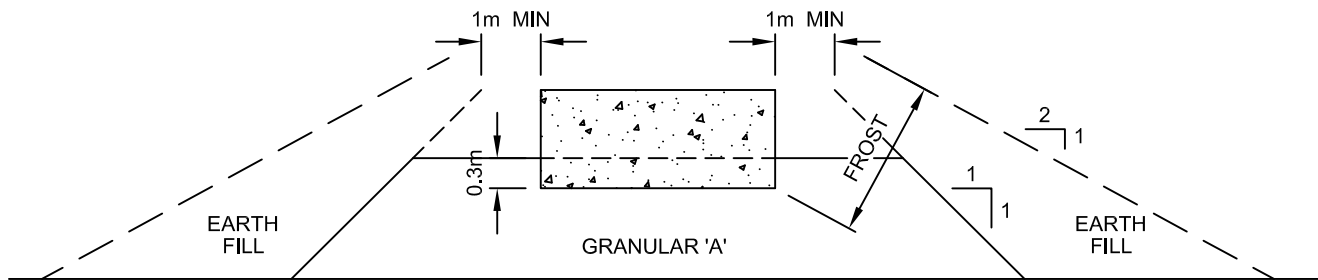
Project 30915- Hwy 401 & Park Road South - Bridge		
Analysis Seismic Analysis - Highway 401 Widening		
Seismic Coefficient H: 0.0845g, V: 0g	Last Run 2023-11-01,02:19:52 PM	Scale 1:285

Additional Details Name: Hwy 401 & Park Road South- Bridge	GWL- Elevation 104.0
Method: Morgenstern-Price, Half-Sine	

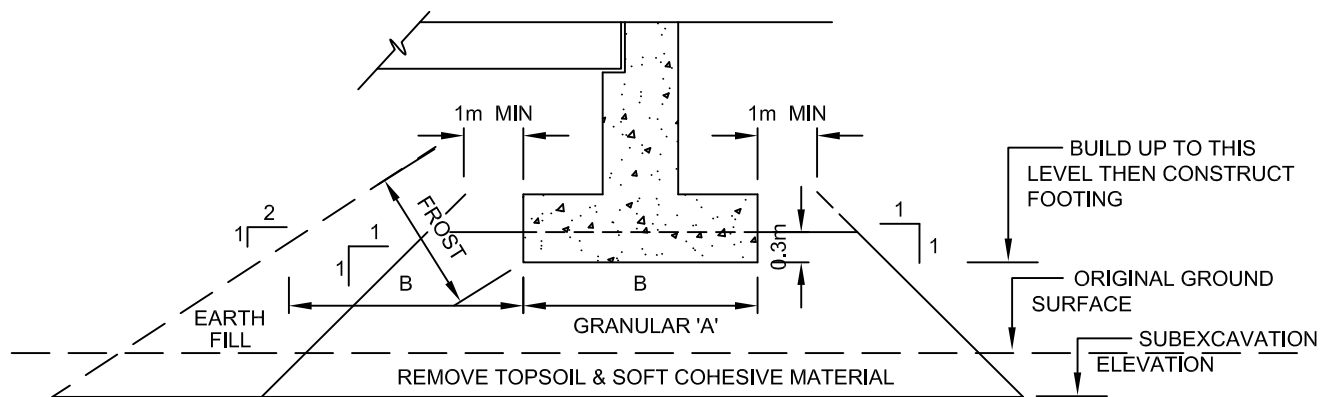
Figure G10

APPENDIX H

Figure – Granular Pad



CROSS-SECTION



LONGITUDINAL SECTION

NOTES:

1. REMOVE TOPSOIL AND OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH FILL.
2. PLACE GRANULAR 'A' AND EARTH FILL TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO O.P.S.S. 501.
3. CONSTRUCT CONCRETE FOOTING.
4. PLACE REMAINDER OF GRANULAR 'A' AND EARTH FILL AS REQUIRED.
5. SOURCE M.T.C. 1982.

ABUTMENT ON COMPACTED FILL
SHOWING GRANULAR 'A' CORE



THURBER ENGINEERING LTD.

