



THURBER ENGINEERING LTD.

**FOUNDATION INVESTIGATION AND DESIGN REPORT
SUNNIDALE ROAD UNDERPASS
HIGHWAY 400
CITY OF BARRIE, ONTARIO
SITE 30-173
G.W.P. 2445-15-00**

GEOCRES NO. 31D-771

**Latitude:44.392498°
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**Report
to
McIntosh Perry**

Date: October 12, 2021
File: 22424



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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 400 Sunnidale Road underpass structure located in the City of Barrie, 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 and selected data from previous investigations by others, to describe the geotechnical conditions influencing design and construction of the foundations and approach embankments for the structure.

Thurber was retained by McIntosh Perry (MP) to carry out this foundation investigation under the Ministry of Transportation Ontario (MTO) Assignment Number 2017-E-0032. The overall assignment includes reconstruction of the Dunlop Street and Highway 400 interchange, and replacement of the Highway 400 at Dunlop Street and Anne Street underpass structures. This report addresses the proposed replacement of the Sunnidale Road underpass structure.

Reference has been made to information on subsurface conditions contained in a previous foundation report prepared by others for this site. The title of this report is:



- Preliminary Foundation Investigation Report, Sunnidale Road Underpass, Site 30-173, Highway 400 Widening, from 1 Km South of Highway 89 to Junction of Highway 11, Ministry of Transportation, Ontario, W.O. 06-20016, prepared by Golder Associates Ltd., GEOCREs No. 31D-665, dated October 20, 2016. (Reference 1).

2 SITE DESCRIPTION

The existing Sunnidale Road underpass structure is located at the intersection of Highway 400 and Sunnidale Road in Barrie, Ontario. The underpass structure runs in a east-west direction and carries two lanes of Sunnidale Road traffic over Highway 400. Based on the General Arrangement (GA) drawing, the existing underpass consists of a single-span reinforced concrete rigid frame supported on spread footings. The total length of the bridge is approximately 28.8 m between abutments, and the width is about 11.8 m. The bridge is at an approximate 5° skew to the centreline of Highway 400. There is also a 1.8 m wide sidewalk with steel railing along the north side of the structure. There are approach slabs at both ends of the structure. The length of the west and east approach slabs are 7.0 m and 6.8 m, respectively.

The overall surface topography in the vicinity of the site is relatively flat and consists of residential and commercial properties to the east and west of Highway 400. At the structure site, Highway 400 was constructed in a cut of up to approximately 8.5 m deep. The existing grade of Highway 400 is at approximate Elevation 248.5. The Sunnidale Road grade rises from approximate Elevations 254.0 to 257.6, east to west.

Photographs of the site, taken during the course of the investigation, are presented in Appendix D.

Based on published geological mapping, the study area is located within the Simcoe Lowlands physiographic region. This region borders Georgian Bay and Lake Simcoe and can generally be separated into two major divisions: the Nottawasaga basin to the west, consisting of plains draining into Nottawasaga Bay, and the Lake Simcoe basin to the east, consisting of the lowlands which surround Lake Simcoe. These two basins are connected at Barrie by a flat- floored valley which extends from the shores of Kempenfelt Bay. The Simcoe Lowlands region is generally comprised of sand, silt and clay deposits of deltaic and lacustrine origin.



3 SITE INVESTIGATION AND FIELD TESTING

The current borehole investigation and field testing program for this site was carried out between January 28 and July 20, 2019, and consisted of drilling and sampling five (5) boreholes, designated as Boreholes SUN-01 to SUN-05. Three boreholes (SUN-01, SUN-02 and SUN-05) were drilled near the locations of the proposed foundation elements (abutments and pier) and terminated at depths ranging from 24.7 m to 24.8 (Elevations 223.7 to 232.7). The other two boreholes (SUN-03 and SUN-04) were drilled for the immediate approaches and terminated at 9.5 m to 9.8 m depths (Elevations 244.6 and 247.9). The records of borehole sheets of the present investigation are provided in Appendix A.

A geotechnical investigation was carried out at this site on February 6 and 7, 2001 (Reference 1), and consisted of advancing two boreholes (labelled B13-1 and B13-2). Boreholes B13-1 and B13-2 were drilled near the existing west and east abutments, respectively. The depths of the boreholes were 12.4 m and 13.4 m (Elevations 244.4 and 241.7). The Record of Borehole sheets for the boreholes from this previous investigation are included in Appendix C.

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

McIntosh Perry surveyed the boreholes in the field using a combination of GPS and total station equipment, and provided Thurber with the borehole coordinates and ground surface elevations. It is understood that the horizontal and vertical accuracy of the survey results meet the MTO terms of reference requirements of 0.5 m and 0.1 m, respectively. The coordinates and elevations of the boreholes are given on the individual Record of Borehole Sheets and the drawings in Appendices A and E, respectively.

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 casing. 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.



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 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 standpipe piezometers (25 mm diameter) were installed and enclosed in filter sand in selected boreholes to permit groundwater level monitoring. One standpipe piezometer was installed in Borehole B13-2 during the 2001 investigation. The details of the piezometers are shown in Table 3.1.

Table 3.1 – Borehole Completion Details

Foundation Unit	Borehole	Borehole Depth / Base Elevation (m)	Piezometer Tip Depth/ Elevation (m)	Completion Details
West Abutment	SUN-01	24.7 / 232.7	24.4 / 233.0	Piezometer with 3.0 m slotted screen installed with sand filter from 24.7 m to 18.3 m. Borehole caved to 17.7 m, then backfilled with bentonite holeplug from 17.7 m to ground surface.
East Abutment	SUN-05	24.7 / 227.9	12.2 / 240.4	Borehole caved to 12.2 m. Piezometer with 3.0 m slotted screen installed with sand filter from 12.2 m to 8.8 m, bentonite holeplug from 8.8 m to 7.9 m, then auger cuttings from 7.9 m to ground surface.
	B13-2 ⁽¹⁾	13.4 / 241.7	13.1 / 242.0	Piezometer installed with sand filter from 13.4 m to 10.6 m, bentonite holeplug from 10.6 m to 0.4 m, then concrete from 0.4 m to ground surface.

(1) Borehole drilled during the previous investigation (Reference 1). Borehole backfilled description was interpreted from Record of Borehole Sheet (B13-2).



All boreholes without standpipe piezometer were backfilled upon completion of drilling in general accordance with O.Reg. 903. Once the final readings are taken, the two piezometers from the current investigation will be decommissioned in general accordance with O.Reg. 903. Asphalt was reinstated in boreholes drilled on the highway or road platform (Boreholes SUN-02 to SUN-04).

4 LABORATORY TESTING

The recovered soil samples were subjected to visual identification (VI) and to natural moisture content determination. Selected samples were subjected to grain size distribution analyses (sieve and/or hydrometer), and Atterberg Limits testing. Geotechnical laboratory testing results of the current investigation, are summarized on the Record of Borehole sheets included in Appendix A and are presented on the figures included in Appendix B. Laboratory tests results from the previous investigation (Reference 1) are included 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 structure, samples of the existing fills and native sands and silts were collected and 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 6 and are presented in Appendix B.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

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

In general, the subsurface stratigraphy encountered at the site consists of pavement structure or topsoil overlying embankment fill which is underlain by an upper layer of silty sand to sand and silt till, and layers of sand, silty sand and sandy silt. Interbedded layers of hard silty clay/clayey silt and clayey silt till were encountered within the cohesionless soils. A lower deposit of very dense silty sand till with occasional lenses of gravelly sand were encountered in Borehole SUN-01. The groundwater level was observed to be at approximately 9 m to 11 m depths below the Sunnidale Road grade and in the order of 2 m to 5 m below the Highway 400 grade.



More detailed descriptions of the individual stratum are presented below.

5.1 Topsoil

A layer of topsoil was encountered surficially in Boreholes SUN-01 and SUN-05 with measured thicknesses of 700 mm and 200 mm, respectively.

The natural moisture contents measured on samples of the topsoil were 9 percent and 24 percent.

An SPT 'N' value of 11 blows per 0.3 m penetration was measured in Borehole SUN-01 indicating a compact condition.

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

5.2 Pavement Structure

Pavement structure consisting of approximately 100 mm to 200 mm of asphalt overlying granular (sand and gravel fill, sand fill) road base was encountered in the boreholes advanced through the Sunnidale Road platform (Boreholes SUN-03 and SUN-04), and the Highway 400 platform (Borehole SUN-02). The granular fill ranged in thickness from 1.0 m to 1.2 m. In Boreholes B13-1 and B13-2, the asphalt was approximately 200 mm thick, but a road base thickness was not identified.

SPT 'N' values recorded in the sand and gravel fill and sand fill ranged from 8 to 45 blows per 0.3 m of penetration indicating a loose to dense condition. The moisture contents measured on samples of the road base fill ranged from 2 percent to 7 percent.

The results of grain size analyses conducted on samples of the granular road base fill are provided on the Record of Borehole sheets in Appendix A, and illustrated on Figure B1 Appendix B. The results are summarized as follows:

Soil Particle	Granular Road Base Fill (Percent)
Gravel	9
Sand	79
Silt and Clay	12



5.3 Embankment Fill

Embankment fill was encountered underlying the pavement structure in the boreholes advanced through the Sunnidale Road platform (Boreholes SUN-03, SUN-04, B13-1 and B13-2), and below the topsoil in Borehole SUN-01.

The embankment fill typically consisted of layers of brown sand and silt, sand and gravel, and sand, containing some gravel and trace to some clay. The fill also contained occasional cobbles. A 1.1 m thick layer of silty clay fill was contacted in Borehole SUN-03 at 1.1 m depth.

The overall thickness of the embankment fill varied from 1.1 m to 3.4 m in Boreholes SUN-01, SUN-03 and SUN-04. In Boreholes B13-1 and B13-2, the embankment fill was 7.1 m and 6.7 m thick, respectively.

The SPT 'N' values recorded in the cohesionless embankment fill ranged from 9 to 62 blows per 0.3 m of penetration indicating a loose to very dense condition. SPT 'N' values of 8 and 11 blows per 0.3 m of penetration were measured in the silty clay fill indicating a firm to stiff consistency.

The natural moisture contents measured on samples of the cohesionless fill generally ranged from 9 percent to 21 percent. Moisture contents of 8 to 12 percent were measured in the silty clay fill.

The results of a grain size analyses conducted on samples of the sand and silt fill are provided on the Record of Borehole sheets in Appendix A, and illustrated on Figure B2 Appendix B. The results are summarized as follows:

Soil Particle	Embankment Fill (Percent)
Gravel	1 to 12
Sand	45 to 57
Silt	40
Clay	14
Silt and Clay	13

5.4 Upper Silty Sand to Sand and Silt Till

An upper layer of native brown silty sand to sand and silt till containing trace gravel, trace clay and occasional cobbles was contacted below the fill at depths ranging from 1.4 m to 5.6 m, and at 0.2 m below the topsoil in Borehole SUN-5. The thickness of this till varied from 2.7 m to



11.4m. The depth to the base of this upper till ranged from 4.0 m to 16.3 m (Elevations 248.6 to 241.1).

Borehole SUN-04 was terminated within the upper silty sand till at 9.8 m depth (Elevation 247.9).

The SPT 'N' values recorded in the upper silty sand till and sand and silt till layer ranged from 17 to over 100 blows per 0.3 m of penetration indicating a 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 SUN-05 just below the topsoil. The natural moisture contents measured on samples of cohesionless till ranged from 6 percent to 19 percent.

The results of grain size distribution analyses carried out on selected samples of the upper silty sand till and sand and silt till are shown on Figure B3 in Appendix B, and Figure 1 in Appendix C. The results are summarized as follows:

Soil Particle	Silty Sand Till to Sand and Silt Till (Percent)
Gravel	4 to 7
Sand	50 to 59
Silt	26 to 39
Clay	3 to 9

Glacial tills inherently contain cobbles and boulders. Auger grinding was noted in the till in Borehole SUN-01.

5.5 Gravelly Sand

An 800-mm thick layer of brown gravelly sand was contacted at 12.2 m depth in Borehole SUN-01.

An SPT 'N' value measured in the gravelly sand was 100 blows per 0.225 m of penetration, indicating a very dense state. The moisture content measured in the gravelly sand was 9 percent.

5.6 Sand to Silty Sand

Layers of brown sand to silty sand containing trace to some gravel, trace to some silt and trace clay were contacted at depths ranging from 3.0 m to 7.3 m in Boreholes SUN-02 to SUN-05, B13-1 and B13-2, and at 16.3 m in Borehole SUN-01. The thickness of the sand to silty sand



ranged from 0.7 m to 7.6 m.

The depth to the base of the sand to silty sand varied from 5.6 m to and 24.0 m (Elevations 252.0 to 233.4) in Boreholes SUN-01, SUN-02, SUN-04 and B13-1.

Boreholes SUN-03, SUN-05 and B13-2 were terminated within the sand to silty sand layer at depth varying from 9.5 m to 24.7 m (Elevations 244.6 to 227.9).

The SPT 'N' values recorded in the sand varied from 13 to 134 blows per 0.3 m of penetration to greater than 100 blows for 0.175 m of penetration indicating compact to very dense condition. The natural moisture contents measured on the sand/silty sand samples ranged from 2 percent to 33 percent.

The results of grain size analyses conducted on sand samples are provided on the Record of Borehole sheets in Appendix A, and illustrated on Figures B4 and B5 of Appendix B, and Figure 2 of Appendix C. The results are summarized as follows:

Soil Particle	Sand (Percent)	Silty Sand (Percent)
Gravel	0	0
Sand	84 to 97	65 to 76
Silt	7 to 13	22 to 32
Clay	0 to 3	2 to 3
Silt and Clay	3	-

5.7 Silty Clay and Clayey Silt

Interbedded layers of brown to grey silty clay and clayey silt containing trace sand were contacted in the cohesionless soils at 14.8 m and 8.7 m depths in Boreholes SUN-02 and SUN-05, respectively. The thickness of the silty clay/clayey silt was 3.6 m and 7.6 m.

In Borehole B13-1, drilled during the previous investigation, a layer of brown clayey silt till was contacted at 10.5 m depth.

The depth to the base of the silty clay/clayey silt was at 22.4 m and 12.3 m (Elevations 226.2 and 240.3) in Boreholes SUN-02 and SUN-05, respectively. Borehole B13-1 was terminated within the clayey silt till at 12.4 m (Elevation 244.4) depth.

SPT 'N' values in the silty clay/clayey silt ranged from 37 to 100 blows per 0.3 m of penetration, indicating a hard consistency. The SPT 'N' values measured in Borehole B13-1, in the clayey silt



till were 102 blows per 0.3 m of penetration and 100 blows per 0.18 m of penetration, indicating a hard consistency. Moisture contents measured in the silty clay/clayey silt/clayey silt till ranged from 18 percent to 35 percent in the boreholes drilled during the present investigation. Moisture content measured in the clayey silt till in Borehole B13-2 drilled during the previous investigation had a moisture content of 9 percent.

The results of grain size distribution analyses carried out on selected samples of the silty clay/clayey silt are presented on the Record of Borehole sheets included in Appendix A. Grain size distribution curves of samples tested during the present investigation are presented on Figure B6 Appendix B. The results of the grain size distribution analyses are summarized below:

Soil Particle	Silty Clay /Clayey Silt (Percent)
Gravel	0
Sand	1
Silt	50 to 77
Clay	22 to 49

The results of Atterberg Limits tests conducted on samples of the silty clay/clayey silt are presented on the Record of Borehole sheets in Appendix A, and illustrated in Figure B8 of Appendix B. The results are summarized as follows:

Index Property	Percentage (%)
Liquid Limit	29 to 41
Plasticity Index	14 to 22

The results of the Atterberg Limits testing indicate that the silty clay/clayey silt is of low to medium plasticity with group symbols of CL and CI.

5.8 Lower Silty Sand Till/Sandy Silt Till

Lower layers of brown silty sand till and sandy silt till containing trace to some gravel and trace clay were contacted in Boreholes SUN-01 and SUN-02 at 20.9 m and 11.7 m depth, respectively, and also at 24.0 m depth in Borehole SUN-01. The thickness of the lower sandy silt till was 3.1 m in Borehole SUN-02.

The depth to the base of the lower silty sand till/sandy silt till was at 14.8 m (Elevation 233.8) in Borehole SUN-02. Borehole SUN-01 was terminated within the lower silty sand till at 24.7 m depth (Elevation 232.7).



SPT 'N' values recorded in the lower silty sand till were greater than 100 blows per 0.175 m, indicating a very dense state. The natural moisture contents measured on the lower silty sand till samples ranged from 19 percent to 22 percent.

Glacial tills inherently contain cobbles and boulders.

5.9 Sandy Silt

Grey sandy silt containing trace clay was contacted at 22.4 m depth in Borehole SUN-02.

Borehole SUN-02 was terminated within the sandy silt at 24.8 m depth (Elevation 223.7).

The SPT 'N' values recorded in the sandy silt deposit ranged from 100 blows per 0.3 m of penetration and 100 blows for 0.25 m of penetration indicating a very dense condition. The natural moisture content measured on samples of the sandy silt was 19 percent.

The results of grain size distribution analyses carried out on the sandy silt are shown on Figure B7 in Appendix B. The results are summarized as follows:

Soil Particle	Silty Sand to Sandy Silt (Percent)
Gravel	0
Sand	22
Silt	73
Clay	5

5.10 Groundwater Conditions

Groundwater levels in the boreholes were observed during the drilling operations and measured upon completion of drilling. Standpipe piezometers were installed in Boreholes SUN-01, SUN-05 and B13-2 to permit monitoring of groundwater levels. Water levels measured in the three installed standpipes and open boreholes from the current investigation are presented in Table 5.1 below.

Table 5.1 - Groundwater Level Measurements

Foundation Unit	Borehole	Date	Groundwater Level		Comments
			Depth (m)	Elev. (m)	
West Abutment	SUN-01	June 18, 2019	-	-	Open borehole (caved to 18.3 m) Piezometer
		July 4, 2019	10.6	246.8	
		August 27, 2019	10.9	246.5	
		March 4, 2021	11.2	246.2	
		April 19, 2021	10.9	246.5	
West Abutment	SUN-04	April 30, 2021	10.9	246.5	Open borehole
		June 22, 2021	11.0	246.4	
		July 4, 2019	Dry	-	
West Abutment	B13-1 ⁽¹⁾	February 6, 2001	Dry	-	Open borehole
Pier	SUN-02	January 20, 2019	-	-	Unable to determine water level due to use of drilling mud
East Abutment	SUN-03	July 4, 2019	Dry	-	Open borehole
	SUN-05	July 20, 2019	-	-	Open borehole (caved to 12.2 m) Piezometer
		August 27, 2019	9.5	243.1	
		March 4, 2021	10.2	242.4	
		April 19, 2021	9.9	242.7	
		April 30, 2021	9.9	242.7	
East Abutment	B13-2 ⁽¹⁾	June 22, 2021	9.9	242.7	Open borehole
		February 7, 2001	11.0	244.1	
		March 15, 2001	9.0	246.1	

The values shown in Table 5.1 are short-term readings, and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after periods of significant or prolonged precipitation.

5.11 Corrosivity and Sulphate 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 5.2. The laboratory certificates of analysis are presented in Appendix B.



Table 5.2 – Analytical Corrosivity Test Results

Sample ID	Depth (m)	Soil Sample Descripti	Sulphide (percent)	Chloride (µg/g)	Sulphate (µg/g)	pH	Resistivity (ohm.cm)	Redox Potential (mV)	Electrical Conductivity (µS/cm)
SUN-01 SS5	3.0 - 3.6	Sand and silt fill	<0.2	34	2.3	9.17	4720	365	212
SUN-02 SS4	3.0 - 3.6	Silty sand till	<0.2	52	4.4	9.09	4550	348	220
SUN-03 SS2	1.5 - 2.1	Silty clay fill	<0.2	1900	57	8.64	331	278	3020
SUN-05 SS7	6.1 - 6.7	Silty sand	<0.2	66	6.3	9.69	3160	303	316

6 MISCELLANEOUS

Thurber staked and/or marked the borehole locations in the field and obtained utility clearances prior to drilling. McIntosh Perry surveyed the boreholes in the field, and provided the borehole coordinates and ground surface elevations.

Walker Drilling of Utopia, 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. Bryan Lui and Mr. Kevin Kweon of Thurber. Overall supervision of the field program was performed by Mr. Karel Furbacher, P.Eng. of Thurber.

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



THURBER ENGINEERING LTD.



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PART 2: ENGINEERING DISCUSSION AND ECOMMENDATIONS

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 400 Sunnidale Road underpass structure in Barrie, Ontario.

This foundation investigation and design report, with the interpretation and recommendations, is intended for the use of the Ministry of Transportation (MTO) and McIntosh Perry (MP), 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.

Based on the AECOM Structural Design Report (SDR) dated July 2017 and site observations, the existing underpass structure is a single-span cast-in place reinforced concrete rigid frame structure. Information provided by McIntosh Perry and a preliminary foundation investigation report dated October 2016 (Reference 1) indicate that the existing structure is supported on spread footings designed to be founded at approximate Elevation 247.4. There is no construction record available to confirm the foundation depth of the existing structure. The existing bridge is 28.8 m in length and 11.8 m in width. The existing approach slopes are in the order of 9 m high at the west abutment and 5.5 m high at the east abutment, with a design inclination of 2H : 1V for the side slope. Near the structure, the existing Highway 400 grade is at approximate Elevation 248.5, whereas the Sunnidale Road grade decreases from approximate Elevations



257.6 to 254.0 from the west abutment to east abutment.

Based on the AECOM SDR (2017), the existing structure was originally constructed in 1951 under MTO Contract No. 50-11 and 50-66. The structure was rehabilitated in 1990, which included removal of the entire 180 mm thick deck top slab including asphalt, curb and sidewalk, and replacement with a new 225 mm deck top slab with 90 mm waterproofing and paving. The parapet walls were also removed and replaced with concrete barrier walls. Miscellaneous concrete patch repairs were carried out on the balance of the structure. A holding strategy via Contract 2013-2020 was completed in 2014 for the Sunnidale Road Underpass structure. The scope of this strategy included the removal and repair of the deteriorated and loose concrete areas on the deck soffit.

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.

In general, GA drawings provided by MP dated March 2021 show that the existing structure will be removed, and Highway 400 will be widened by approximately 25.6 m to the west and to the east requiring deep cuts on both sides. The proposed replacement two-span structure will be longer and wider than the existing structure, and consist of precast, prestressed concrete girders supported on two abutments and one pier. Integral abutments are proposed to be supported on single rows of driven H-piles, while two alternatives for the piers are being considered as follows:

- Pier supported on 1.8-m diameter caissons
- Pier supported on spread footings with a width of approximately 5.5 m.

The new bridge will have two clear spans each of approximately 40 m in length, and a deck of about 14.6 m in width. The new structure will have an approximately 6° skew with Highway 400, and be located along the same horizontal alignment as the existing bridge. It will accommodate the proposed 10-lane configuration of Highway 400 in the vicinity of Sunnidale Road. The existing Sunnidale Road will also be widened to accommodate two lanes of traffic, new bike lanes, new 1.5 m wide shoulders and 2.0 m wide sidewalks.

On Sunnidale Road, grade raise of up to 1.0 m is proposed at the west approach and up to 1.5 m at the east approach. It is understood that there will be no change to the road grade on Highway 400.

Permanent earth cuts extending to approximate Elevation 248 will be required for widening of



Highway 400. On the west side, the base of cut will be in the order of 9 m to 11 m below the original ground surface (plateau above the highway). On the east side, the cut will range between 4.0 m and 5.5 m in depth.

RSS retaining walls will be constructed beyond the wingwalls at each of the four corner of the underpass for retaining the widened Sunnidale Road embankment. These walls will be approximately 6.5 m long at the west abutment and 4.0 m long at the east abutment. In addition, another retaining wall is also proposed in front of the west abutment (parallel to Highway 400) although the extent and location of this wall has not been finalized.

The discussion and recommendations presented in this report are based on the design information provided by McIntosh Perry, 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, 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 FOUNDATION DESIGN

In general, the subsurface stratigraphy encountered at the site consists of pavement structure or topsoil overlying embankment fill which is underlain by an upper deposit of silty sand to sand and silt till, and layers of sand, silty sand and sandy silt. Interbedded layers of hard silty clay/clayey silt and clayey silt till were encountered within the cohesionless soils. A lower deposit of very dense silty sand till with occasional lenses of gravelly sand were encountered in Borehole SUN-01. The groundwater level was observed to be at approximately 9 m to 11 m depths below the Sunnidale Road grade and in the order of 2 m to 5 m below the Highway 400 grade.



9.1 Foundation Alternatives

Based on the subsurface conditions at the site, consideration was given to supporting the new bridge 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 and 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 foundations perspective is then recommended.

Spread Footings on Native Soils

Spread footings founded on native dense to very dense soils are considered feasible to support the new bridge abutments and pier, and for the retaining walls associated with the bridge. The footings would likely be at or above the groundwater table. Sump pumping will be required, but other groundwater control systems such as vacuum well points for the sands may also be needed to facilitate footing construction in the dry. Temporary protection will also be required at various locations. This foundation option will preclude the use of integral abutments. At the pier, the use of spread footings is feasible, but construction will require excavation and dewatering within temporary protection. These activities will have to be well co-ordinated as part of staging to minimize impact on the travelled lanes on Highway 400.

Spread Footings on Engineered Fill

In light of the fact that dense to very dense competent soils are present below the existing fill, and that the proposed founding levels for spread footings on the native soils would be relatively similar to the proposed founding levels for footings on engineered fill, use of spread footings on engineered fill is not cost effective and therefore this option has not been developed further.

Driven Steel H-Piles

Steel H-piles driven to practical refusal into the very dense sand and silt till to sand could be used to support the new bridge abutments and pier. This foundation option would permit integral abutment design should it be considered. The pile caps could either be perched within the



approach embankments or founded in the native sand to silty sand deposits at shallow depths.

The driven pile option will likely require limited groundwater control during pile cap construction. However, there are potential obstructions due to cobbles and boulders, and hard driving conditions are anticipated within the very dense sand and silt till to sand.

Drilled Shafts (Caissons)

If integral abutments are not used, augered caisson foundations founded on the underlying very dense sand and silt till and sand are technically feasible for foundation support of the proposed bridge at this site. In particular, augered caissons may be useful at the pier should consideration be given to a continuous caisson to column transition without using a cap, in order to cope with space restrictions and to minimize excavation adjacent to highway travelled lanes.

However, this alternative carries a higher risk due to the presence of water-bearing cohesionless soils at this site. Construction of caissons through these soils will require use of a temporary steel liner in conjunction with water and/or slurry methods to control the ingress of groundwater, support the sidewalls of the hole and mitigate basal instability. The augered caisson alternative is expected to be less cost effective than that of spread footings or driven piles.

Recommended Foundations

From a foundation technical and cost effectiveness perspective, the preferred foundation alternatives for the new Sunnidale Road bridge are driven piles for the integral abutments, and either spread footing or augered caissons for the pier. The subsurface conditions encountered at this site are considered suitable for integral abutment design.

9.2 Spread Footings on Native Soils

9.2.1 Geotechnical Resistances

Based on the subsurface conditions encountered at this site, new abutment footings and pier footings may be founded on the native, undisturbed dense to very dense sand to silty sand till and silty sand below the frost depth.

Provided a minimum footing width of 3 m is maintained, and taking into consideration the abutment wall geometry (base at about Elevations 249.5 and 247.5 at west and east abutments, respectively), spread footings founded on the above recommended strata may be designed in accordance with the elevations and bearing resistances given in Table 9.1. According to a GA drawing dated March 2021, the proposed spread footings at the pier will be founded at Elevation 246.0 with a width of 5.5 m. Bearing resistances for this option, is also provided in Table 9.1.



Table 9.1 – Geotechnical Resistances for Spread Footings

Foundation Unit	Reference Borehole	Highest Founding Elevation (m)	Founding Soil Type	Factored Geotechnical	
				ULS (kPa)	SLS (kPa)
West Abutment	SUN-01 SUN-04	247.0	Very dense silty sand to sand and silt till	600	400
Pier	SUN-02	247.0 246.0 ⁽¹⁾	Dense silty sand till	450	300
East Abutment	SUN-03 SUN-05	247.0	Dense to very dense silty sand	600	400

(1) Footing width: 5.5 m, as per GA drawing dated March 2021

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 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 dense silty sand to sand and silt till may be computed based on an ultimate coefficient of friction, $\tan \delta$, of 0.5. A resistance Factor of 0.8 should be applied for cohesionless soils, as indicated in Table 6.2 in the CHBDC 2019.

For frost protection purposes, all footing bases should have a minimum earth cover of 1.5 m or its thermal equivalent.



9.2.2 Footing Construction

The base of the footing excavation should be inspected by a Foundation Specialist to confirm that the footing subgrade is in the native, undisturbed dense to very dense sand and silt till, silty sand till, and silty sand 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 with the same class of concrete as that of the footings. Where sub-excavation is required to remove unsuitable material from below the design founding level, the founding surface should be re-established using engineered fill or mass concrete of the same class as that of the footing. All footing construction procedures should follow the guidelines provided in OPSS 902. Suggested wordings for an NSSP outlining the above are included in Appendix H.

The groundwater table is at or just below the recommended footing base. The native silty sand till/sand and silt till and silty sand are prone to disturbance and base boiling due to water seepage and accumulation due to precipitation and surface runoff. Groundwater control will be required to facilitate dry excavations and to minimize subgrade disturbance for footing construction. Feasible means of groundwater control may include sheetpile enclosures in conjunction with sump pumping and well pointing where required. Further details on groundwater control are provided in a later Section 15.0.

At the pier, temporary protection and an effective dewatering system must be implemented for constructing the spread footings. Details for temporary protection design are presented in Section 16.

9.3 Driven Steel H-Piles

From a foundation engineering perspective, it is feasible to support the structure on steel H-piles driven to practical refusal in the very dense cohesionless deposits.

9.3.1 Axial Resistance

It is recommended that the H-piles be driven to achieve resistance in the underlying very dense ("100 blow") soils consisting of sand, sandy silt or silty sand till.

The GA drawing indicates that the underside elevations of the abutment stem at the west and east abutments are at approximately elevations 252.0 and 248.8, respectively. At the west abutment, results of discussions with MTO and MP indicate that excavations will be carried out to about Elevation 247 to facilitate retaining wall construction, and to be followed by pre-augering from



Elevation 247 to 244 prior to driving the piles. The pre-augering is to create a pilot hole to enhance pile penetration through potential obstructions and very dense soils, and to minimize the risk of vertical pile mis-alignment. It is understood that MP will carry out lateral pile analysis to determine the design pile embedment depth required to satisfy lateral resistance requirements. The implications of pre-augering on geotechnical lateral resistance is considered in Section 9.3.3 below.

The axial geotechnical resistances of HP 310 X 110 and HP 360 x 132 steel piles, driven to refusal in very dense (100-blow) cohesionless soils were assessed based on the subsurface conditions encountered at the abutment locations and the design considerations outlined above. 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 9.2.

Table 9.2 – Estimated Pile Tip Elevation and Geotechnical Resistances for H-Piles

Foundation Unit	Borehole	Approx. Pile Tip Elevation (m)	Minimum Pile Length assumed (m)	HP 310 X 110		HP 360 X 132	
				Factored ULS (kN)	Factored SLS (kN)	Factored ULS (kN)	Factored SLS (kN)
West Abutment	SUN-01 SUN-04	235.0	17	1,200	1,000	1,400	1,200
East Abutment	SUN-03 SUN-05	231.8	17	1,200	1,000	1,400	1,200

The values of the Factored Geotechnical Resistance at ULS were assessed based on static analysis assuming a Consequence Factor equal to 1 (Typical), and a geotechnical resistance factor equal to 0.4 for axial compressive loading (typical degree of understanding of the subsurface conditions), as per CHBDC 2019. The SLS values correspond to a pile settlement up to 25 mm. The Geotechnical Resistance at SLS was assessed based on static analysis assuming a geotechnical resistance factor of 0.8 for typical degree of understanding of the subsurface conditions.

The structural capacity of a pile must not be exceeded and should be confirmed by the structural designer. The pile embedment depth must satisfy lateral stability requirements.

The pile tip elevations shown in Table 9.2 may be used for estimating purposes only. The actual



pile tip elevations will be controlled during pile driving as indicated in Section 9.3.2.

Downdrag is not considered a pile design issue at this site.

The structural resistance of the pile must be checked by the structural designer.

From a foundation engineering perspective, the conditions at this site are considered suitable for integral abutments. For integral abutment design, the flexibility of the upper portion of the pile can be provided by a single corrugated steel pipe (CSP) system. A typical single CSP system involves installing a pile through a 600 mm diameter, 3 m long, CSP with the void between the pile and the CSP to be backfilled with uncompacted uniformly graded sand after driving the piles. Due to the presence of 100-blow materials at this site, there is a possibility that some piles would become misaligned to such an extent after driving that they would end up making contact with the inside surface of the CSP. As such, consideration should be given to using 800 mm diameter CSPs instead to allow more room for accommodating potential pile misalignment.

Reference should be made to the integral abutment manual for details of the system.

9.3.2 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. An appropriate pile driving note is "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 as indicated in the table below".

Foundation Unit	Ultimate Geotechnical Resistance per HP 310 x 110 pile (kN)	Ultimate Geotechnical Resistance per HP 360 x 132 pile (kN)
West Abutment	2,400	2,800
East Abutment	2,400	2,800

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. 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 this site, the piles will have to be driven through very dense soils and therefore hard driving conditions should be expected. In order to protect the piles while being driven through obstructions and denser 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 H with statements stipulating that pre-augering should be carried out at the pile locations at the west abutment, between the elevations outlined above, 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 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 7 days be specified before allowing retapping for confirmation of the pile geotechnical resistance.

9.3.3 Lateral 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 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:

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



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:

$$\begin{array}{lll} k_s & = & 67 C_u / B \quad (\text{kN/m}^3) \\ p_{ult} & = & 9 C_u \quad (\text{kPa}) \\ \text{Where} \quad C_u & = & \text{undrained shear strength (kPa)} \\ B & = & \text{pile width or diameter in metres} \end{array}$$

The above equations and recommended parameters in Table 9.3 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.

Lateral soil-pile interaction analyses may be carried out using the coefficient of horizontal subgrade reaction values provided in Table 9.3 below.



Table 9.3 – Recommended Geotechnical Parameters for Lateral Resistance Design

Location	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
West Abutment	SUN-01	249.0 to 244.0	-	21	3.2	5,000	Compact sand fill
		244.0 to 241.0	-	12**	3.5	6,000	Very dense sand and silt till
		241.0 to 232.7	-	12**	3.5	6,000	Very dense sand / silty sand till
Pier	SUN-02	246.0* to 244.5	-	11**	3.2	4,000	Dense silty sand till
		244.5 to 237.0	-	11**	3.5	6,000	Very dense sand
		237.0 to 234.0	-	12**	3.4	5,000	Very dense sandy silt till and sand till
		234.0 to 226.0	200	21	-	-	Hard clayey silt
		226.0 to 223.7	-	11**	3.4	5,000	Very dense sandy silt to silt
East Abutment	SUN-05	247.5* to 244.0	-	11**	3.2	4,000	Dense to very dense silty sand
		244.0 to 240.0	200	21	-	-	Hard silty clay
		240.0 to 236.5	-	11**	3.4	5,000	Compact to very dense silty sand
		236.5 to 228.0	-	12**	3.5	6,000	Very dense sand

* Bottom of CSP.

** Submerged unit weight for cohesionless soils below groundwater level at Elevation 247 m).

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 (kN/m³), 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).

9.4 Augered Caissons (Drilled Shafts)

Augered caissons founded within the very dense sand to silty sand till and very dense sand are feasible to support the pier and the abutments. At the pier where space is limited for construction, it is understood that consideration is being given to using caissons to support the pier so that columns can be structurally connected and extended above ground without a cap.

It is recommended that the caissons be drilled to a minimum of 3 m into the “100-blow” soils. Table 9.4 present the recommended founding depths and elevations for the caissons at the pier and abutments, as well as geotechnical resistances recommended for typical diameter caissons.

Table 9.4 – Founding Levels and Geotechnical Resistances for Augered Caissons

Foundation Element	Borehole	Highest Elevation (m)	Approx. Caisson Length ⁽¹⁾ (m)	Axial Geotechnical Resistance					
				Factored ULSf (kN)	SLS (kN)	Factored ULSf (kN)	SLS (kN)	Factored ULSf (kN)	SLS (kN)
				1.2 m Diameter		1.5 m Diameter		1.8 m Diameter	
West Abutment	SUN-01	237.5	12	3,400	2,700	5,000	4,000	7,000	5,600
Pier	SUN-02	228.0	18	3,800	3,000	5,600	4,500	7,700	6,200
East Abutment	SUN-05	235.5	12	3,400	2,700	5,000	4,000	7,000	5,600

⁽¹⁾ Caisson lengths measured from the underside of the abutment stem, or below the Highway 400 grade at the pier.

The SLS values above correspond up to 25 mm settlement.

The resistance values provided in Tables 9.4 above are based on shaft friction and a portion of end bearing, assuming that the walls and base of each caisson are free of loose, soft or otherwise disturbed material prior to placement of concrete.

Downdrag on caissons is not a design issue at this site.



9.4.1 Lateral Resistance

Lateral bridge loadings can be geotechnically resisted by the caissons through passive pressure developed along the embedded portion of the shaft. The methodology outlined in Section 9.3.3 above for driven piles may be used to estimate the lateral geotechnical resistance of the caissons by substituting the pile width, B, with the caisson diameter, D.

9.4.2 Caisson Installation

Caisson installation must be carried out in accordance with OPSS.PROV 903 where applicable.

The caisson installation equipment should be able to dislodge and remove any obstructions such as cobbles and boulders and to penetrate the very dense layers within the cohesionless tills. An NSSP addressing this issue must be included in the contract documents to alert the bidders. Suggested wording for such an NSSP is provided in Appendix H. Selection of the methods and equipment employed to install the caissons is the responsibility of the Contractor.

The resistance values provided in Table 9.4 above are based on end bearing and shaft friction, assuming that the walls and base of each caisson are cleaned of loose material prior to placement of concrete. Based on piezometric measurements, the groundwater level was found to be up to Elevations 246 to 247. Soil sloughing and water seepage will occur in unsupported holes primarily from the sand, silty sand and cohesionless till layers. Construction of caissons will require the use of temporary steel liners to support the caisson sidewalls and to provide seepage cut-off where required. Water and/or polymer slurry may also be required inside the liners to prevent basal instability. Any accumulated water may have to be pumped out from the hole prior to placing concrete. Concrete should be placed with a minimum delay after each caisson is drilled, cleaned and inspected. If accumulated water in the caisson hole cannot be removed or basal stability cannot be maintained after water removal, consideration should then be given to using the pumped tremie technique to place concrete inside the caisson hole. Suggested wording for an NSSP addressing caisson construction is provided in Appendix H.

9.5 Frost Cover

The design depth of frost penetration at this site is 1.5 m. The base of footings or pile/caisson caps must be provided with a minimum of 1.5 m of earth cover, or its thermal insulation equivalent, as protection against frost action.



10 RETAINING WALLS

It is understood that retaining walls will be built beyond the wingwalls, at each of the four corners of the east and west abutments, to retain the widened Sunnidale Road after the earth cuts are formed. A retaining wall is also proposed in front of the west abutment. At the time of preparation of this report, the following options for retaining walls have been considered:

- Concrete cantilever retaining walls beyond the wingwalls supported on spread footings
- Concrete cantilever or Retained Soil System (RSS) wall in front of the west abutment.

Foundation recommendations for these retaining walls are presented in the following sections.

10.1 Retaining Walls Beyond Wingwalls

It is understood that concrete cantilever retaining walls are being considered beyond the wingwalls on each side of the east and west abutments. The proposed height of the walls is approximately 4.5 m. The length of the southwest and northwest walls will be 6.5 m, and 4.0 m for the southeast and northeast walls.

Based on the subsurface conditions encountered at this site, the cantilever wall footings may be founded on a minimum 1.0 m thick of engineered fill pad 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 and placed in 300 mm loose lifts. The engineered pad should be founded at or below elevations presented in Table 10.1. The corresponding geotechnical resistances recommended are also presented in Table 10.1.



Table 10.1 – Retaining Wall Founding Elevations

Retaining Wall	Reference Borehole	Founding Elevation (m) of Granular Pad	Founding Soil Type	Factored Geotechnical Resistance	
				ULS (kPa)	SLS (kPa)
Northwest	SUN-01/ SUN-04	253.5	Very dense silty sand till	375	250
Southwest	SUN-04		Dense sand / very dense silty sand till		
Northeast	SUN-03	250.5	Compact to dense sand and silt till/silty sand till	375	250
Southeast	SUN-05				

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 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 compact to very dense silty sand till, sand and silt till and sand may be computed based on an ultimate coefficient of friction, $\tan \delta$, of 0.5. A resistance Factor of 0.8 should be applied for cohesionless soils, as indicated in Table 6.2 in the CHBDC 2019.

For frost protection purposes, all footing bases should have a minimum earth cover of 1.5 m or its thermal equivalent.

Footing construction should be conducted as per Section 9.2.2.



10.2 Retaining Wall in front of West Abutment

It is understood that a retaining wall is required in front of the west abutment to retain the forward slopes adjacent to the abutment wall. A concrete cast-in-place cantilever retaining wall may be used for this purpose. A GA drawing dated April 2021 shows that the wall is proposed to be founded at approximate Elevation 247.0.

Foundation recommendations and comments for the design and construction for this retaining wall are the same as those outlined above in Section 9.2 of the spread footing option for the west abutment.

10.3 Global Stability at the Retaining Walls

Global stability analyses were carried for the proposed retaining walls beyond the west abutment wingwalls. The analyses were carried out utilizing the commercially available slope stability analysis program Slope/W (Version 2020) 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.

The soil parameters used in the analyses were estimated from empirical correlations using the results of the in situ Standard Penetration Tests (SPTs) 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 also checked under seismic loading assuming a peak horizontal acceleration of 0.064 g.

Results of the stability analyses are presented on Figures G1 and G2 in Appendix G. The results are also summarized in Table 10.2 below.

Table 10.2 Computed Factors of Safety

Condition	Factor of Safety	Figure (Appendix G)
Retaining walls beyond the West Abutment wingwalls		
Static Drained	3.0	G1
Seismic (PGA 0.064g)	2.5	G2

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 1.0. The estimated F.S. values for global stability in the table above are acceptable for the proposed retaining wall founded on spread footings.



No slope stability concerns are anticipated for the retaining walls at the east abutment due to the lower approach fill height.

10.4 Retained Soil System (RSS) Wall

It is understood that an RSS wall system is also considered as an alternative in front of the west abutment. The proposed wall height is up to about 5.5 m across the west abutment, decreasing on both sides away from the bridge to about 1.6 m high. The length of the wall is approximately 40 m. Beyond the bridge footprint, RSS wall will require excavations upslope for reinforcing strip installation (up to the order of 0.7 to 1.0 times the wall height) and backfill placement. Temporary protection (shoring) will be required to facilitate construction of this type of wall.

RSS walls used on 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-sectional 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, it is recommended that the RSS mass be founded at or below elevations presented in Table 10.3. The corresponding geotechnical resistances recommended are also presented in Table 10.3.

Table 10.3 – Retaining Wall Founding Elevations

Retaining Wall	Reference Borehole	Highest Founding Elevation (m)	Founding Soil Type	Factored Geotechnical Resistance	
				ULS (kPa)	SLS (kPa)
In front of West Abutment	SUN-01	246.0	Very dense sand and silt till	375	250

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 to Clause 6.10.5.

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. 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. It is noted that the subgrade level will be just above the groundwater level. 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.

For RSS wall adjacent to structures, the design of the RSS wall should follow the Provincial Engineering Memorandum, Bridge Office #2019-02, dated March 13, 2019.

An RSS block consists of free-draining granular materials. It is anticipated that the risk of a hydrostatic build-up behind the wall is low. However, a perforated sub-drain running parallel to the wall face and within the lower portion of the RSS block should be provided to enhance drainage away from the wall face.



11 EARTH CUTS

Earth cuts are required for the proposed widening of Highway 400 on both sides, and the corresponding lengthening and widening of the underpass structure. It is anticipated that a staged approach will be adopted during construction. Highway widening cuts will be required towards the west and east behind the existing bridge approaches.

A separate report includes additional boreholes and discusses the full extent of the proposed cuts. This section focuses on temporary cut slopes adjacent to the immediate bridge approaches during construction.

According to a GA drawing, the west cut and east cut will be in the order of 15 m and 17 m long, respectively, along the Sunnidale Road alignment. The cuts will extend up to approximately 9 m and 5.5 m deep near the west and east abutment, respectively, and will be formed through compact to very dense cohesionless fill (sand, sand and gravel, sand and silt) and native compact to very dense sand, silty sand and silty sand till, to about Elevation 248.5. The groundwater levels measured in the piezometers to date ranged from approximately 9 m to 11 m depth below the existing Sunnidale Road grade (approximate Elevations 246.5 to 243). As such, the cuts will largely be made above the groundwater table, although 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 highway and Sunnidale Road corridors. It is recommended that the water be controlled by means of permanent drains incorporated within the highway design.

For permanent slopes, 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) may be required at some locations for the temporary earth cut operations. Recommendations for temporary protection (shoring) are presented in Section 16 of this report. The cut slopes will be backfilled as part of the new north abutment construction.

11.1 Earth Cut Stability

Global stability analysis was carried out for the proposed temporary earth cuts utilizing the same software mentioned previously. Analyses were completed for both static and seismic loading conditions.



The soil parameters used in the analyses were estimated from empirical correlations based on the in situ Standard Penetration Tests (SPTs) and geotechnical laboratory testing data. The groundwater level in our analysis was based on readings obtained from standpipe piezometers.

Analyses of global stability was conducted for cut slope configurations near the west and east abutments assuming existing compact to dense fill, compact to dense sand and very dense silty sand till with temporary 1H : 1V and 1.5H : 1V slope inclinations.

The computed factors of safety are as shown in Table 11.1. Graphical outputs of these analyses are included in Appendix G.