ENGINEER :	DRAWN :	APPROVED :
-	MFA	-
DATE :	SCALE :	DRAWING No.
MAY 2024	N.T.S.	FIGURE 1

APPENDIX I

OPS Used in the Report and Suggested Wordings for NSSP



1. List of OPSS and OPSD Referenced in this Report

- OPSS PROV 206 Construction specification for grading
- OPSS.PROV 212 Construction Specification for Earth Borrow
- OPSS PROV 501 Construction specification for compacting
- OPSS.PROV 517 Construction specification for dewatering
- SP 517F01 Amendment to OPSS 517
- OPSS PROV 539 Construction specification for temporary protection systems
- OPSS PROV 804 Construction specification for seed and cover
- OPSS PROV 902 Construction specification for excavating and backfilling – Structures

- OPSS PROV 903 Construction specification for deep foundations
- OPSS PROV 1010 Material specification for aggregates - base, subbase, select subgrade, and backfill material

- OPSD 3102.100 Wall Abutments, backfill drain
- OPSD 3101.150 Wall Abutment, backfill, minimum granular requirement

2. Suggested Text for NSSP on “Spread Footings”

Along the southerly portion of the EBL (south) bridge footprint, existing boreholes revealed the presence of less competent firm to stiff clayey silt/silty clay and clayey silt till. Once identified by subgrade inspection during construction, sub-excavation is required to remove these unsuitable materials from below the design founding level up to about 1 m depth to expose the underlying very stiff to hard clayey silt till. The founding surface should be re-established using a compacted Granular A pad or mass concrete.

3. Suggested Text for NSSP on “Installation of H-Piles”

All H-piles shall be installed in accordance with OPSS 903. Very dense/hard glacial tills were encountered at shallow depths at this site.



The Contractor is alerted that there are risks of encountering obstructions such as cobbles, boulders and other man-made debris within the embankment fill and native soils. Such obstructions and hard/very dense zones in the soils can impede pile penetration to the design pile tip elevations. Pile driving must be controlled according to the criteria specified for the site. Pre-augering may be required at some pile locations prior to driving the piles. Each pre-augered hole should not be greater than 200 mm in diameter and reverse augering should be carried out as the auger is retrieved to leave as much soil cuttings as possible inside the hole.

Should a pile achieve the design ultimate geotechnical resistance or refusal at an elevation higher than that indicated in the contract, the Contract Administrator (CA) shall be informed immediately who should consult with the design team for resolution. Over-driving must be avoided to minimize the risk of damaging the pile.

4. Suggested Text for NSSP on Groundwater Control

The groundwater table is approximately 1 m above the highest recommended footing base level, which is up to 3 m below the perched water level in the sand and silt till. Groundwater control measures such as perimeter ditches and pumping from filtered sumps will need to be employed to remove any accumulation of water from the excavation base prior to placing concrete. In addition to effective sump pumping, other measures of groundwater control including the localized use of well points may be required where the excavations extend below the groundwater table in order to maintain a reasonably dry subgrade for construction. Filtered sumps must be designed properly so that construction drainage water containing eroded soil and fines do not flow onto the existing roadways. It is also important to minimize disturbance of the exposed sand, and sand and silt fill surfaces by limiting construction traffic. Dewatering in conjunction with temporary protection (shoring) will be required for groundwater control due to seepage from the sand and silt till and other water-bearing layers, accumulation of precipitation and surface runoff.

5. Suggested Text for NSSP on “Impact on Adjacent Structures and Embankments”

It is critical that the Contractor’s excavation and construction activities do not undermine or have any adverse impact on the integrity and performance of the following adjacent structures and embankments:

- The operating lanes of the Highway 401 during excavation and foundation construction for the new abutments.
- Protection of the existing structure foundations and utilities during construction. Relocation of the existing watermain below Park Road South will be required prior to new bridge construction.
- Protection of existing approach embankments.



6. Suggested Text for NSSP on “Preloading”

Preloading shall be carried out for one (1) month at the southwest quadrant to induce ground settlement. The preloading shall be co-ordinated with the staging plans currently included in the contract. The Contractor shall not proceed with final granular placement and paving until approval has been given by the Contract Administrator.

7. Suggested Text for NSSP on “Granular Pad”

The Granular A for the engineered fill pad must be compacted to 100% Standard proctor maximum dry density (SPMDD) at optimum moisture content $\pm 2\%$, and placed in 150 mm lifts.