Table 11.1 Computed Factors of Safety

Condition	Factor of Safety	Figure (Appendix G)
WEST ABUTMENT, Height = 9.0 m		
Cut Slope 1H : 1V		
Static Drained	1.2	G3
Seismic (PGA 0.064g)	1.1	G4
Cut Slope 1.5H : 1V		
Static Drained	1.5	G5
Seismic (PGA 0.064g)	1.3	G6
EAST ABUTMENT, Height = 5.5 m		
Cut Slope 1H : 1V		
Static Drained	1.7	G7
Seismic (PGA 0.064g)	1.5	G8
Cut Slope 1.5H : 1V		
Static Drained	1.8	G9
Seismic (PGA 0.064g)	1.6	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. The above results show that a 1.5H : 1V inclination would be acceptable for the temporary cut slopes at all locations provided adequate drainage and erosion protection, as discussed in this report, are provided during construction. A temporary mid-height bench is recommended to enhance stability on the west side. A 1H : 1V inclination may be used at the east abutment where the cut heights are much lower. Alternatively, temporary protection (shoring) may be used locally to limit the size of the excavation required for new abutment construction.



12 APPROACH EMBANKMENTS

The west and east approach embankments will be widened to the north and south parallel to the highway to accommodate the new and wider underpass structure. These widenings will require placement of some new fill, up to 1.0 m and 1.5 m, near the west and east abutments, respectively. Grade raise of about 0.7 m is proposed at the east approach embankment, and 1.4 m at the west approach.

All sideslopes of the approach embankments should be designed for an inclination of 2H : 1V or flatter. Where earth fill embankments are higher than 8 m, mid-height berms should be incorporated in each 8 m vertical interval. The berms should:

- extend for the length through which the embankment height exceeds 8 m
- be at least 2 m wide
- have 2 percent positive grade to shed run-off water

The GA drawing shows that a 2.0 m wide mid-height bench will be built at the west and east approaches. This bench is required at the west approach to satisfy the current MTO practice for slopes that are higher than 8 m. At the east approach, it is understood that the bench is required to improve the visibility of the Province of Ontario Coat of Arms to be displayed on the wingwall.

Prior to fill placement, the subgrade must be adequately prepared to receive the new fill. All vegetation, topsoil, organics, soft/loosened or wet soils should be sub-excavated. 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 or Select Subgrade Material (SSM) 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 must be provided for the slopes.



12.1 Widened Embankment Slope Stability

Analyses of global stability were conducted for the new forward slope configurations, and also for the side slope configuration behind the retaining walls at the east and west approach embankments of the new Sunnidale Bridge.

The computed factors of safety are as shown in Table 12.1. Graphical outputs of these analyses are included in Appendix G.

Selected results of the stability analyses are presented on Figures G11 to G16 in Appendix G. The results are also summarized in Table 12.1 below.

Table 12.1 Computed Factors of Safety

Condition	Factor of Safety	Figure (Appendix G)
WEST APPROACH		
Forward Slope		
Static Drained	1.6	G11
Seismic (PGA 0.064g)	1.4	G12
Side Slope		
Static Drained	1.5	G13
Seismic (PGA 0.064g)	1.3	G14
EAST APPROACH		
Forward Slope		
Static Drained	> 2	G15
Seismic (PGA 0.064g)	2.0	G16

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. The above F.S. values indicate that the long term and seismic conditions are satisfied. The short-term condition is not applicable to this site.

Based on results of the stability analyses, global stability of the approach embankments is not expected to be an issue if the grade raise and widening is constructed using granular materials or Select Subgrade Material (SSM) with side slopes not steeper than 2H : 1V.

No slope stability concerns are anticipated for the road widening at the east abutment due to the much lower embankment height.



12.2 Settlement

Placement of new fill for the grade raise of Sunnidale Road and for the embankment widening will induce settlements within the existing fills and native soils. Based on the soil conditions at this site, it is estimated that not more than 25 mm of foundation settlement will occur beneath the new widened embankments. This settlement is expected to take place as the fill is placed and be completed by the end of construction.

13 SEISMIC CONSIDERATIONS

In accordance with the CHBDC 2019, the selection of the seismic site class is based on the soil conditions encountered in the upper 30 m of the stratigraphic profile. In general, the subsurface stratigraphy encountered at the site consists of a pavement structure overlying embankment fill which is underlain by an upper layer of compact to very dense silty sand to sand and silt till, and layers of sand, silty sand and sandy silt. Interbedded layers of hard silty clay/clayey silt and clayey silt till were encountered within the cohesionless soils. A lower deposit of very dense silty sand till with occasional lenses of gravelly sand were encountered at some locations.

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 2015), 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 0.064 g at the site. Based on the site class and the PGA, the Site Coefficient is determined to be 1.00.

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 data, seismically induced liquefaction of foundation soils is not anticipated under the design earthquake.

14 ABUTMENT 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.

In accordance with Clause 6.12.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 1.7 m for Granular B Type I, or 2.0 m for Granular A and Granular B Type II.

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 abutment walls must incorporate measures such as weep holes and/or subdrains 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 and earth cuts 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, the compact to very dense silty sand till and sand deposits are classified as Type 2 above the groundwater level and, Type 4 below the groundwater level. The embankment fills are classified as Type 3.

Earth cuts will be required at this site, extending to approximately Elevation 248.5 (Highway 400 grade) as discussed in Section 11.0. It is anticipated that bulk excavation of the earth cuts through existing embankment fill and native sands and silts for highway widening will likely not require measures other than sump pumping and surface water diversion from the excavations.

It is anticipated that pile cap or spread footing construction will be carried out following the bulk excavation associated with the earth cuts for highway widening. Excavations for these foundation elements will extend up to 2.5 m below the existing Highway 400 grade. The excavations will extend through the pavement materials and cohesionless embankment fill into the compact to very dense silty sand till and sand/silty sand deposits.

Based on currently available water level readings, excavations for pile cap or spread footing construction are generally expected to extend up to the order of 0.5 m below the groundwater level. Flow of perched water from the embankment fill into the excavations should also be expected. In addition to effective pumping from filtered sumps and perimeter ditches, other measures of groundwater control including the use of interlocking sheetpiles and well points may be required locally 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 may 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 relevant experience will be required to design and implement a dewatering system. It is recommended that a pre-construction condition survey of existing structures within 100 m of the piling locations be carried out prior to commencement of piling. As a preconstruction survey is required at this site, the Designer Fill-In ** in SP FOUN0003 should be "Yes". The radius of influence for dewatering will be in the order of 30 m. SP FOUN0003 and SP517F01 have been included in Appendix H.

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 H.

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

16 TEMPORARY PROTECTION SYSTEMS

Temporary protection (shoring) systems may be required to maintain live traffic lanes during construction of the new Sunnidale Road bridge and to permit construction of the centre pier foundation at the Highway 400 median.

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 predominantly through the existing embankment fill into the underlying native very dense sand and cohesionless till to develop the required toe resistance. Installation of roadway protection should consider that the existing embankment fill and native till may contain obstructions.

For conceptual planning and costing purposes, an augered soldier pile and lagging wall is



considered suitable for temporary protection at this site. There may be difficulties in installing sheetpile walls due to the presence of very dense soils at shallow depths. 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 silty sand, sand, silty sand till, sand and silt till)
	=	0.35 (native clayey silt and silty clay)
K_p	=	3.1 (approach fills)
	=	3.2 (native silty sand, sand, silty sand till, sand and silt till)
	=	2.9 (native clayey silt and silty clay)

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. Full hydrostatic pressure will need to be incorporated for design of sheetpile walls if this type of protection system is used.

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. All shoring systems should be designed by a Professional Engineer experienced in such designs.

Temporary protection system (TPS) installed for the purposes of shoring and/or groundwater control for construction of the centre pier should be left in place. It is recommended that this 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 H.

17 ADJACENT STRUCTURES AND BURIED UTILITIES

The potential presence of 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 foundations of the proposed replacement structures, new fills and associated works.



Protection and/or relocation of utilities, if necessary, should be provided. 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 during the current investigation are included in Appendix B. Based on the test results, the following statements can be made:

- The potential for sulphate attack on concrete from the surrounding fill and native soils is considered low due to the low concentration of sulphate and slightly alkaline pH values.
- The overall potential for corrosion on metal is considered moderate for the sand and silt fill, silty sand till and silty sand at depths ranging from 3.0 m to 6.7 m (Elevations 192.3 and 192.9). Results from a sample of the silty clay fill taken at 1.5 m depth (Elevation 252.5) revealed its severe potential for corrosion on metal. It is recommended that the existing fill not be reused on site for the new construction.
- The effects of road de-icing salts should also be considered when selecting the class of concrete and corrosion mitigation measures.

The vast majority of excavated materials will have to be removed from the site. Some of the excavated native soils may be considered for reuse elsewhere, provided they satisfy all the requirements of the receiving sites as well as applicable engineering and environmental properties.

19 CONSTRUCTION CONCERNS

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

- Staging of the earth cuts and bridge replacement must be carried out in a manner that minimizes the potential for disturbance of the existing bridge foundations adjacent to the work area. Currently available information indicates that the existing bridge is supported on spread footings. Construction activities must avoid undermining the existing foundations while it remains operational.
- The existing fills and native tills may contain obstructions. The Contractor must be equipped and prepared to remove, penetrate or otherwise handle these obstructions during construction.
- Foundation excavations and earth cuts will typically be carried out above the groundwater



level. Seepage and perched groundwater may be encountered within the embankment fill. Sump pumping, diversion of surface runoff, precipitation and other forms of temporary dewatering 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.

- 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 all backfill materials 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 CLOSURE

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



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Appendix A
Record of Borehole Sheets
Present 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

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


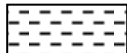



 Water Level
 Shear Strength Determination by Pocket Penetrometer

- (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 Can be peeled by a pocket knife, crumbles under firm blows of geological pick. Indented by thumbnail	
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		
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		
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 SUN-01

1 OF 3

METRIC

GWP# 2445-15-00 LOCATION Sunnidale Road N 4 916 997.6 E 288 541.5 ORIGINATED BY BL
DIST Central HWY 400 BOREHOLE TYPE Hollow Stem Augers and Tricone COMPILED BY AN
DATUM Geodetic DATE 2019.06.17 - 2019.06.18 LATITUDE 44.392803 LONGITUDE -79.704080 CHECKED BY GRL

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						
257.4	GROUND SURFACE							20 40 60 80 100						
0.0	TOPSOIL , some silt and sand, trace gravel, occasional organics Compact Dark Brown Moist		1	SS	11		257							
256.8														
0.7	SAND and SILT , trace gravel, some clay Compact to Dense Brown Moist (FILL) Clayey silt layer at 1.2m (200mm)		2	SS	15		256							
			3	SS	28		255							
			4	SS	30		254							
			5	SS	49		253							
253.3														
4.1	Silty SAND , trace gravel, trace clay, occasional cobbles Very Dense Brown Moist (TILL)		6	SS	100		252							
			7	SS	100/ 0.050		251							
	Occasional cobbles from 6.9m to 7.2m		8	SS	100/ 0.225		250							
			9	SS	100/ 0.225		249							
			10	SS	100/ 0.075		248							
247.5														

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SUN-01

2 OF 3

METRIC

GWP# 2445-15-00 LOCATION Sunnisdale Road N 4 916 997.6 E 288 541.5 ORIGINATED BY BL
DIST Central HWY 400 BOREHOLE TYPE Hollow Stem Augers and Tricone COMPILED BY AN
DATUM Geodetic DATE 2019.06.17 - 2019.06.18 LATITUDE 44.392803 LONGITUDE -79.704080 CHECKED BY GRL

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	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
	Continued From Previous Page												
9.9	SAND and SILT , trace gravel, trace clay Very Dense Brown Moist to Wet (TILL)		11	SS	58		247						
							246						
245.3													
12.2	Gravelly SAND Very Dense Brown Wet		12	SS	100/ 0.225		245						
244.4													
13.0	SAND and SILT , trace gravel, trace clay Very Dense Brown Moist (TILL)		13	SS	100/ 0.100		244						
							243						
							242						
241.1													
16.3	SAND , trace to some gravel, trace to some silt Very Dense Brown Wet		14	SS	100/ 0.275		241						
							240						
			15	SS	100/ 0.250		239						
							238						

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SUN-01

3 OF 3

METRIC

GWP# 2445-15-00 LOCATION Sunnidale Road N 4 916 997.6 E 288 541.5 ORIGINATED BY BL
 DIST Central HWY 400 BOREHOLE TYPE Hollow Stem Augers and Tricone COMPILED BY AN
 DATUM Geodetic DATE 2019.06.17 - 2019.06.18 LATITUDE 44.392803 LONGITUDE -79.704080 CHECKED BY GRL

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 (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								
	Continued From Previous Page		16	SS	100/ 0.250											
236.6	SAND , trace to some gravel, trace to some silt Very Dense Brown to Grey Wet						237									
20.9	Silty SAND , trace to some gravel, trace clay Very Dense Brown Moist (TILL)		17	SS	100/ 0.275		236									
235.0							235									
22.4	SAND , trace to some silt, trace clay Very Dense Brown Wet		18	SS	100/ 0.275		234									
233.4							233									
24.0	Silty SAND , trace clay Very Dense Brown Moist (TILL)		19	SS	100/ 0.175											
232.7																
24.7	END OF BOREHOLE AT 24.7m. BOREHOLE CAVED TO 18.3m AND WATER LEVEL NOT OBSERVED. Piezometer installation consists of 25mm diameter Schedule 40 PVC pipe with a 3.05m slotted screen. WATER LEVEL READINGS DATE DEPTH(m) ELEV.(m) 2019.07.04 10.6 246.8 2019.08.27 10.9 246.5 2021.03.04 11.2 246.2 2021.04.19 10.9 246.5 2021.04.30 10.9 246.5 2021.06.22 11.0 246.4															

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

RECORD OF BOREHOLE No SUN-02

1 OF 3

METRIC

GWP# 2445-15-00 LOCATION Sunnisdale Road N 4 916 963.7 E 288 561.3 ORIGINATED BY BL
DIST Central HWY 400 BOREHOLE TYPE Hollow Stem Augers/Tricone COMPILED BY AN
DATUM Geodetic DATE 2019.01.28 - 2019.01.30 LATITUDE 44.392498 LONGITUDE -79.703831 CHECKED BY GRL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									WATER CONTENT (%)
248.6	GROUND SURFACE							20	40	60	80	100		20	40	60	
0.0	ASPHALT: (200mm)																
0.2	SAND, trace gravel, some silt and clay Dense Brown Moist (FILL)		1	GS			248						○				9 79 12 (SI+CL)
			1	SS	45								○				
247.1																	
1.4	Silty SAND, trace to some gravel, trace clay, occasional cobbles Dense Brown Moist to Wet (TILL)		2	SS	32		247						○				
			3	SS	42		246						○				
			4	SS	35		245						○				
244.5																	
4.1	SAND, trace silt, trace clay Very Dense Brown Moist		5	SS	134		244						○				0 97 3 (SI+CL)
							243										Switch to Tricone
	Moist to Wet		6	SS	100/ 0.250		242						○				
			7	SS	100/ 0.275		241						○				
							240										
			8	SS	100/ 0.250		239						○				

Continued Next Page

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

METRIC

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			20 40 60 80 100	○ UNCONFINED + FIELD VANE	W _P W W _L	WATER CONTENT (%)				
	Continued From Previous Page						● QUICK TRIAXIAL × LAB VANE	20 40 60 80 100	20 40 60					
236.9 11.7	SAND, trace silt, trace clay Very Dense Grey Moist													
			9	SS	100									
233.8 14.8	Sandy SILT, trace clay, trace gravel Very Dense Grey Moist (TILL)													
			10	SS	100/ 0.250									
233.8 14.8	Clayey SILT, trace sand Hard Grey Moist													
			11	SS	100/ 0.225									
233.8 14.8	Clayey SILT, trace sand Hard Grey Moist													
			12	SS	38									
233.8 14.8	Clayey SILT, trace sand Hard Grey Moist													
			13	SS	49									
233.8 14.8	Clayey SILT, trace sand Hard Grey Moist													
			14	SS	74									
233.8 14.8	Clayey SILT, trace sand Hard Grey Moist													
233.8 14.8	Clayey SILT, trace sand Hard Grey Moist													
233.8 14.8	Clayey SILT, trace sand Hard Grey Moist													
233.8 14.8	Clayey SILT, trace sand Hard Grey Moist													
233.8 14.8	Clayey SILT, trace sand Hard Grey Moist													
233.8 14.8	Clayey SILT, trace sand Hard Grey Moist													
233.8 14.8	Clayey SILT, trace sand Hard Grey Moist													

+³, ×³: Numbers refer to Sensitivity

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RECORD OF BOREHOLE No SUN-02

3 OF 3

METRIC








GWP# 2445-15-00 LOCATION Sunnidale Road N 4 916 963.7 E 288 561.3 ORIGINATED BY BL
 DIST Central HWY 400 BOREHOLE TYPE Hollow Stem Augers/Tricone COMPILED BY AN
 DATUM Geodetic DATE 2019.01.28 - 2019.01.30 LATITUDE 44.392498 LONGITUDE -79.703831 CHECKED BY GRL

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							
	Continued From Previous Page						20	40	60	80	100	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	
							20	40	60	80	100	w _P	w	w _L	
													</		

ONTMT4S2 MTO-22424.GPJ 2017TEMPLATE(MTO).GDT 7/26/21

METRIC

GWP#	2445-15-00	LOCATION	Sunnidale Road N 4 916 957.8 E 288 605.2			ORIGINATED BY	KK			
DIST	Central	HWY	400	BOREHOLE TYPE	Solid Stem Augers			COMPILED BY	AN	
DATUM	Geodetic		DATE	2019.07.04 - 2019.07.04	LATITUDE	44.392447	LONGITUDE	-79.703280	CHECKED BY	GRL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	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						WATER CONTENT (%)	
							20 40 60 80 100							
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE							
254.0	GROUND SURFACE													
0.0 0.1	ASPHALT: (100mm)		1	GS										
	SAND and GRAVEL Loose Brown Moist (FILL)													
252.9			1	SS	8									
1.1	Silty CLAY, some sand, trace gravel, occasional cobbles Stiff Brown Moist (FILL)		2	SS	11									
251.8														
2.2	SAND and SILT, trace gravel, trace clay, occasional cobbles Dense to Very Dense Brown Moist (TILL)		3	SS	39									
				4	SS	32								
														
				5	SS	70								
														
				6	SS	87								
246.9														
7.2	Silty SAND, trace clay Very Dense Brown Moist			7	SS	72								
														
				8	SS	100/ 0.175								
244.6														
9.5	END OF BOREHOLE AT 9.5m. BOREHOLE OPEN AND DRY UPON COMPLETION.													

+³, ×³: Numbers refer to Sensitivity

ONTMT4S2 MTO-22424.GPJ 2017TEMPLATE(MTO).GDT 7/26/21

RECORD OF BOREHOLE No SUN-03

2 OF 2

METRIC

GWP# 2445-15-00 LOCATION Sunnisdale Road N 4 916 957.8 E 288 605.2 ORIGINATED BY KK
DIST Central HWY 400 BOREHOLE TYPE Solid Stem Augers COMPILED BY AN
DATUM Geodetic DATE 2019.07.04 - 2019.07.04 LATITUDE 44.392447 LONGITUDE -79.703280 CHECKED BY GRL

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			20	40	60	80	100	W _p	W	W _L		
	Continued From Previous Page																
	BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND AUGER CUTTINGS TO 0.1m, THEN ASPHALT TO SURFACE.																

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+³, ×³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SUN-04

1 OF 2

METRIC

GWP# 2445-15-00 LOCATION Sunnisdale Road N 4 916 985.9 E 288 533.3 ORIGINATED BY KK
 DIST Central HWY 400 BOREHOLE TYPE Solid Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2019.07.04 - 2019.07.04 LATITUDE 44.392697 LONGITUDE -79.704182 CHECKED BY GRL

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					
257.6	GROUND SURFACE												
0.0	ASPHALT: (125mm)												
0.1	SAND and GRAVEL Dense Brown Moist (FILL)		1	GS			257						
256.5			1	SS	34								
1.1	SAND, some gravel, trace to some silt and clay Compact Brown Moist (FILL)		2	SS	21		256						
			3	SS	23		255						
254.7													
3.0	SAND, trace silt, trace clay Compact to Dense Brown Moist		4	SS	29		254						
			5	SS	36		253						
252.0													
5.6	Silty SAND, trace gravel, trace clay, occasional stains Very Dense Brown Moist (TILL)		6	SS	94		252						
			7	SS	96		251						
			8	SS	73		250						
247.9							249						
9.8	END OF BOREHOLE AT 9.8m.						248						

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SUN-04

2 OF 2

METRIC

GWP# 2445-15-00 LOCATION Sunnidale Road N 4 916 985.9 E 288 533.3 ORIGINATED BY KK
 DIST Central HWY 400 BOREHOLE TYPE Solid Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2019.07.04 - 2019.07.04 LATITUDE 44.392697 LONGITUDE -79.704182 CHECKED BY GRL

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			20	40	60	80	100	W _p	W	W _L		
	Continued From Previous Page																
	BOREHOLE OPEN AND DRY UPON COMPLETION. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG AND AUGER CUTTINGS TO 0.125m, THEN ASPHALT TO SURFACE.																

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RECORD OF BOREHOLE No SUN-05

1 OF 3

METRIC

GWP# 2445-15-00 LOCATION Sunnisdale Road N 4 916 945.5 E 288 599.7 ORIGINATED BY BL
DIST Central HWY 400 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
DATUM Geodetic DATE 2019.07.19 - 2019.07.20 LATITUDE 44.392336 LONGITUDE -79.703348 CHECKED BY GRL

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												
252.6	GROUND SURFACE							20	40	60	80	100								
0.0	TOPSOIL (200mm)							20	40	60	80	100								
0.2	Silty SAND , trace gravel, trace to some clay Loose to Compact Brown Moist (TILL) Occasional cobbles from 3.0m to 3.7m Very Dense		1	SS	6															
			2	SS	17															
			3	SS	19															
			4	SS	24															
248.6			5	SS	75															
4.0	Silty SAND , trace gravel, trace clay Dense to Very Dense Brown Moist		6	SS	30															
244.0			8	SS	60															
8.7	Silty CLAY , trace sand Hard Grey to Brown Moist		9	SS	37															

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204060801

Continued Next Page


+³, ×³: Numbers refer to Sensitivity 20 15 10 5 0 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SUN-05

3 OF 3

METRIC

GWP# 2445-15-00 LOCATION Sunnidale Road N 4 916 945.5 E 288 599.7 ORIGINATED BY BL
 DIST Central HWY 400 BOREHOLE TYPE Hollow Stem Augers COMPILED BY AN
 DATUM Geodetic DATE 2019.07.19 - 2019.07.20 LATITUDE 44.392336 LONGITUDE -79.703348 CHECKED BY GRL

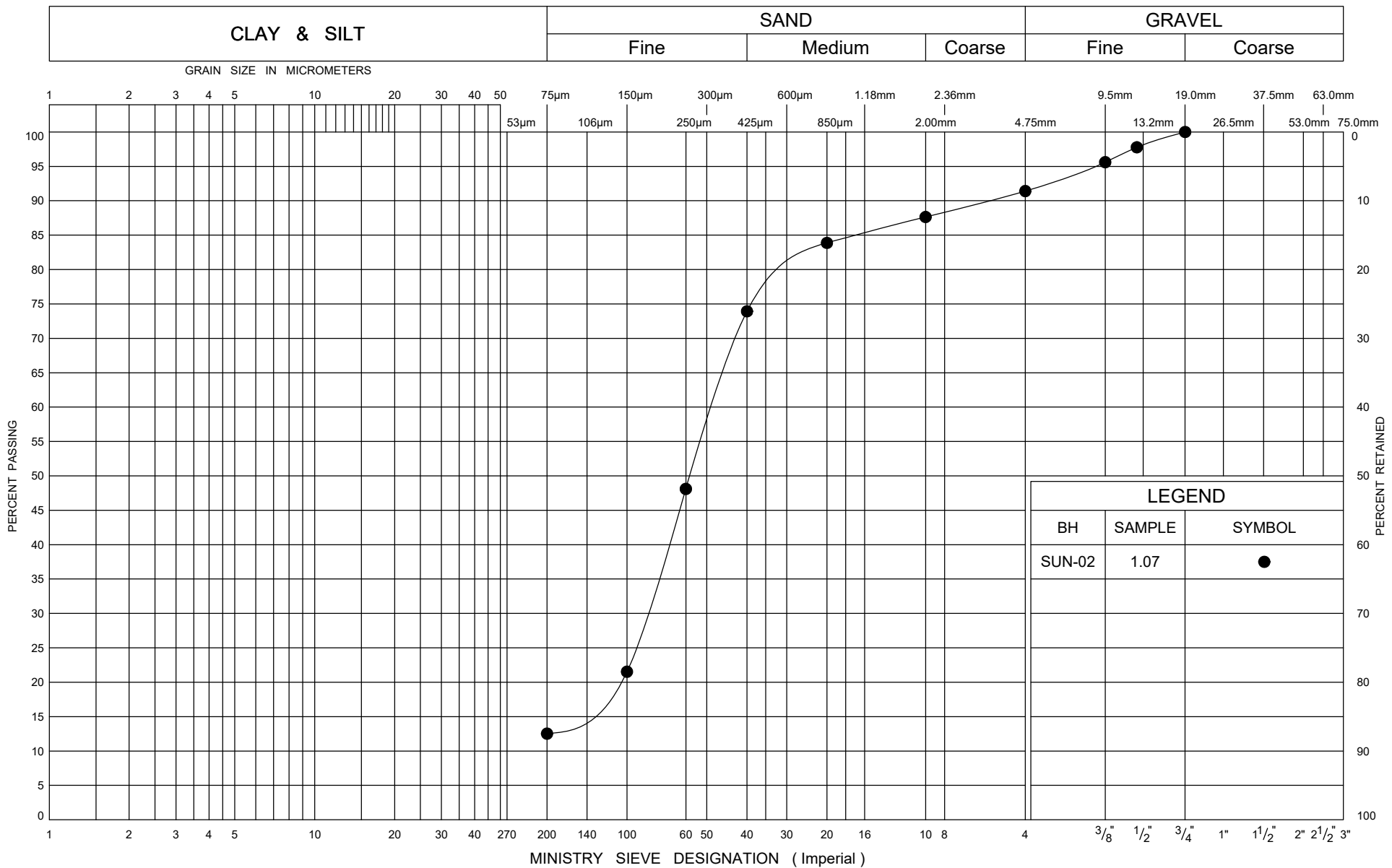
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)			
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE									
	Continued From Previous Page		16	SS	100/ 0.225															
	SAND , trace silt, trace clay Very Dense Brown Moist to Wet																			
			17	SS	100/ 0.225															

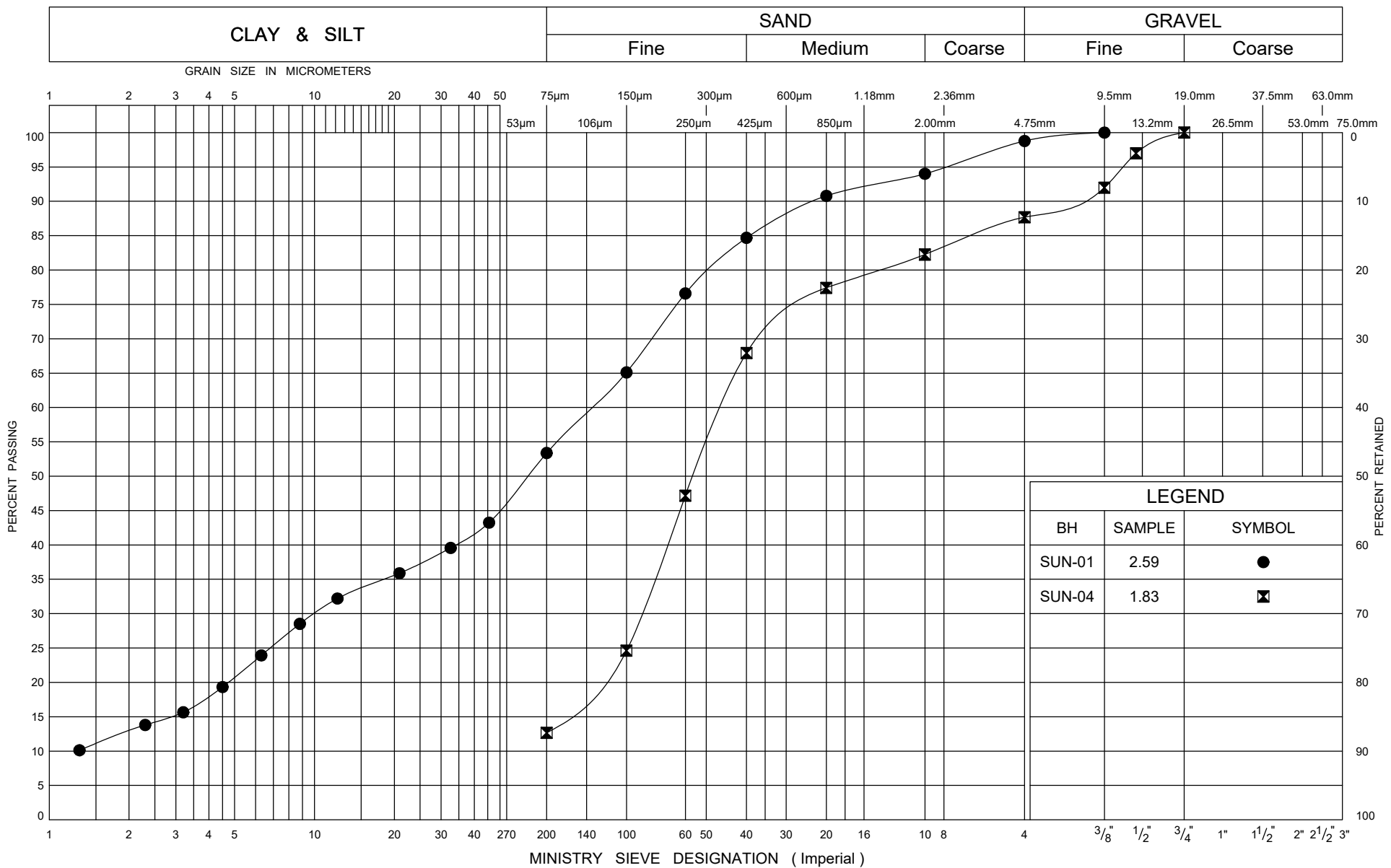
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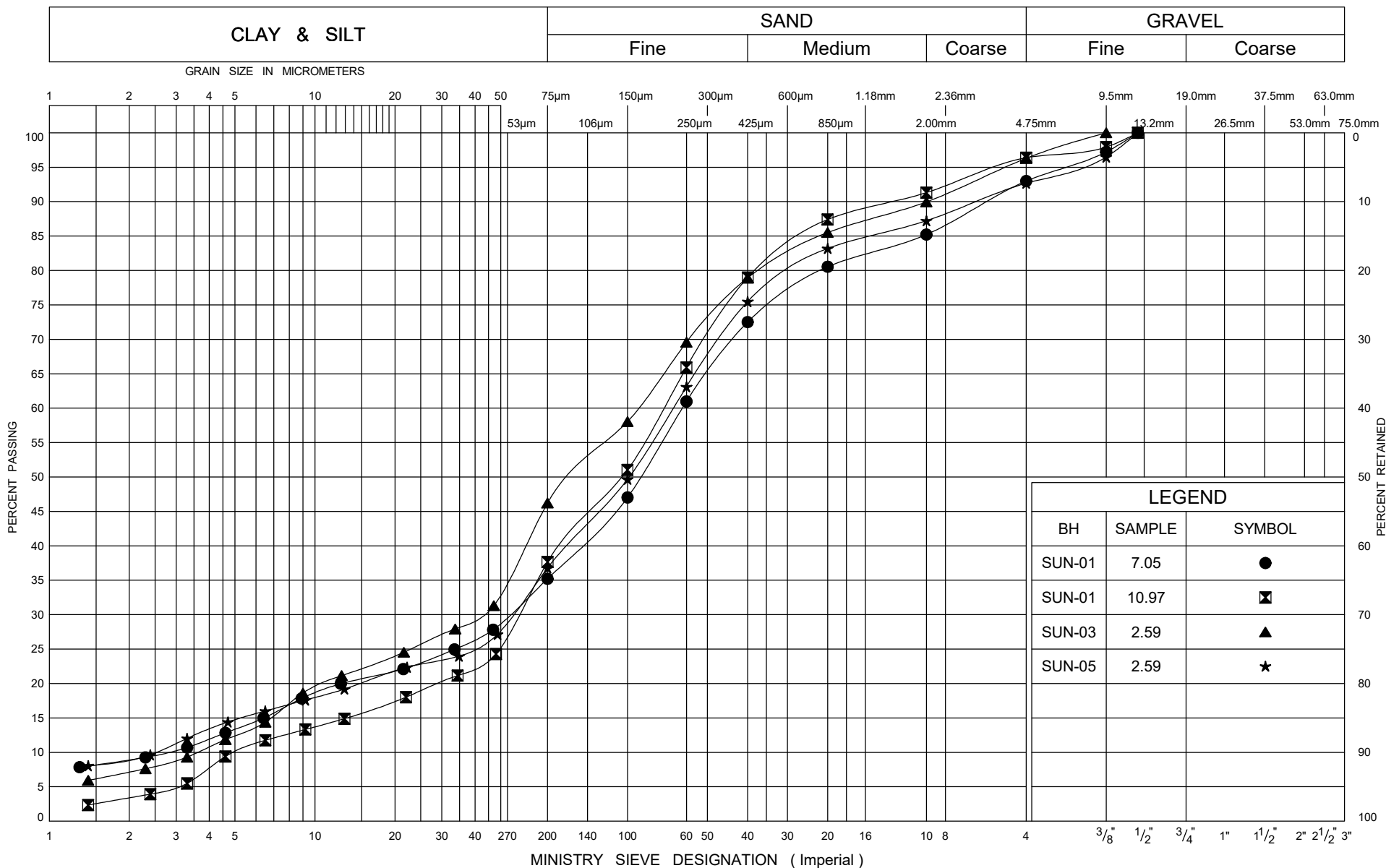


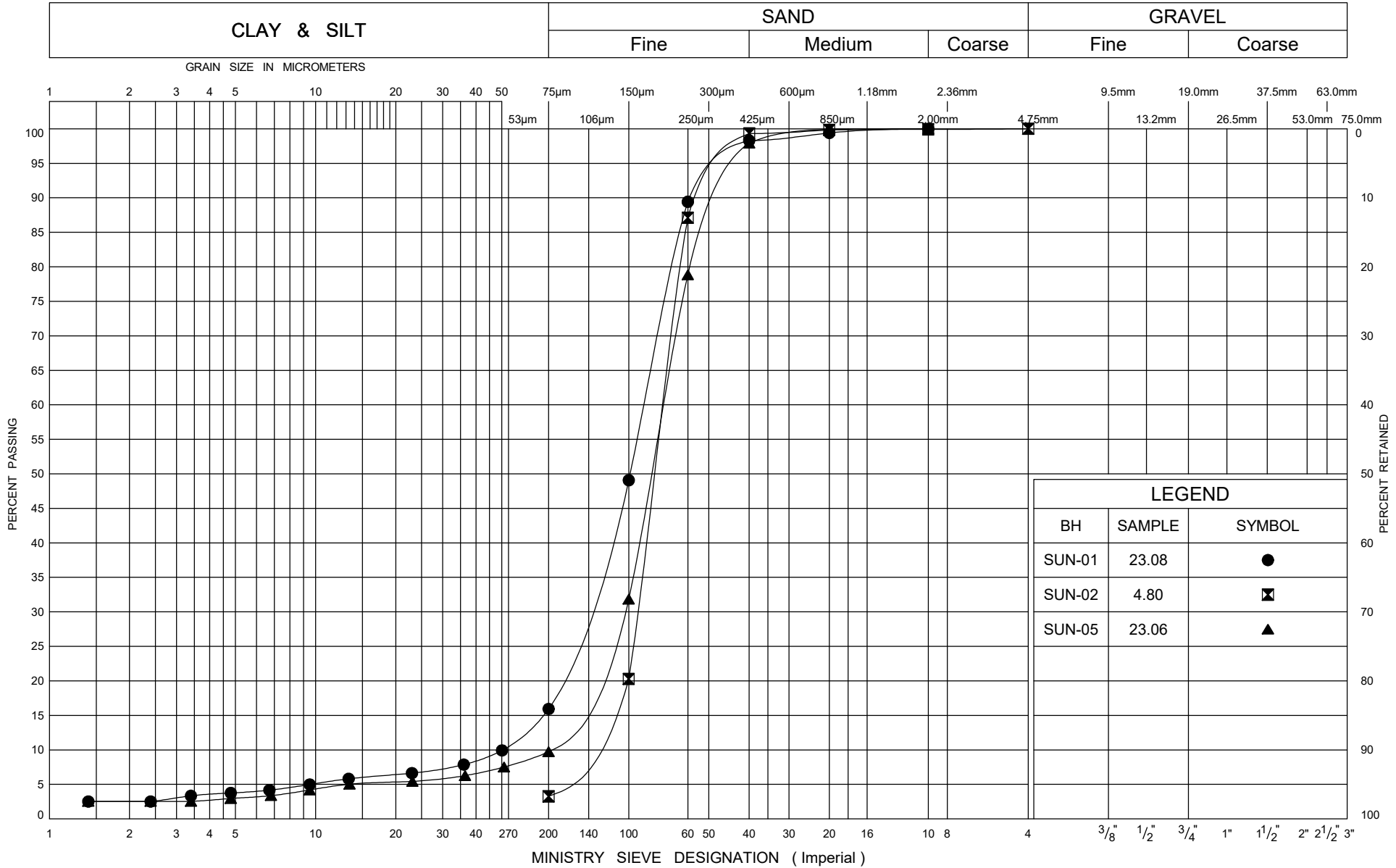
Appendix B

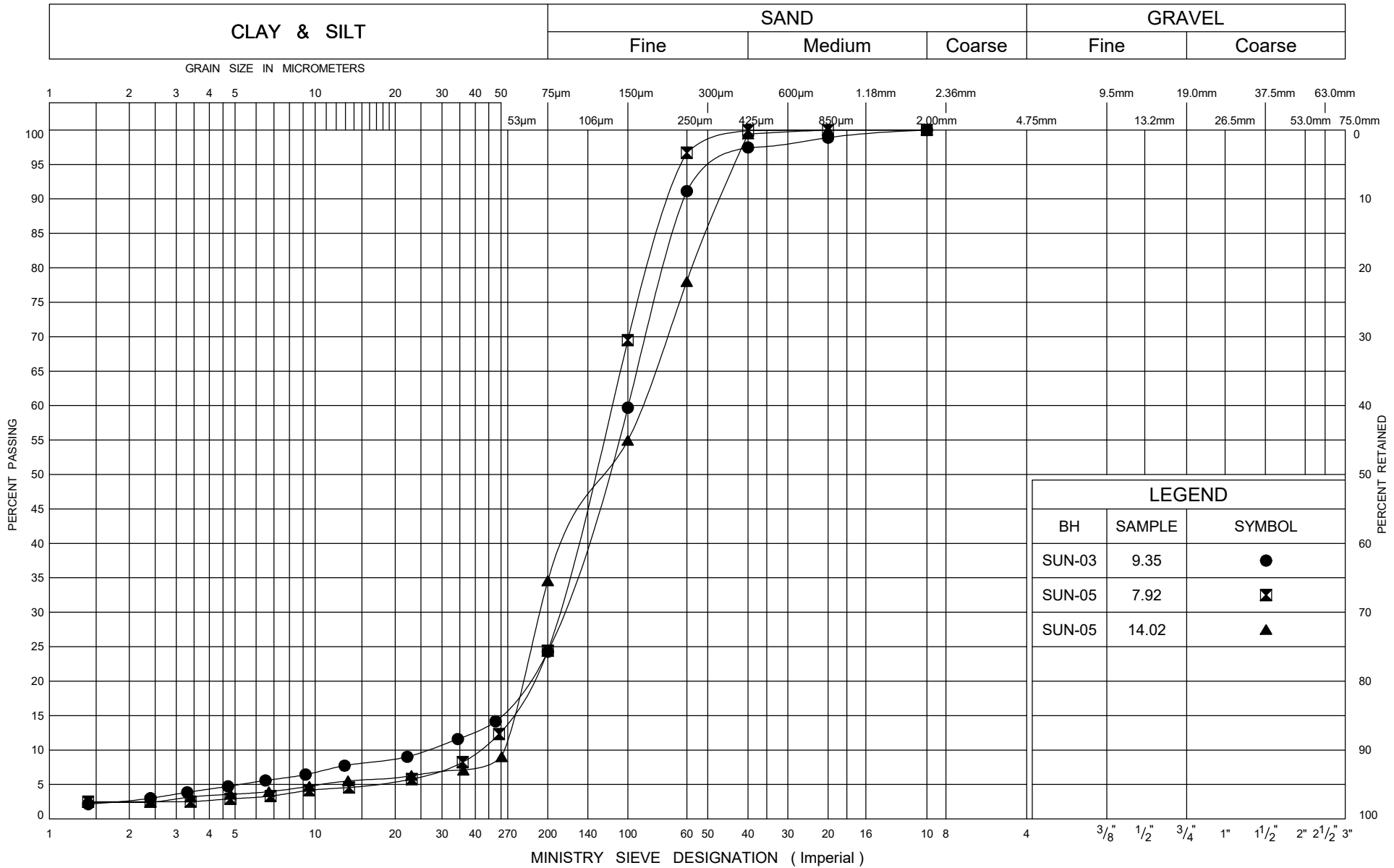
Geotechnical and Analytical Laboratory Test Results Present Investigation

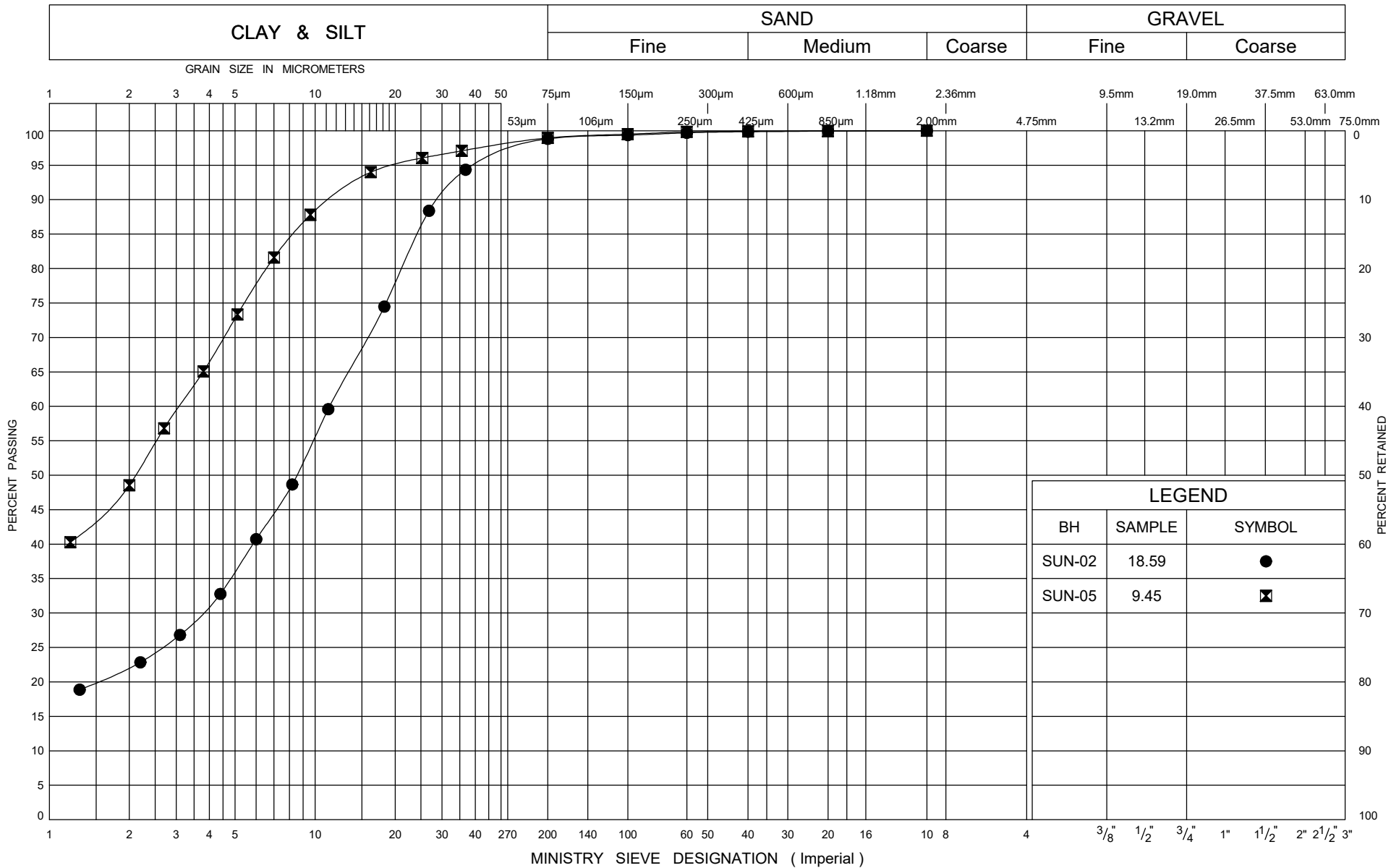


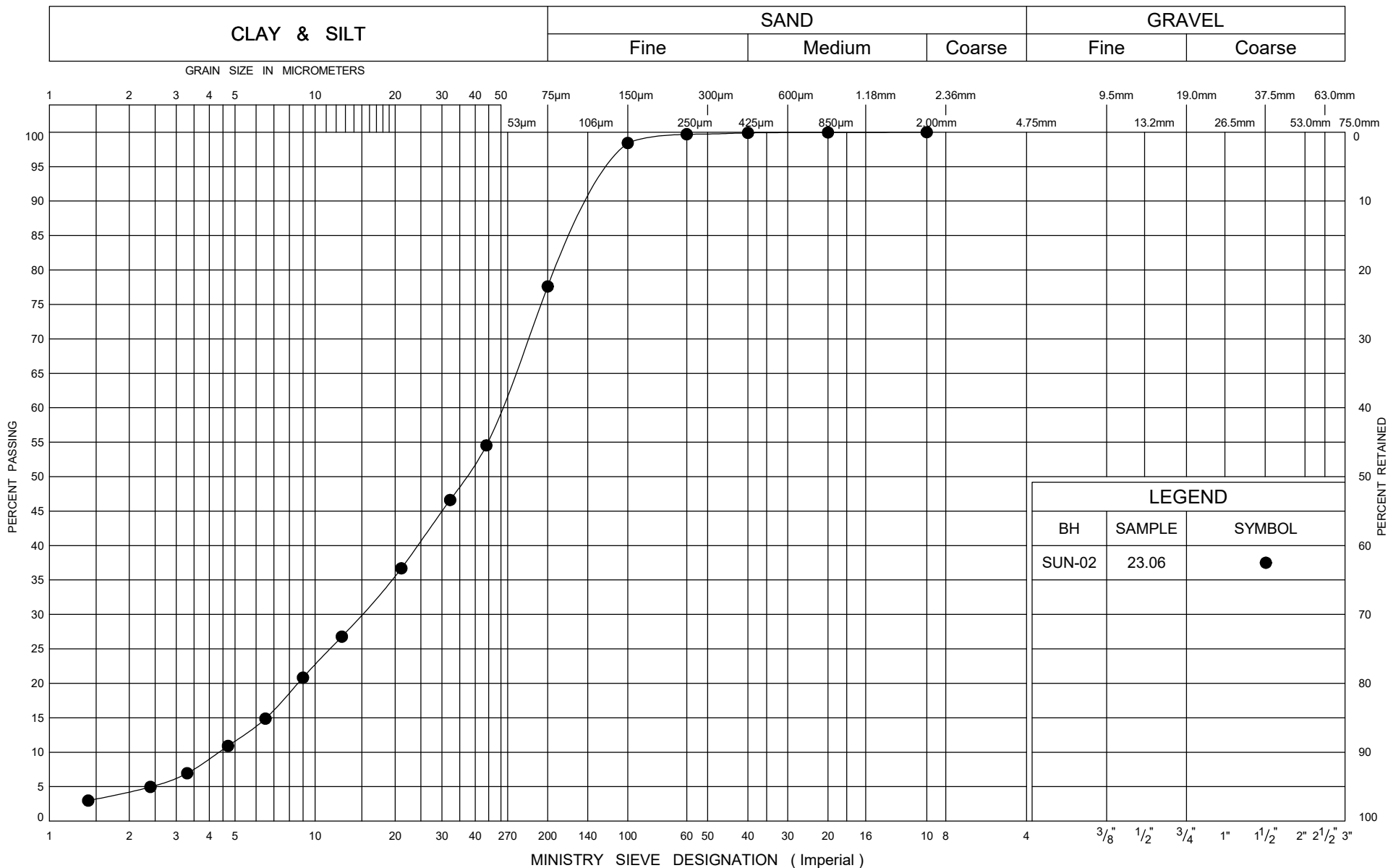


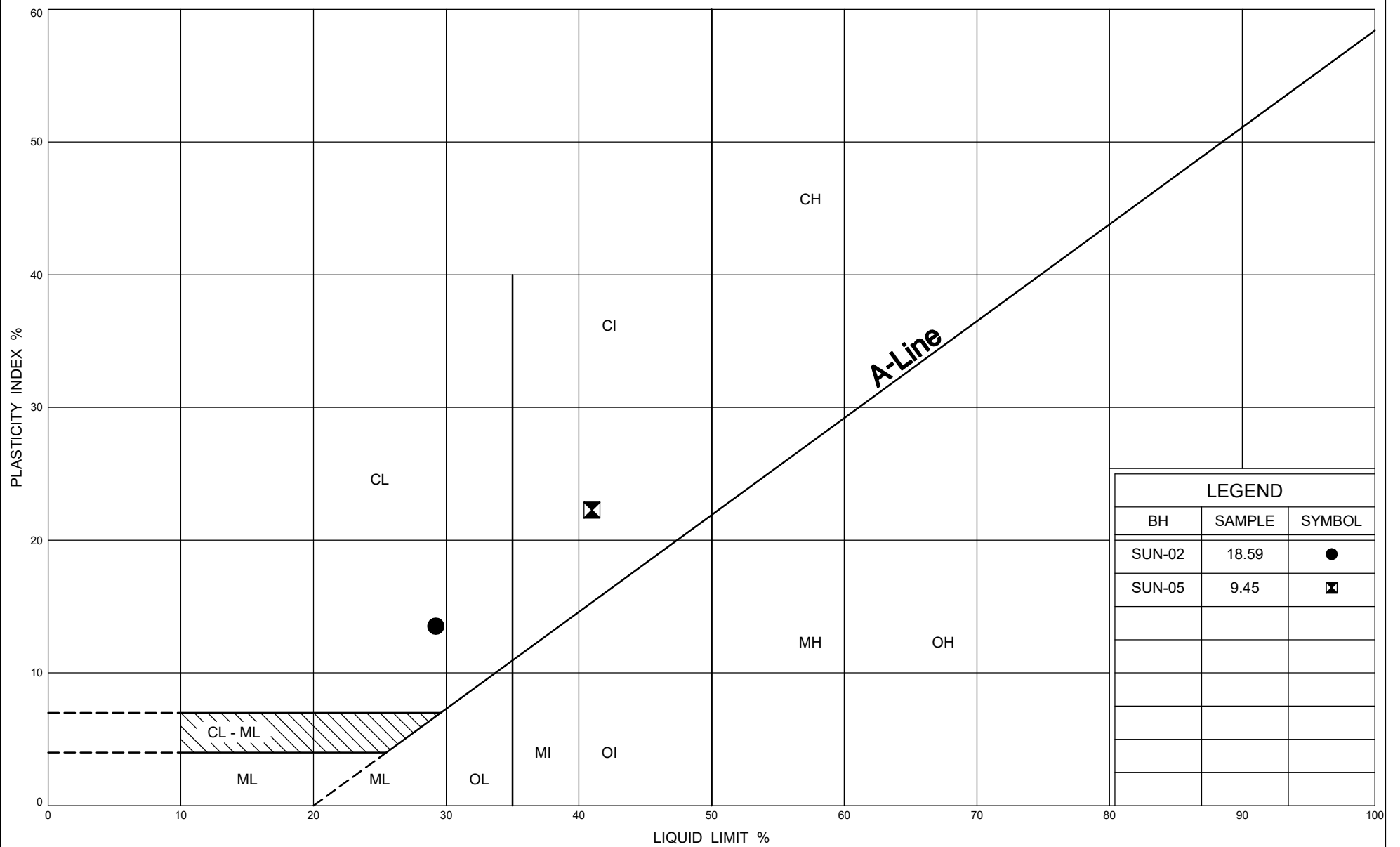












Ministry of
Transportation

PLASTICITY CHART

Silty CLAY to Clayey SILT

FIG No B8

W P 2445-15-00



FINAL REPORT

CA14976-AUG19 R1

22424

Prepared for

Thurber Engineering Ltd.

First Page

CLIENT DETAILS

Client Thurber Engineering Ltd.

Address 103, 2010 Winston Park Drive
Oakville, ON
L6H 5R7, Canada

Contact Rocio Reyna

Telephone 905-829-8666 x 263

Facsimile

Email rreyna@thurber.ca

Project 22424

Order Number

Samples Soil (4)

LABORATORY DETAILS

Project Specialist Brad Moore Hon. B.Sc

Laboratory SGS Canada Inc.

Address 185 Concession St., Lakefield ON, K0L 2H0

Telephone 705-652-2143

Facsimile 705-652-6365

Email brad.moore@sgs.com

SGS Reference CA14976-AUG19

Received 08/27/2019

Approved 09/04/2019

Report Number CA14976-AUG19 R1

Date Reported 09/04/2019

COMMENTS

Temperature of Sample upon Receipt: 9 degrees C

Cooling Agent Present: YES

Custody Seal Present: NO

Chain of Custody Number: 002537

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

SIGNATORIES

Brad Moore Hon. B.Sc

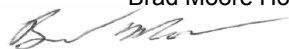




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FINAL REPORT

CA14976-AUG19 R1

Client: Thurber Engineering Ltd.

Project: 22424

Project Manager: Rocío Reyna

Samplers: N/A

PACKAGE: - Corrosivity Index (SOIL)

Sample Number	5	6	7	8
Sample Name	SUN-01, SS5	SUN-02-SS4	SUN-05-SS7	SUN-03-SS2
Sample Matrix	Soil	Soil	Soil	Soil
Sample Date	17/07/2019	28/07/2019	19/07/2019	04/07/2019

Parameter	Units	RL		Result	Result	Result	Result
Corrosivity Index							
Corrosivity Index	none	1		4	3	3	17.5
Soil Redox Potential	mV	-		365	348	303	278
Sulphide	%	0.02		< 0.02	< 0.02	< 0.02	0.02
pH	pH Units	0.05		9.17	9.09	9.69	8.64
Resistivity (calculated)	ohms.cm	-9999		4720	4550	3160	331

PACKAGE: - General Chemistry (SOIL)

Sample Number	5	6	7	8
Sample Name	SUN-01, SS5	SUN-02-SS4	SUN-05-SS7	SUN-03-SS2
Sample Matrix	Soil	Soil	Soil	Soil
Sample Date	17/07/2019	28/07/2019	19/07/2019	04/07/2019

Parameter	Units	RL		Result	Result	Result	Result
General Chemistry							
Conductivity	uS/cm	2		212	220	316	3020

PACKAGE: - Metals and Inorganics (SOIL)

Sample Number	5	6	7	8
Sample Name	SUN-01, SS5	SUN-02-SS4	SUN-05-SS7	SUN-03-SS2
Sample Matrix	Soil	Soil	Soil	Soil
Sample Date	17/07/2019	28/07/2019	19/07/2019	04/07/2019

Parameter	Units	RL		Result	Result	Result	Result
Metals and Inorganics							
Moisture Content	%	0.1		7.4	0.3	4.6	9.0
Sulphate	µg/g	0.4		2.3	4.4	6.3	57



FINAL REPORT

CA14976-AUG19 R1

Client: Thurber Engineering Ltd.

Project: 22424

Project Manager: Rocío Reyna

Samplers: N/A

PACKAGE: - Other (ORP) (SOIL)

Sample Number	5	6	7	8
Sample Name	SUN-01, SS5	SUN-02-SS4	SUN-05-SS7	SUN-03-SS2
Sample Matrix	Soil	Soil	Soil	Soil
Sample Date	17/07/2019	28/07/2019	19/07/2019	04/07/2019

Parameter	Units	RL		Result	Result	Result	Result
Other (ORP)							
Chloride	µg/g	0.4		34	52	66	1900



FINAL REPORT

CA14976-AUG19 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	DIO0563-AUG19	µg/g	0.4	<0.4	9	20	94	80	120	108	75	125
Sulphate	DIO0563-AUG19	µg/g	0.4	<0.4	5	20	96	80	120	95	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	ECS0052-AUG19	%	0.02	<0.02	ND	20	116	80	120			

Conductivity

Method: SM 2510 | Internal ref.: ME-CA-IENVIEWL-LAK-AN-006

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Conductivity	EWL0466-AUG19	uS/cm	2	0.014	0	10	100	90	110	NA		



FINAL REPORT

CA14976-AUG19 R1

QC SUMMARY

pH
Method: SM 4500 | Internal ref.: ME-CA-1ENVIEWL-LAK-AN-001

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
pH	EWL0466-AUG19	pH Units	0.05	NA	0		100			NA		

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

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

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

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

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

RL: Reporting limit

RPD: Relative percent difference

AC: Acceptance criteria

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

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

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

LEGEND

FOOTNOTES

NSS Insufficient sample for analysis.

RL Reporting Limit.

↑ Reporting limit raised.

↓ Reporting limit lowered.

NA The sample was not analysed for this analyte

ND Non Detect

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

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

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

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

-- End of Analytical Report --

Laboratory Information Section - Lab use only

Received By: Ahmed Al-Muradani
Received Date (mm/dd/yyyy): 08/27/2019 (mm/dd/yyyy)
Received Time: 16:00
Cooling Agent Present: ☒ No ☐ Yes
Custody Seal Intact: ☒ No ☐ Yes
Temperature Upon Receipt (°C): 9.9, 9.1
LAB LIMS #: QA 14976-Aug 19

PROJECT INFORMATION

Quotation #: 22424 P.O. #: _____
Project #: 22424 Site Location/ID: _____

TURNAROUND TIME (TAT) REQUIRED

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

☒ Regular TAT (5-7 days)

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

PLEASE CONFIRM RUSH FEASIBILITY WITH SGS REPRESENTATIVE PRIOR TO SUBMISSION

Specify Due Date: _____

Rush Confirmation ID: _____

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

REGULATIONS

Regulation 153/04:

Table 1 ☐ Res/Park
Table 2 ☐ Ind/Com
Table 3 ☐ Agri/Other
Table ☐ _____
Soil Texture: ☐ Coarse ☐ Medium ☐ Fine

Other Regulations:

Reg 347/558 (3 Day min TAT)
☐ PWQO ☐ MMER ☐ Other:
☐ CCME ☐ MISA

Sewer By-Law:

☐ Sanitary
☐ Storm
Municipality: _____

RECORD OF SITE CONDITION (RSC) ☐ YES ☐ NO

SAMPLE IDENTIFICATION

DATE SAMPLED

TIME SAMPLED

OF BOTTLES

MATRIX

1	SUN-01, 555	17/5/19		1	Soil
2	SUN-02, 554	28/5/19		1	"
3	SUN-05, 557	19/5/19		1	"
4	SUN-03, 552	04/5/19		1	"
5					"
6					
7					
8					
9					
10					
11					
12					

ANALYSIS REQUESTED

Field Filtered (Y/N) ☐
Metals & Inorganics ☐
PHC F1-F4 ☐ VOC ☐
BTEX ☐ BTEX/F1 ☐ F2-F4 ☐
PAH ☐ ABN ☐ SVOC(all) ☐
PCB Total ☐ Aroclor ☐
Pesticides OC ☐ OP ☐
TCLP M&I ☐ VOC ☐ PCB ☐
B(a)P ☐ ABN ☐ Ignit. ☐
Water Pkg Gen. ☐ Ext. ☐
Sewer Use: _____

COMMENTS:

Corrosivity Package

Observations/Comments/Special Instructions

Sampled By (NAME): _____ Signature: _____
Relinquished by (NAME): _____ Signature: _____
Date of Issue: 04 April, 2018

Date: ____/____/____ (mm/dd/yy)

Date: 26/08/19 (mm/dd/yy)

Pink Copy - Client

Yellow & White Copy - SGS



Appendix C

Record of Borehole Sheets and Laboratory Test Results Previous Investigation

PROJECT 001-1143F		RECORD OF BOREHOLE No B13-1		1 OF 1		METRIC							
W.P. 30-95-00		LOCATION N 4916980.9; E 288547.3		ORIGINATED BY PKS									
DIST SW HWY 400		BOREHOLE TYPE 108mm ID HOLLOW STEM AUGERS		COMPILED BY LCC									
DATUM Geodetic		DATE Feb. 8/2001		CHECKED BY ASP									
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					
256.8	GROUND SURFACE						20 40 60 80 100						
0.0	Asphalt												
0.2	Sand and Gravel, some cobbles (Fill) Compact to dense Brown Moist												
			1	SS	32								
			2	SS	21								
			3	SS	9								
249.5													
7.3	Silty Sand, trace gravel Compact Brown Moist		4	SS	19								
248.4													
8.4	Silty Sand, trace clay, trace gravel (Till) Very dense Brown Moist		5	SS	104								
			6	SS	100/15								4 57 30 9
			7	SS	101								
246.3													
10.5	Clayey Silt, trace to some sand and gravel (Till) Hard Brown Moist		8	SS	102								
244.4			9	SS	100/18								
12.4	END OF BOREHOLE												
	Notes: 1. Borehole dry on completion of drilling. 2. Borehole backfilled with bentonite and surface cold patch on sand bedding.												

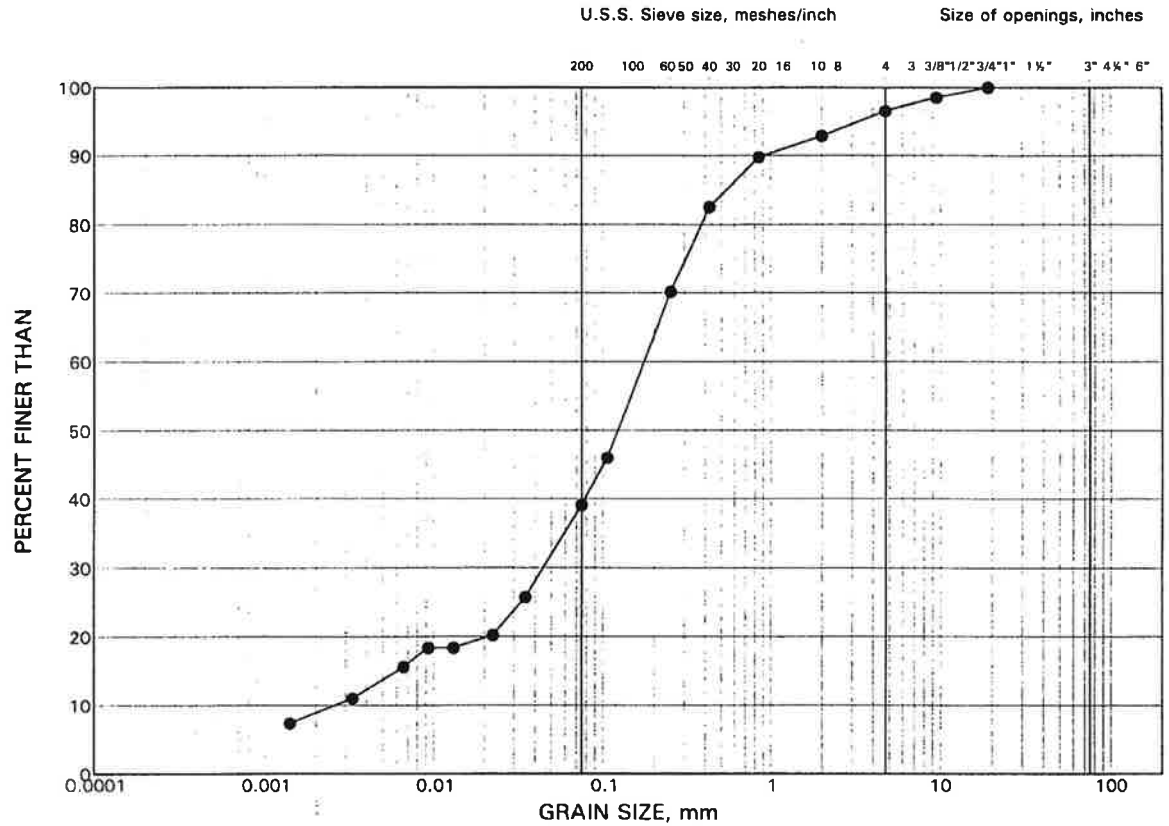
ON_MOT 0011143F.GPJ ON_MOT_GDT 14/1/02

PROJECT 001-1143F				RECORD OF BOREHOLE No B13-2				1 OF 1		METRIC				
W.P. 30-95-00				LOCATION N 4916967.4; E 288585.1				ORIGINATED BY PKS						
DIST SW HWY 400				BOREHOLE TYPE 108mm ID HOLLOW STEM AUGERS				COMPILED BY LCC						
DATUM Geodetic				DATE Feb.6-7/2001				CHECKED BY ASP						
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 X REMOULDED						
255.1	GROUND SURFACE													
0.0	Asphalt													
0.2	Sand and Gravel (Fill) Compact to very dense Brown Moist													
			1	SS	62									
			2	SS	11									
			3	SS	15									
			4	SS	25									
248.2														
6.9	Silty Sand, trace clay, some gravel, trace wood and organics Dense Brown Moist		5	SS	31									
247.5														
7.6	Silty Sand, trace clay, trace to some gravel (Till) Compact to very dense Brown Moist		6	SS	21									
			7	SS	84									
246.0														
9.1	Silty Sand to Sand, some silt, trace gravel Very dense Wet Brown		8	SS	69									0 85 15 0
			9	SS	129									
			10	SS	100/15									
			11	SS	108									
			12	SS	96									
			13	SS	110									
241.7	Thin silty clay layers present in Sample 13.													
13.4	END OF BOREHOLE													
	Notes: 1. Water level on completion of drilling at 11m depth (Elev.244.1m). 2. Water level in piezometer measured at 9m depth (Elev.246.1m) on March 15, 2001.													

GRAIN SIZE DISTRIBUTION TEST RESULT

Silty Sand Till

FIGURE 1



SILT AND CLAY SIZES			FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED			SAND SIZE			GRAVEL SIZE		SIZE

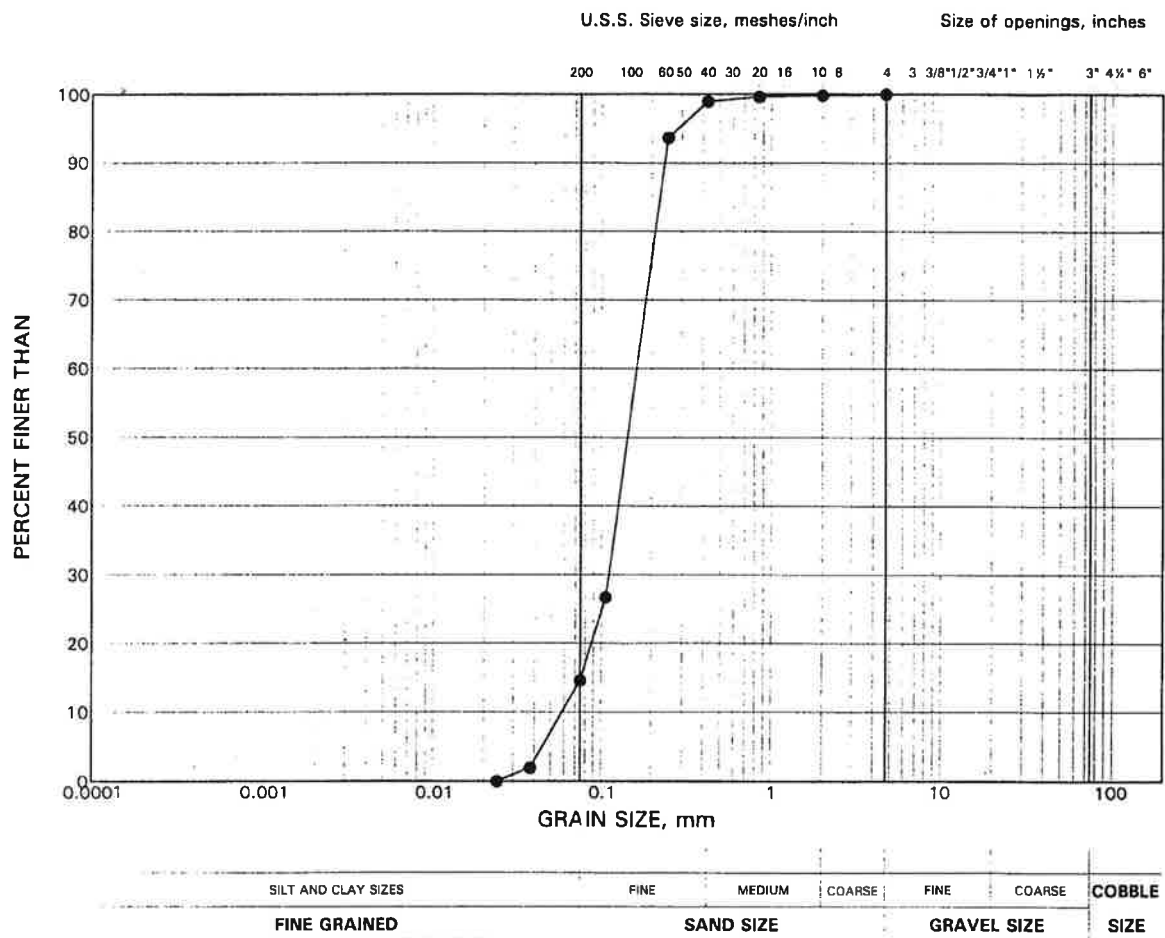
LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	B13-1	6	247.5

GRAIN SIZE DISTRIBUTION TEST RESULT

Sand, some silt

FIGURE 2



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION (m)
•	B13-2	8	245.6



Appendix D

Selected Site Photographs



Photo 1- South side of existing Sunnisdale Road Underpass



Photo 2- North side of existing Sunnisdale Road Underpass



**Photo 3- Northwest side of existing Sunnidale Road Underpass (Borehole SUN-01)
June 18, 2019**



**Photo 4- Southwest side of existing Sunnidale Road Underpass (Borehole SUN-03)
May 8, 2019**



**Photo 5- Southwest side of existing Sunnidale Road Underpass (Borehole SUN-04)
May 8, 2019**

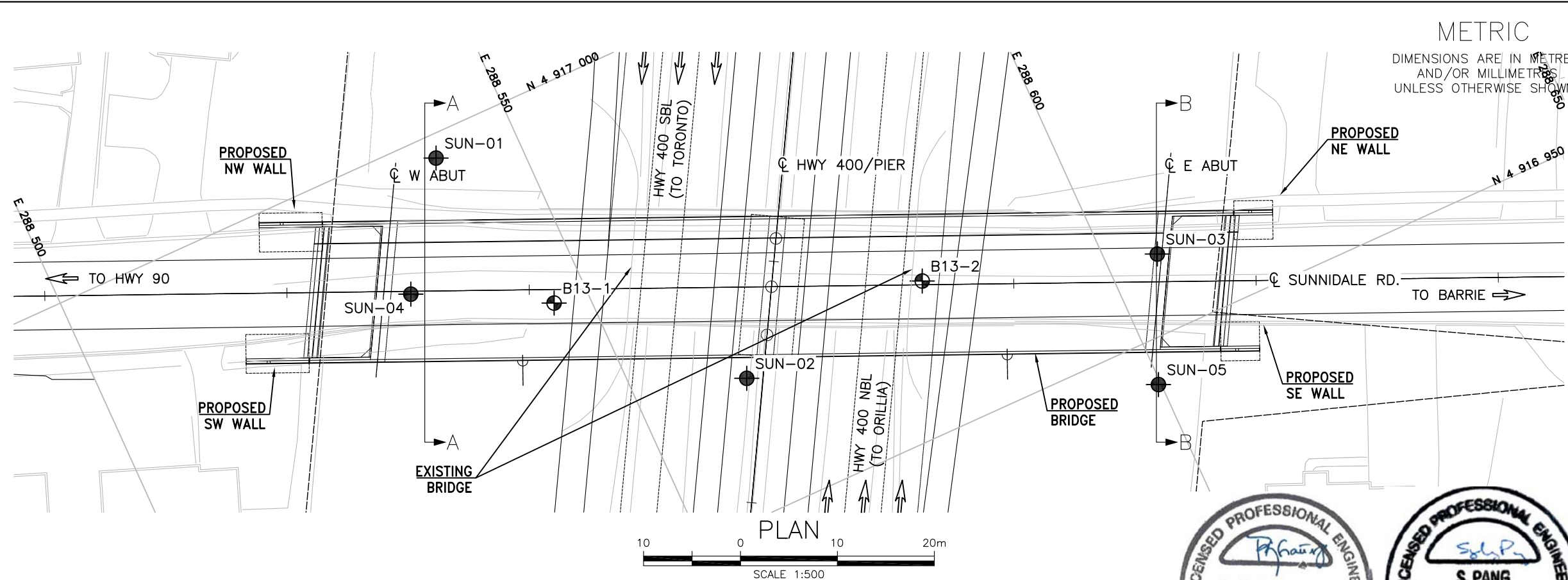


**Photo 6- Southeast side of existing Sunnidale Road Underpass (Borehole SUN-05)
June 20, 2019**



Appendix E

Borehole Locations and Soil Strata Drawing



METRIC

DIMENSIONS ARE IN METERS
AND/OR MILLIMETERS
UNLESS OTHERWISE SHOWN

CONT No
WP No 2445-15-00

HWY 400
SUNNIDALE ROAD
UNDERPASS
BOREHOLE LOCATIONS AND SOIL STRATA



SHEET

McINTOSH PERRY



THURBER ENGINEERING LTD.



Latitude: 44.392498° Longitude: -79.703831°

KEYPLAN

LEGEND

●	Borehole (Current Investigation)
⊕	Borehole (Previous Investigation)
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
W	Water Level
HA	Head Artesian Water
P	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
SUN-01	257.4	4 916 997.6	288 541.5
SUN-02	248.6	4 916 963.7	288 561.3
SUN-03	254.0	4 916 957.8	288 605.2
SUN-04	257.6	4 916 985.9	288 533.3
SUN-05	252.6	4 916 945.5	603 241.9
B13-1	256.8	4 916 978.9	288 582.7
B13-2	255.1	4 916 965.3	288 581.9

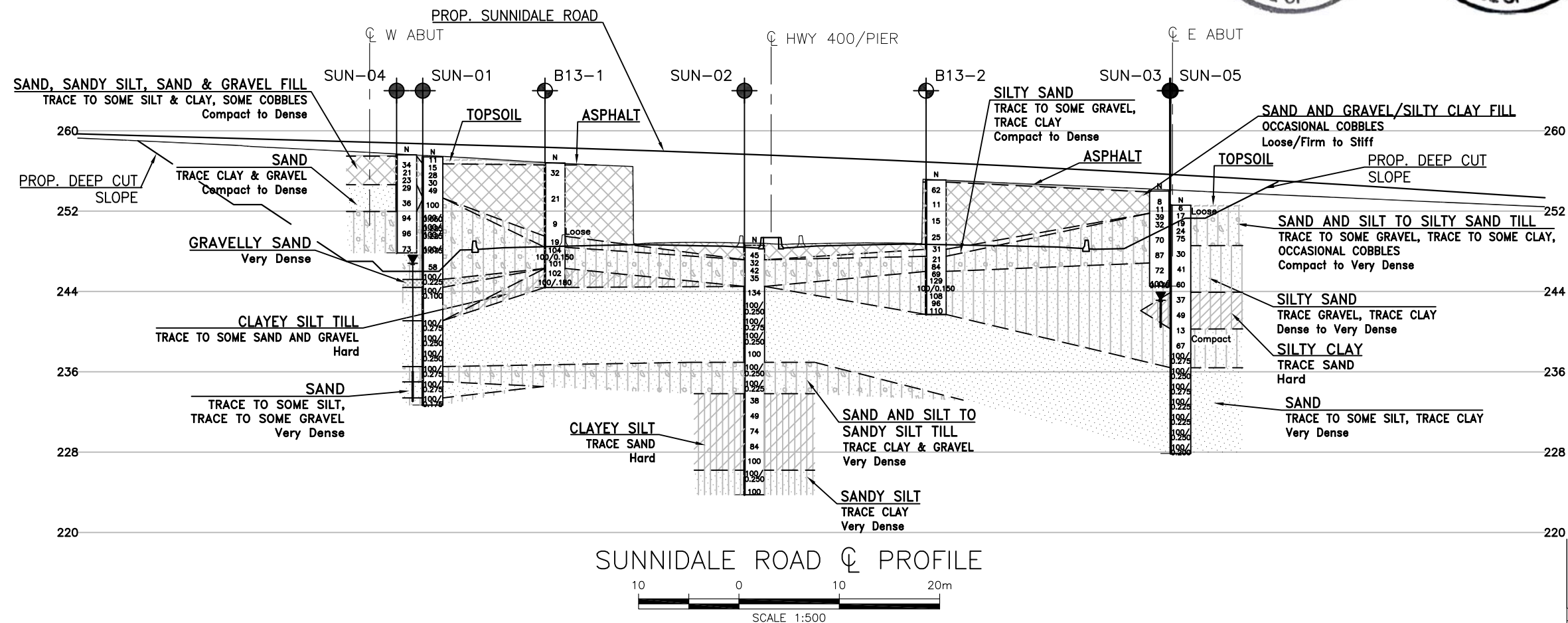
-NOTES-

- 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. 31D-771

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	RPR	CHK SKP	CODE
DRAWN	BH	CHK RPR	SITE 30-173
			LOAD
			DATE
			OCT 2021
			DWG 1

FILENAME: H:\Drafting\22000\22424\TES-22424-PLPP-SUN.dwg
PLOTDATE: 10/16/2021 12:04 PM



SUNNIDALE ROAD Q PROFILE



CONT No
WP No 2445-15-00



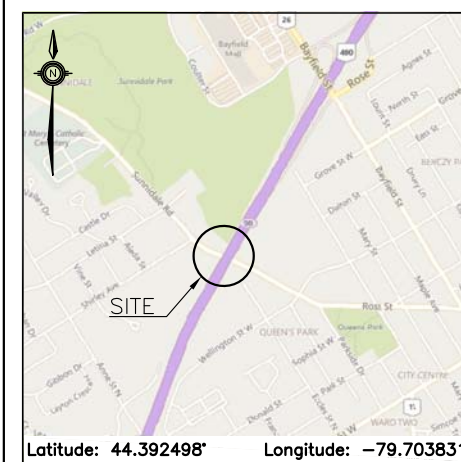
SHEET

HWY 400
SUNNIDALE ROAD
UNDERPASS
BOREHOLE LOCATIONS AND SOIL STRATA

McINTOSH PERRY



THURBER ENGINEERING LTD



KEYPLAN

LEGEND



Borehole (Current Investigation)



Borehole (Previous Investigation)

N

Blows /0.3m (Std Pen Test, 475J/blow)

CONE

Blows /0.3m (60° Cone, 475J/blow)

PH

Pressure. Hydraulic



Water Level



Head Artesian Water



Piezometer



Rock Quality Designation (RQD)



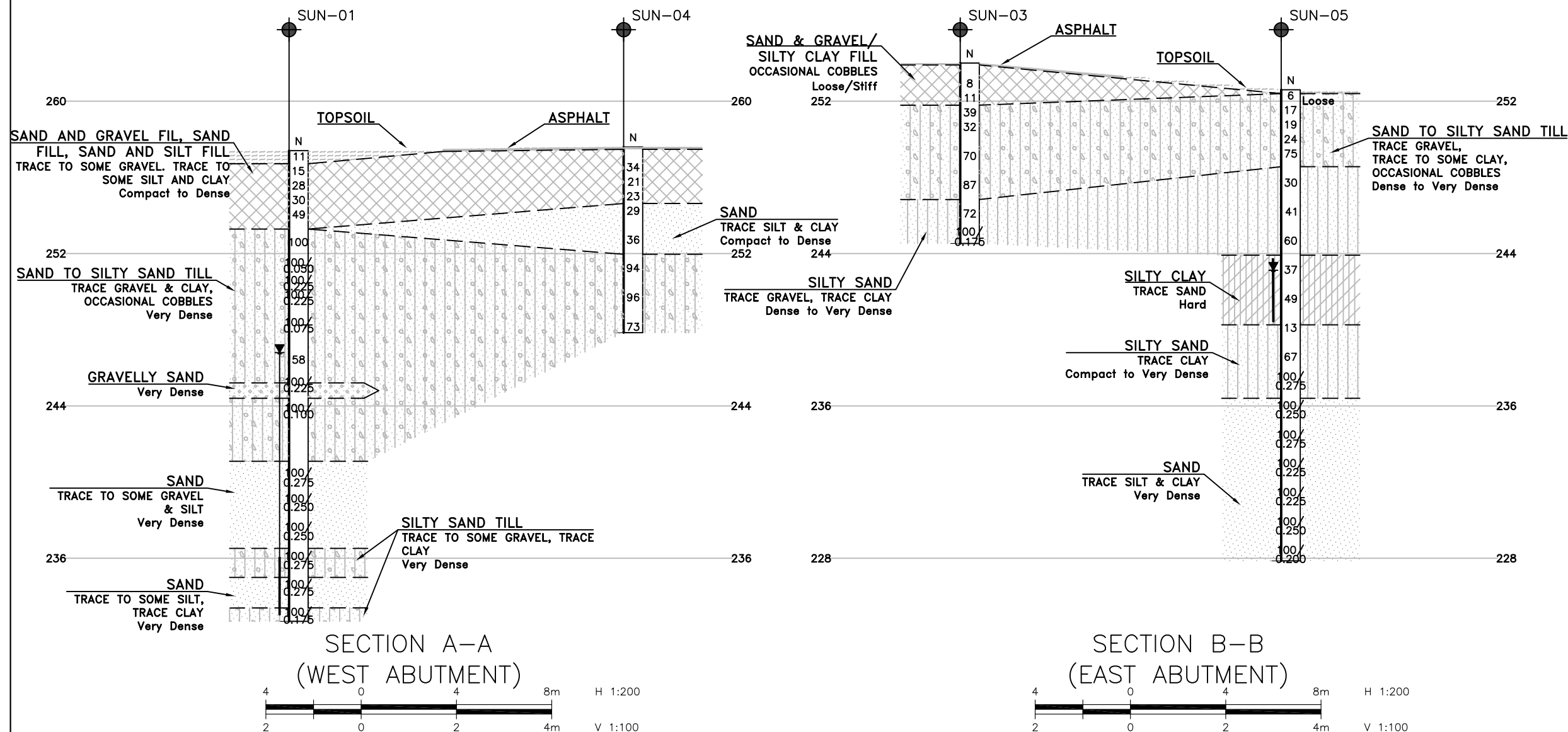
Auger Refusal

NO	ELEVATION	NORTHING	EASTING
SUN-01	257.4	4 916 997.6	288 541.
SUN-02	248.6	4 916 963.7	288 561.
SUN-03	254.0	4 916 957.8	288 605.
SUN-04	257.6	4 916 985.9	288 533.
SUN-05	252.6	4 916 945.5	603 241.
B13-1	256.8	4 916 978.9	288 582.
B13-2	255.1	4 916 965.3	288 581.

-NOTES-

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

GEOCRES No. 31D-771



REVISIONS										
	DATE	BY	DESCRIPTION							
DESIGN	RPR	CHK	SKP	CODE	LOAD	DATE	OCT 2021			
DRAWN	BH	CHK	RPR	SITE 30-173	STRUCT	DWG 2				



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. Generally less costly construction than deep foundation elements. ii. Ease of construction. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Relatively large excavations required. ii. Dewatering may be required, depending on depth of excavation. iii. May increase requirements for roadway protection. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High resistance is available for caissons founded within the very dense cohesionless soils. ii. Construction could continue in freezing weather. iii. Minimal disruption to traffic particularly at the pier since pile caps are not required. iv. Higher lateral resistance is available due to larger diameter. v. Less number of caissons is required for each foundation element than if steel piles were used. vi. Likely requires smaller work zone than other alternatives during construction. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit cost compared to other foundation options such as footings or driven piles. ii. Specialized installation measures such as temporary liners and drilling water/mud required to install caissons under the water table. iii. Potential difficulty in cleaning and inspecting bases. iv. Not suitable for integral abutments. v. Potential difficulties during augering to dislodge, remove or otherwise penetrate cobbles, boulders and hard/very dense zones within the tills. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance may be developed by driving the piles into very dense soils (100-blow" tills). ii. Comparatively short abutment stem possible. iii. Permits integral abutment design. iv. Ease of construction. v. Minimal excavation and dewatering required. vi. Construction could continue in freezing weather. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit cost compared to footings. ii. Very dense soils at shallow depth may limit length of pile. iii. Will require roadway protection for pile cap construction at piers. iv. Cobbles and boulders may be encountered in glacially derived soils that could impede pile penetration to required depths.
ABUTMENTS	FEASIBLE	TECHNICALLY FEASIBLE BUT NOT PREFERABLE	RECOMMENDED
PIER	NOT RECOMMENDED	RECOMMENDED AT PIER DUE TO SPACE RESTRICTION	FEASIBLE



Appendix G

Selected Slope Stability Output

Project Number: 22424
 Proposed Highway 400 Widening &
 Sunnidale Road Underpass Replacement
 West abutment-Concrete Retaining Wall
 Drained Analysis

Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
Blue	01-Pavement	22.8	0	35
Grey	03-Granular Backfill	20	0	30
Light Blue	04-Concrete	24	30,000	0
Dark Grey	05-Granular Pad	22	0	35
Yellow	06-Compact to Dense Sand	20	0	30
Orange	07-Very Dense Silty Sand Till	22	0	32

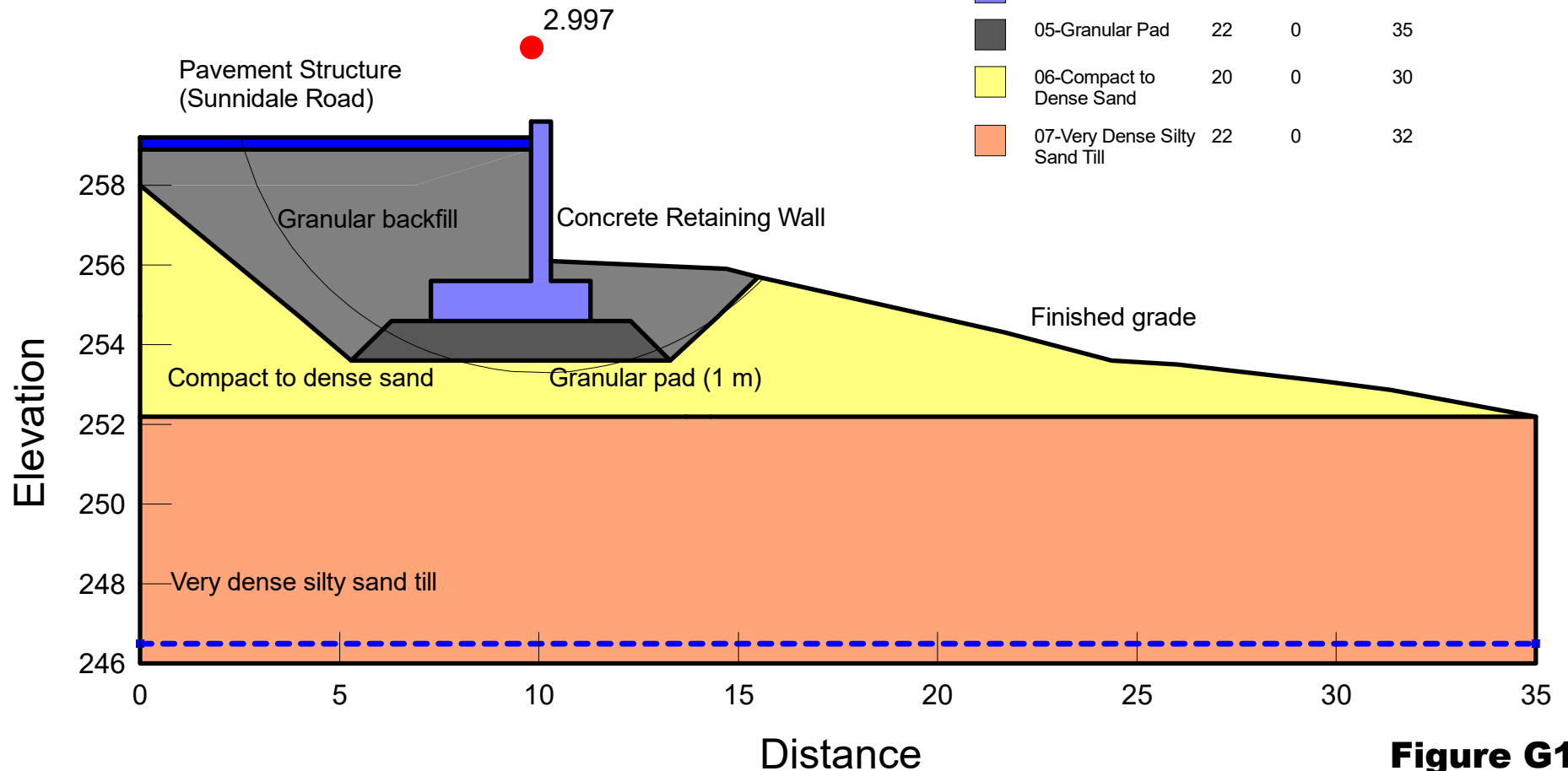


Figure G1

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Project Number: 22424
 Proposed Highway 400 Widening &
 Sunnidale Road Underpass Replacement
 West abutment-Concrete Retaining Wall
 Seismic Analysis

Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
Blue	01-Pavement	22.8	0	35
Grey	03-Granular Backfill	20	0	30
Light Blue	04-Concrete	24	30,000	0
Dark Grey	05-Granular Pad	22	0	35
Yellow	06-Compact to Dense Sand	20	0	30
Orange	07-Very Dense Silty Sand Till	22	0	32

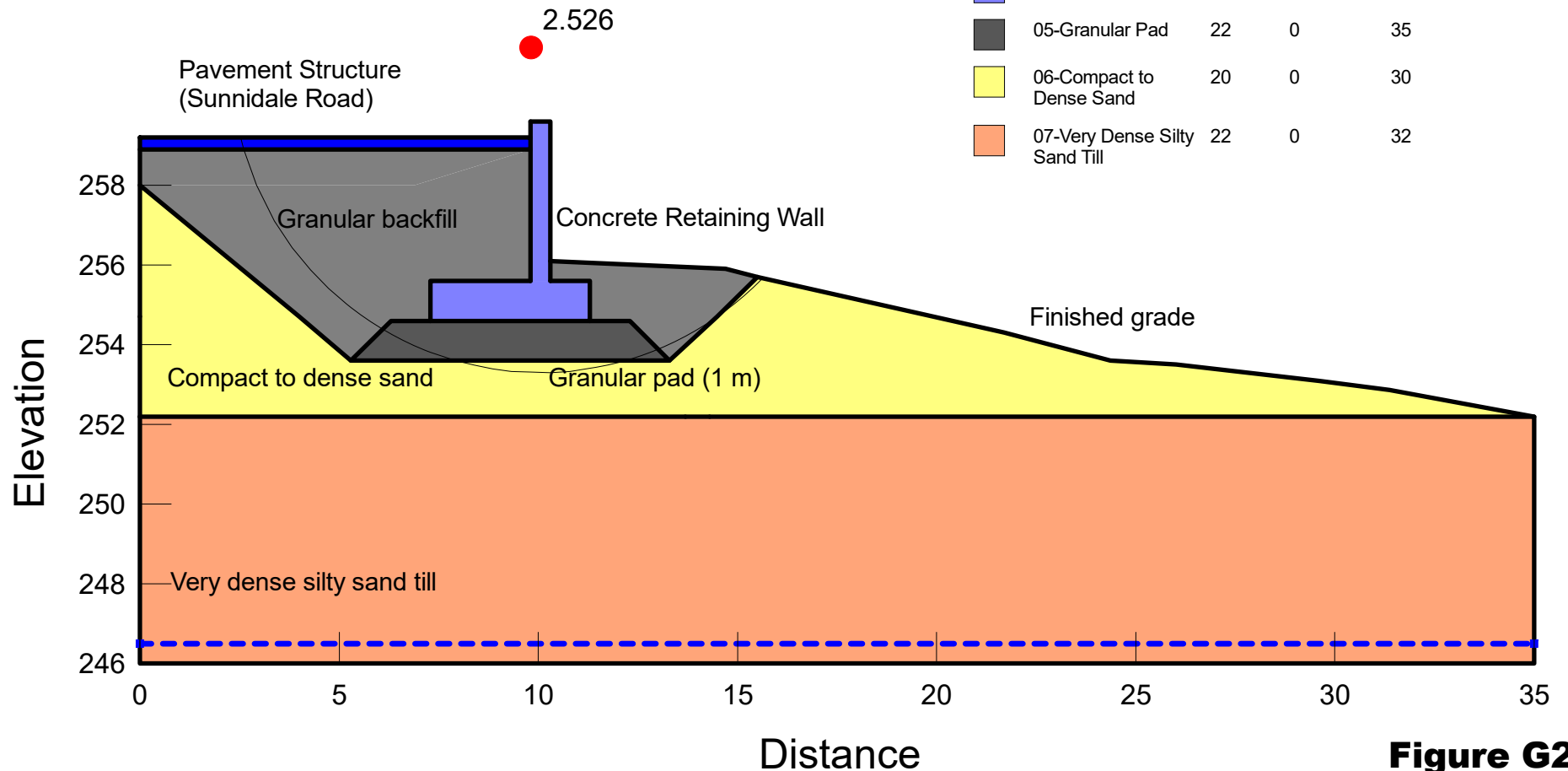
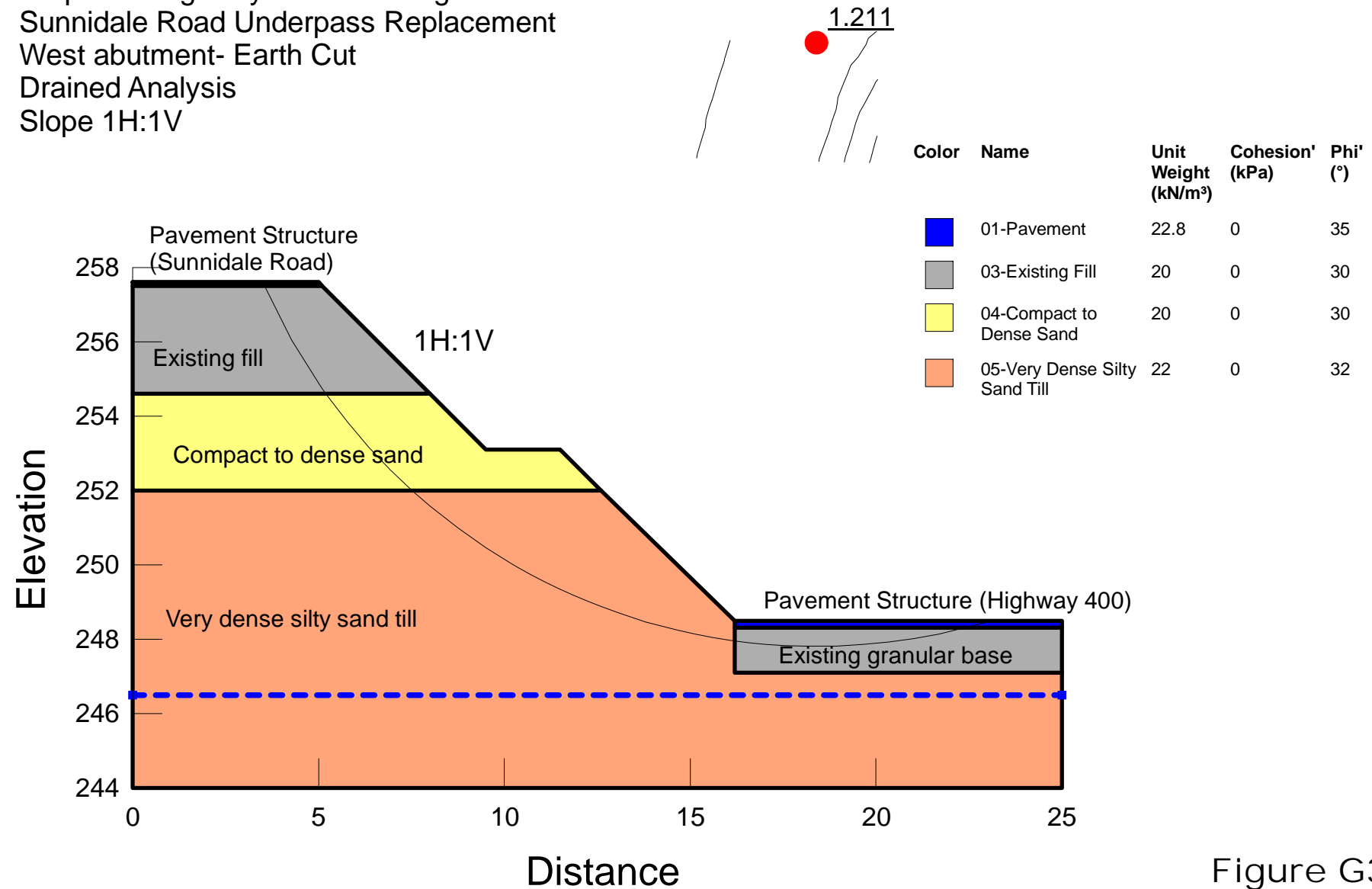


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 Sunnidale Road Underpass Replacement
 West abutment- Earth Cut
 Drained Analysis
 Slope 1H:1V



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Project Number: 22424
 Proposed Highway 400 Widening &
 Sunnidale Road Underpass Replacement
 West abutment- Earth Cut
 Seismic Analysis, PGA=0.064g
 Slope 1H:1V

1.127


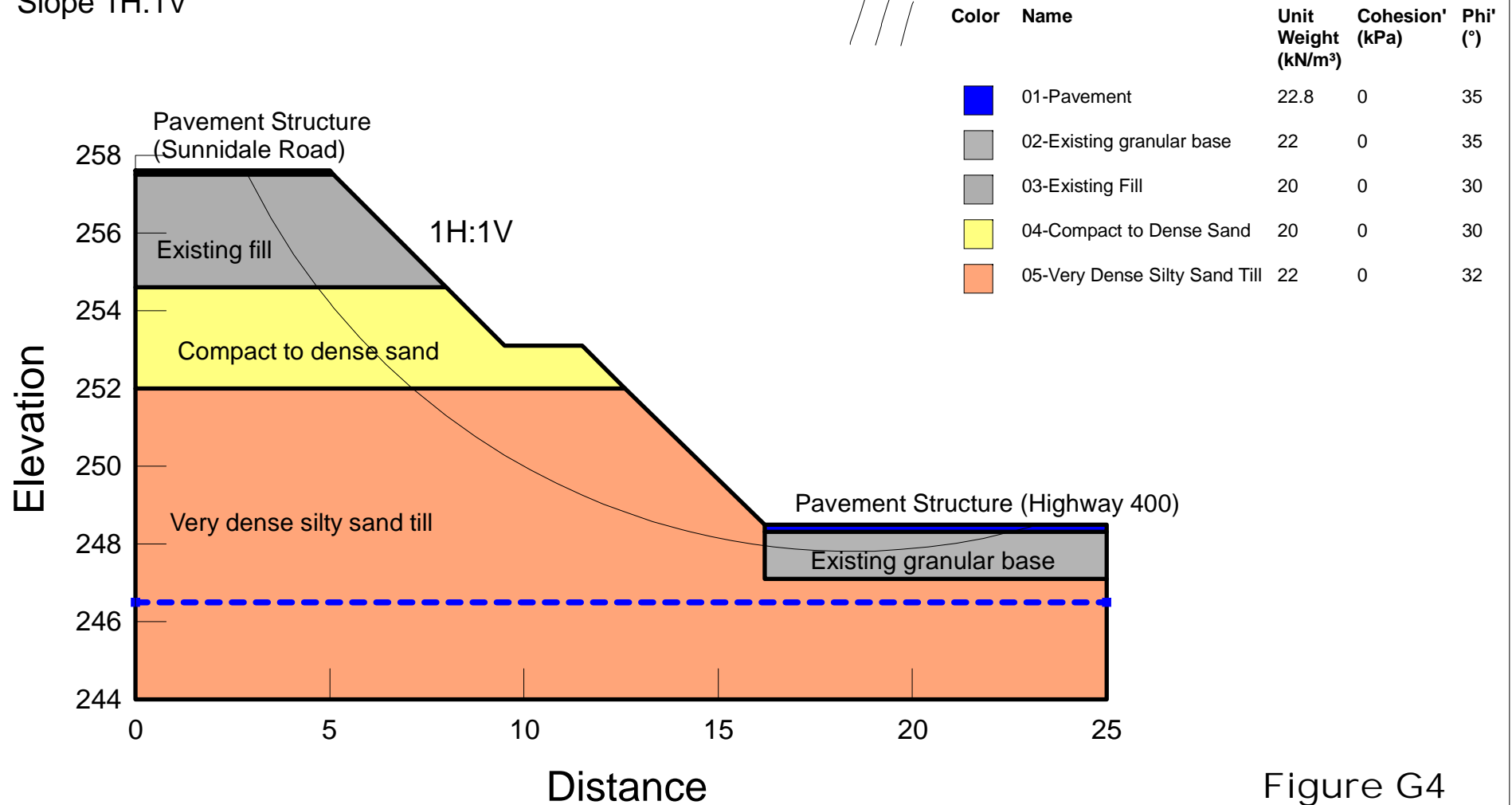
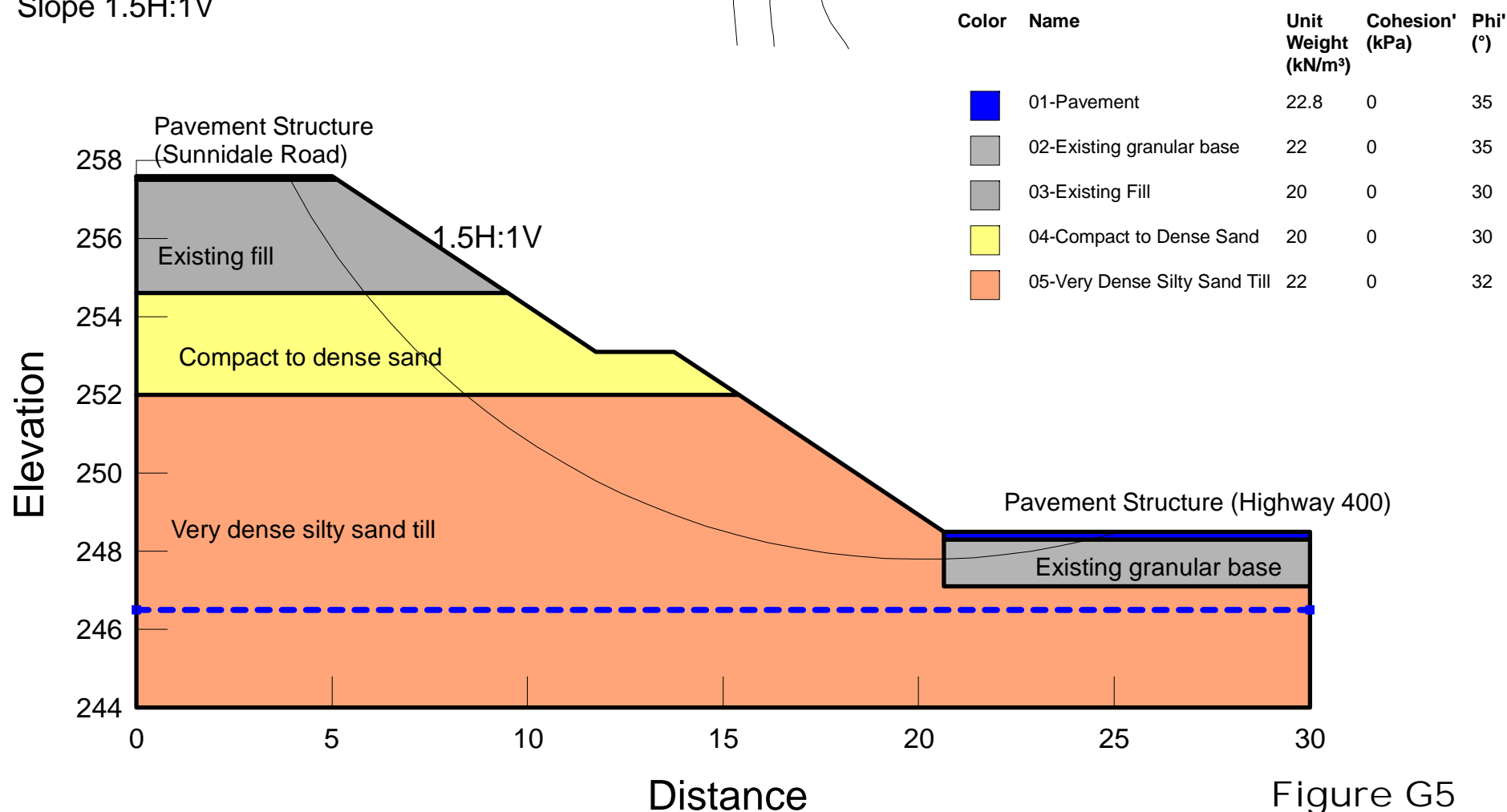



Figure G4

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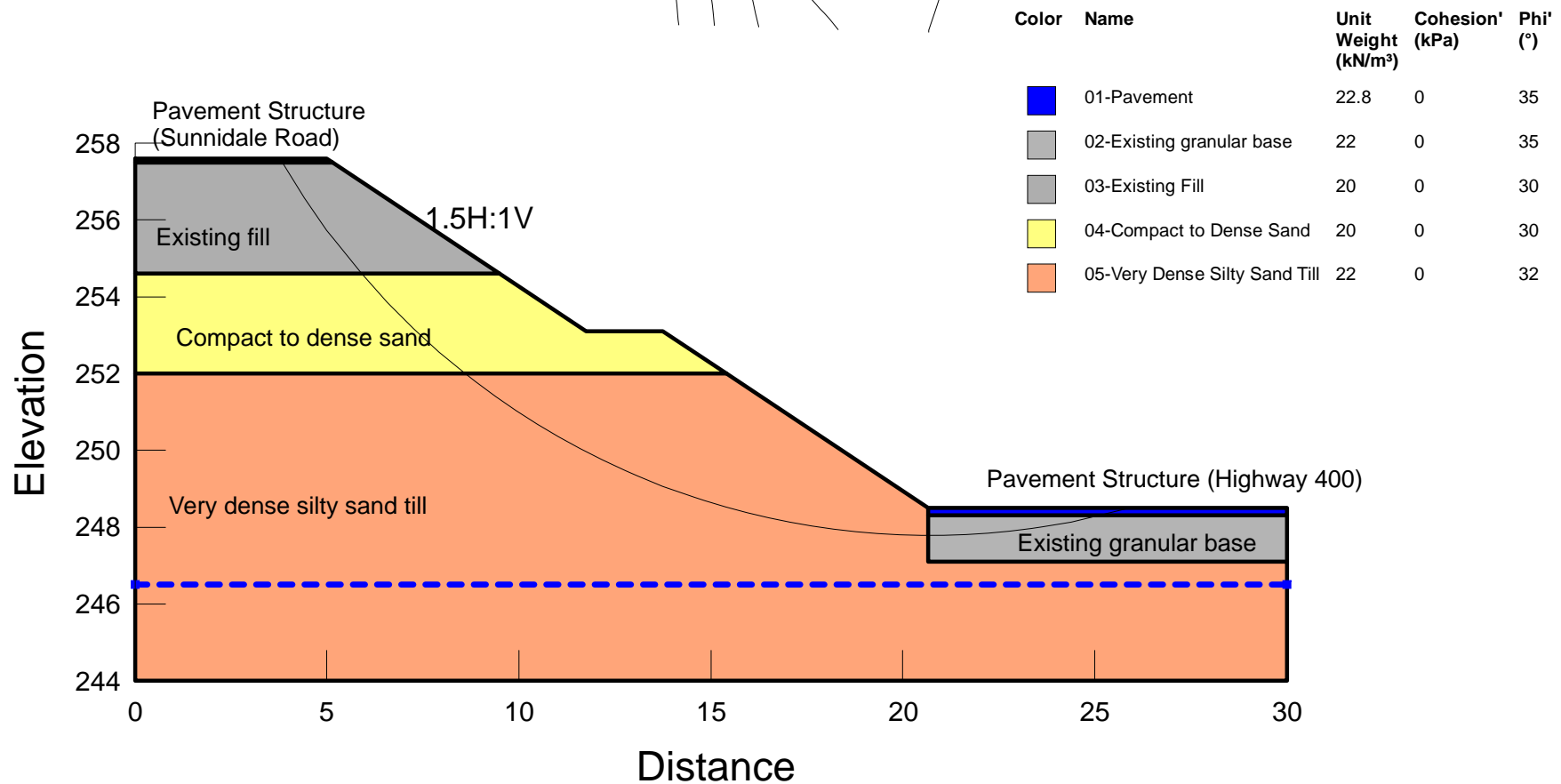
Project Number: 22424
Proposed Highway 400 Widening &
Sunnidale Road Underpass Replacement
West abutment- Earth Cut
Drained Analysis
Slope 1.5H:1V

1.503



Project Number: 22424
 Proposed Highway 400 Widening &
 Sunnidale Road Underpass Replacement
 West abutment- Earth Cut
 Seismic Analysis PGA=0.064g
 Slope 1.5H:1V

1.301



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Figure G6

Project Number: 22424
Proposed Highway 400 Widening &
Sunnidale Road Underpass Replacement
East abutment- Earth Cut
Drained Analysis
Slope 1H:1V

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
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<div></div>	02-Existing granular base	22	0	35
<div></div>	03-Existing Fill	20	0	30
<div></div>	04-Dense to very Dense Silty Sand Till	22	0	32
<div></div>	05- Dense to very dense silty sand	21	0	31

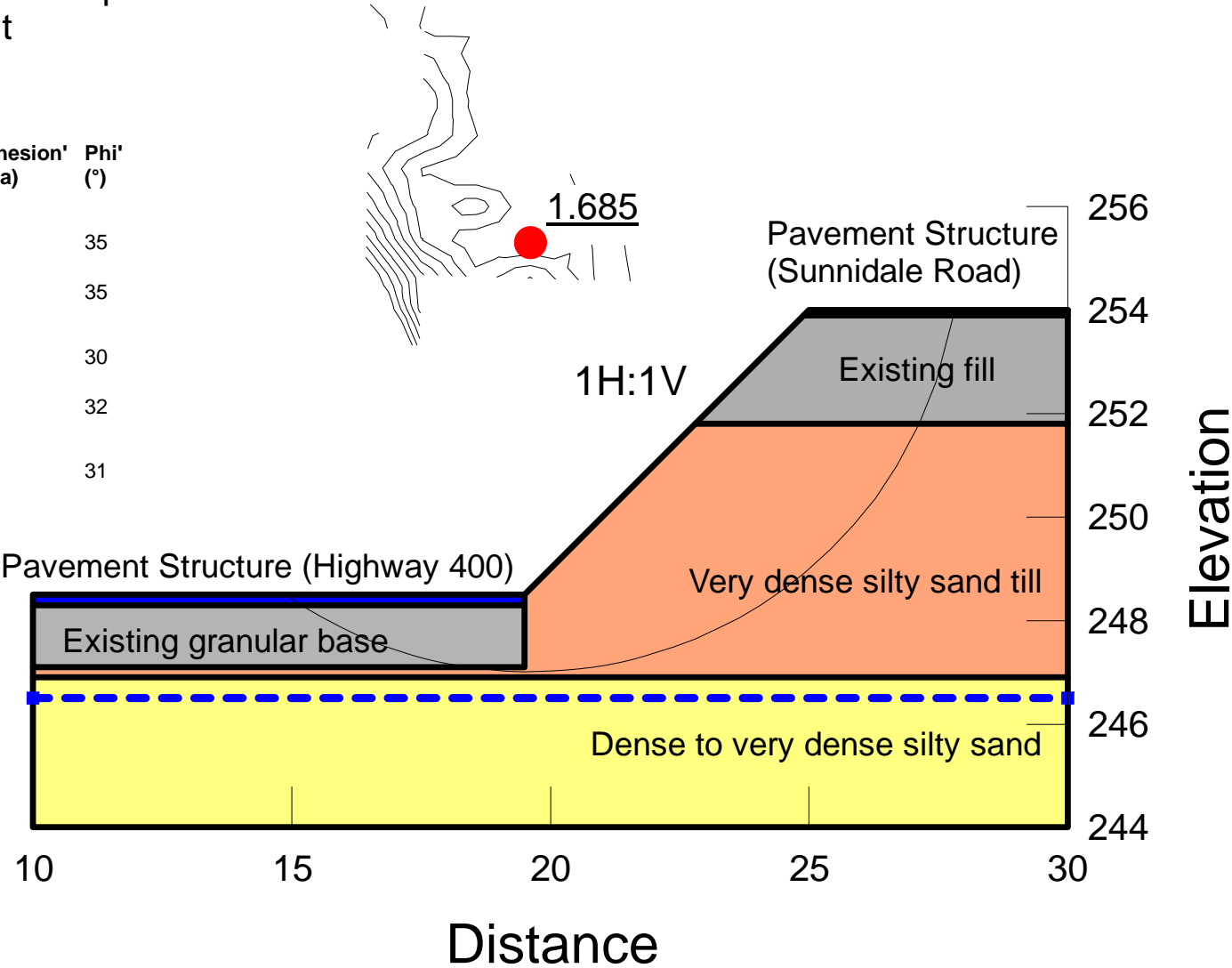
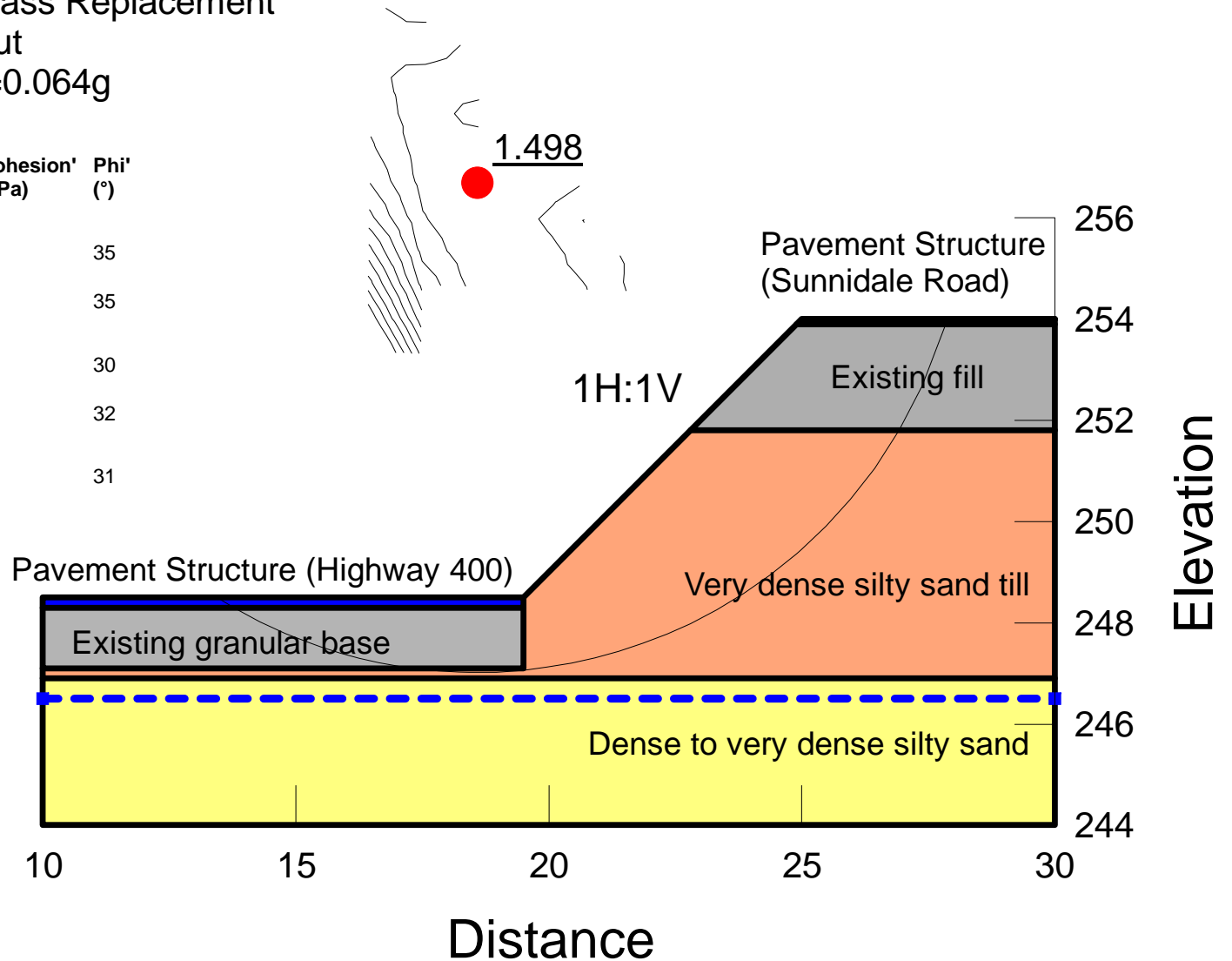


Figure G7

Project Number: 22424
 Proposed Highway 400 Widening &
 Sunnidale Road Underpass Replacement
 East abutment- Earth Cut
 Seismic Analysis, PGA=0.064g
 Slope 1H:1V

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
■	01-Pavement	22.8	0	35
■	02-Existing granular base	22	0	35
■	03-Existing Fill	20	0	30
■	04-Dense to very Dense Silty Sand Till	22	0	32
■	05- Dense to very dense silty sand	21	0	31

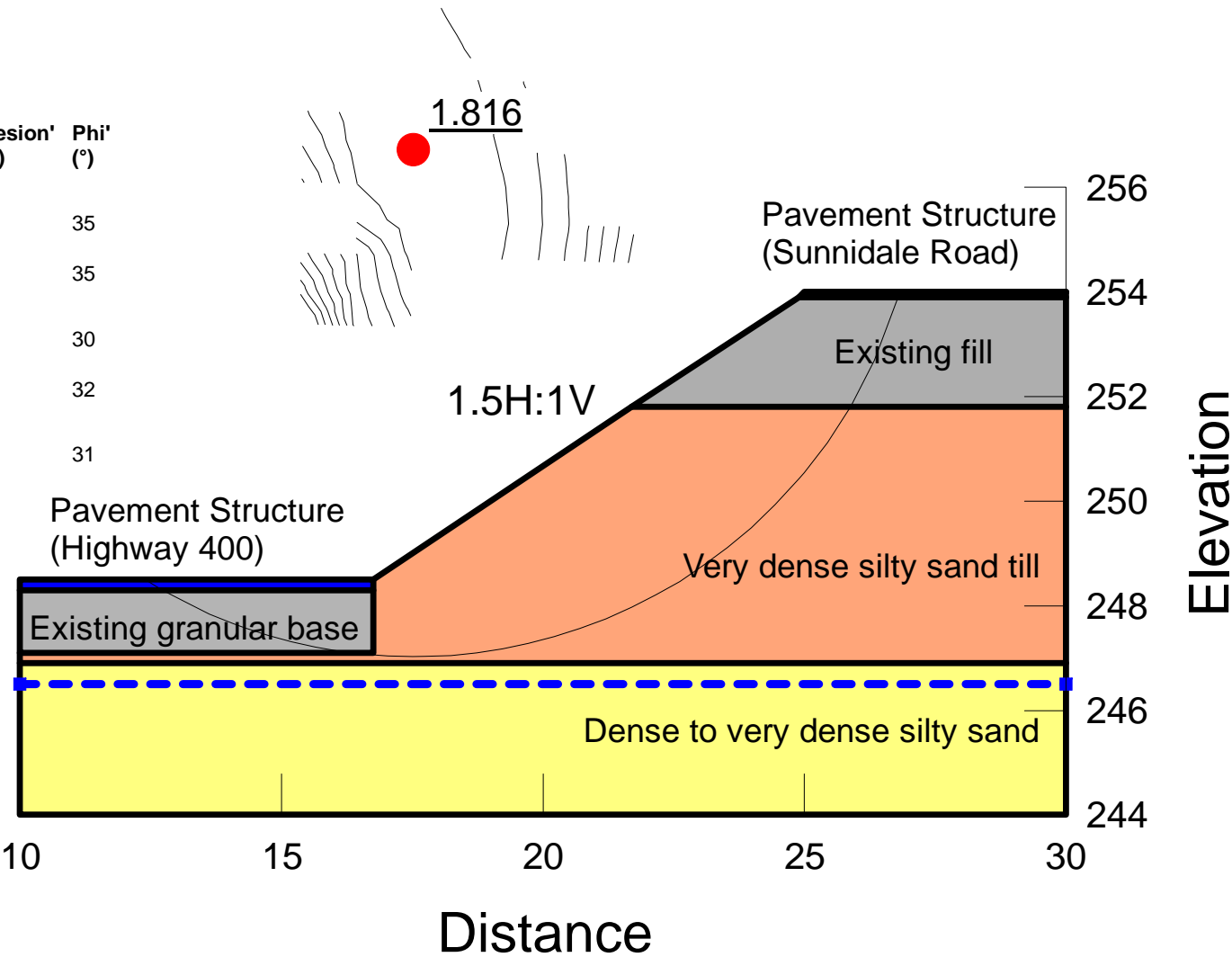


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Figure G8

Project Number: 22424
Proposed Highway 400 Widening &
Sunnidale Road Underpass Replacement
East abutment- Earth Cut
Drained Analysis
Slope 1.5H:1V

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
Blue	01-Pavement	22.8	0	35
Grey	02-Existing granular base	22	0	35
Grey	03-Existing Fill	20	0	30
Orange	04-Dense to very Dense Silty Sand Till	22	0	32
Yellow	05- Dense to very dense silty sand	21	0	31

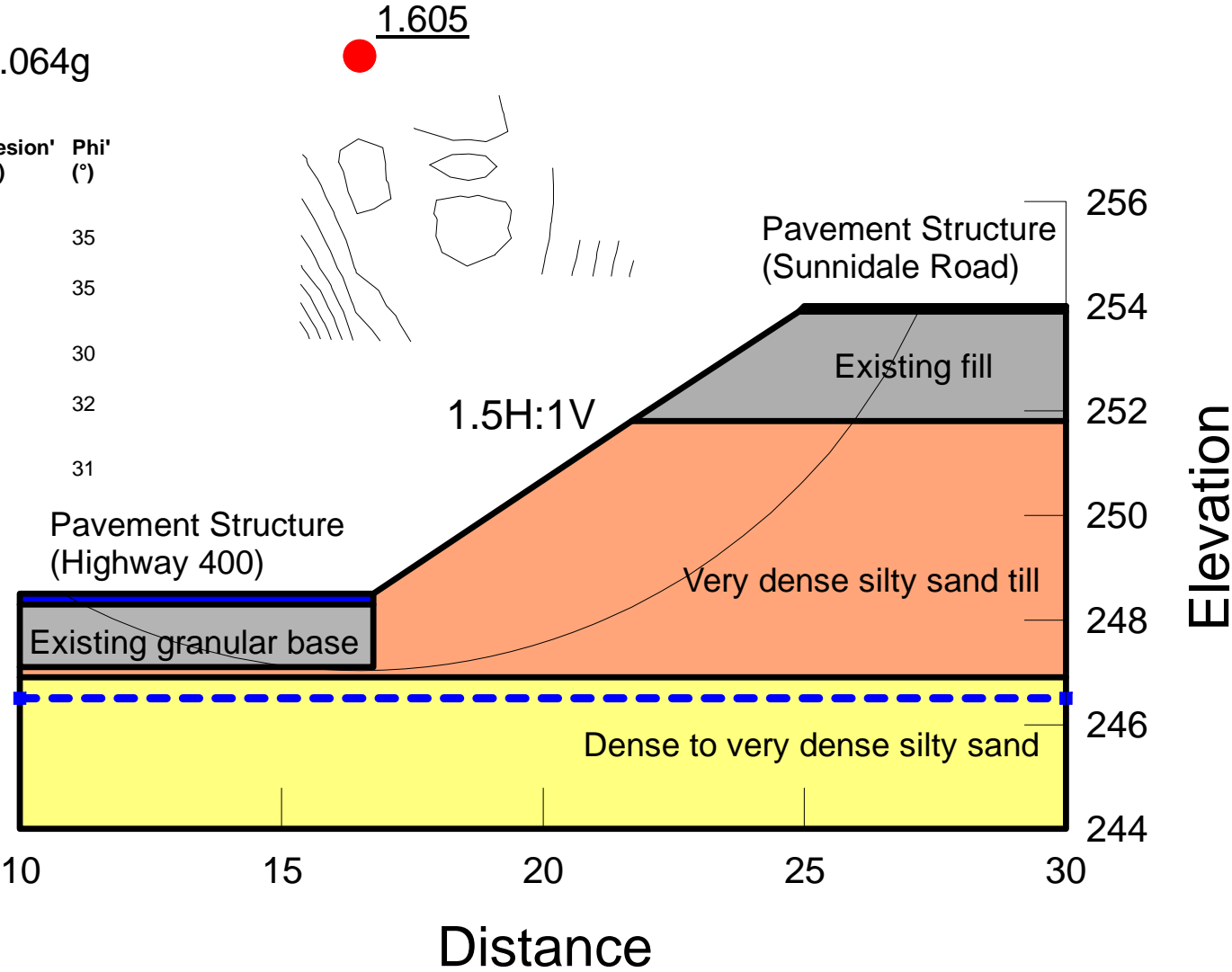


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Figure G9

Project Number: 22424
Proposed Highway 400 Widening &
Sunnidale Road Underpass Replacement
East abutment- Earth Cut
Seismic Analysis, PGA=0.064g
Slope 1.5H:1V

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
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<div></div>	02-Existing granular base	22	0	35
<div></div>	03-Existing Fill	20	0	30
<div></div>	04-Dense to very Dense Silty Sand Till	22	0	32
<div></div>	05- Dense to very dense silty sand	21	0	31



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Figure G10

Project Number: 22424
 Proposed Highway 400 Widening &
 Sunnidale Road Underpass Replacement
 West abutment- Forward slope
 Drained Analysis

Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
■	01-Pavement	22.8	0	35
■	02-Existing Fill	20	0	30
■	03-Granular Backfill	22.8	0	35
■	04-Concrete	24	30,000	0
■	05-Compact to Dense Sand	20	0	30
■	06-Very Dense Silty Sand Till	22.5	0	32

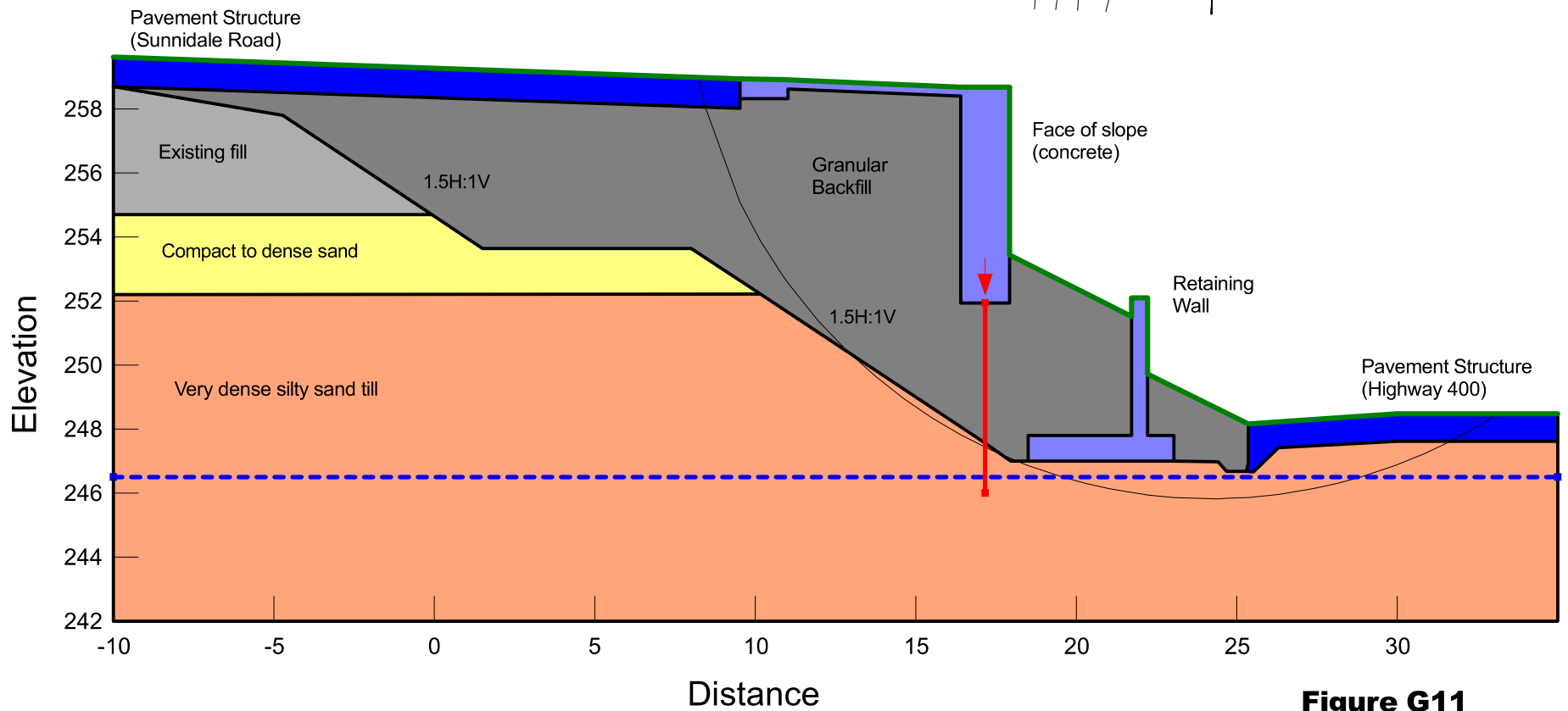


Figure G11

Project Number: 22424
Proposed Highway 400 Widening &
Sunnidale Road Underpass Replacement
West abutment- Forward slope
Seismic Analysis PGA=0.064 g

Color	Name	Unit Weight (kN/m³)	Effective Cohesion (kPa)	Effective Friction Angle (°)
Blue	01-Pavement	22.8	0	35
Grey	02-Existing Fill	20	0	30
Dark Grey	03-Granular Backfill	22.8	0	35
Light Blue	04-Concrete	24	30,000	0
Yellow	05-Compact to Dense Sand	20	0	30
Orange	06-Very Dense Silty Sand Till	22.5	0	32

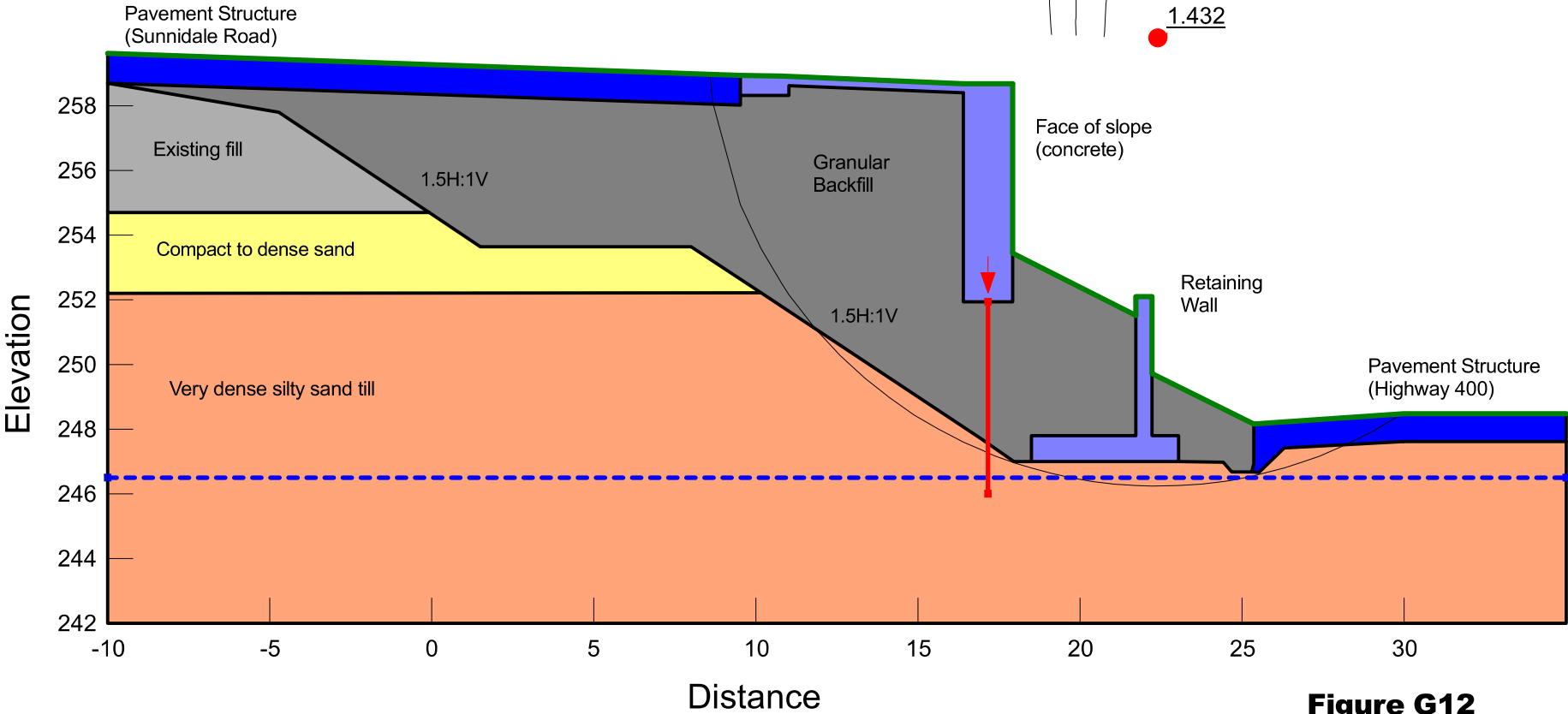
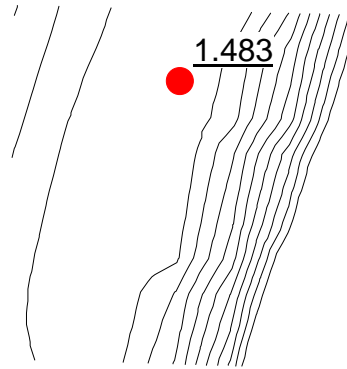


Figure G12

Project Number: 22424
 Proposed Highway 400 Widening &
 Sunnidale Road Underpass Replacement
 West abutment-Side Slope- Widening
 Sunnidale Road, Approx Station 9+945
 Drained Analysis



Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
Blue	01-Pavement	22.8	0	35
Grey	02-Existing Fill	20	0	30
Yellow	04-Compact to Dense Sand	20	0	30
Orange	05-Very Dense Silty Sand Till	22	0	32

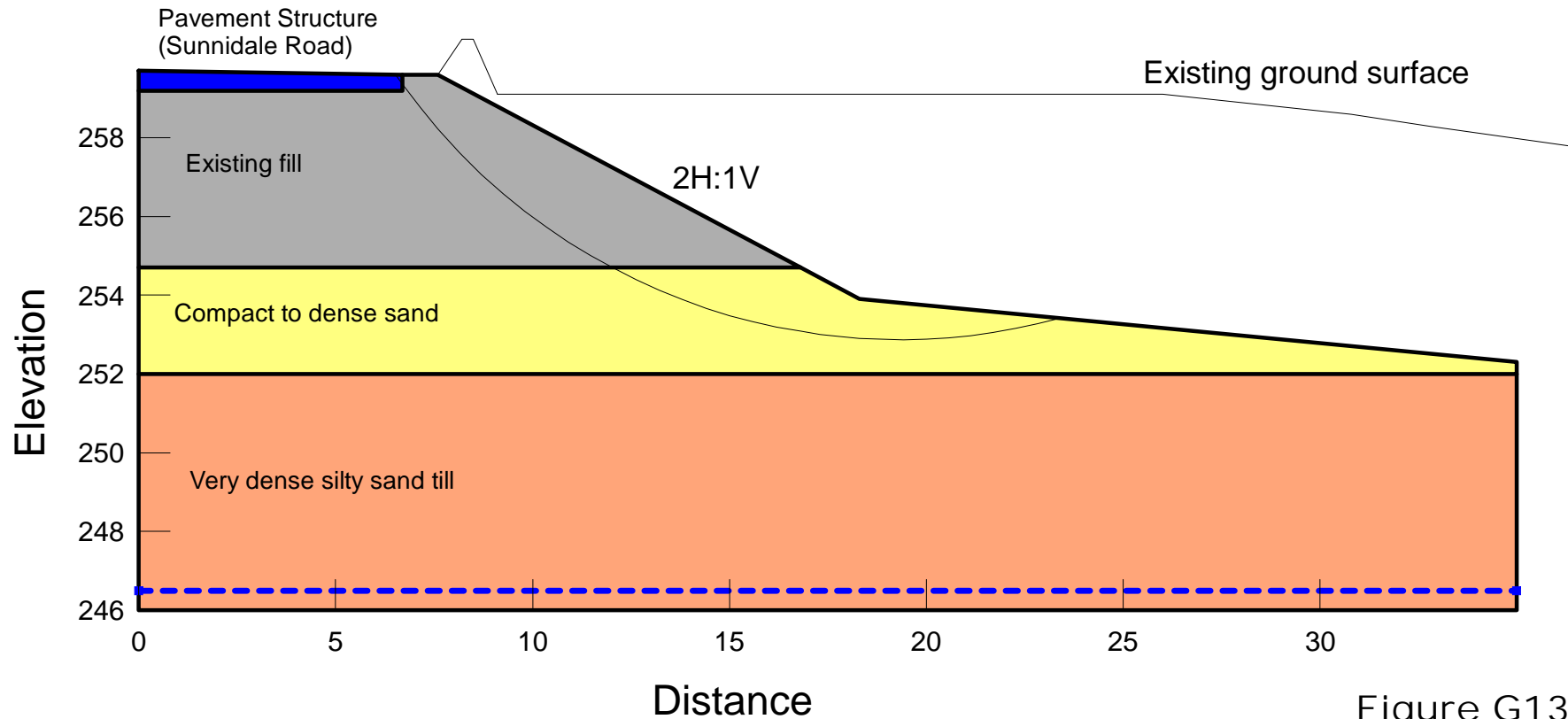


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Project Number: 22424
 Proposed Highway 400 Widening &
 Sunnidale Road Underpass Replacement
 West abutment-Side Slope- Widening
 Sunnidale Road, Approx Station 9+945
 Seismic Analysis PGA=0.064

Color	Name	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)
Blue	01-Pavement	22.8	0	35
Grey	02-Existing Fill	20	0	30
Yellow	04-Compact to Dense Sand	20	0	30
Orange	05-Very Dense Silty Sand Till	22	0	32

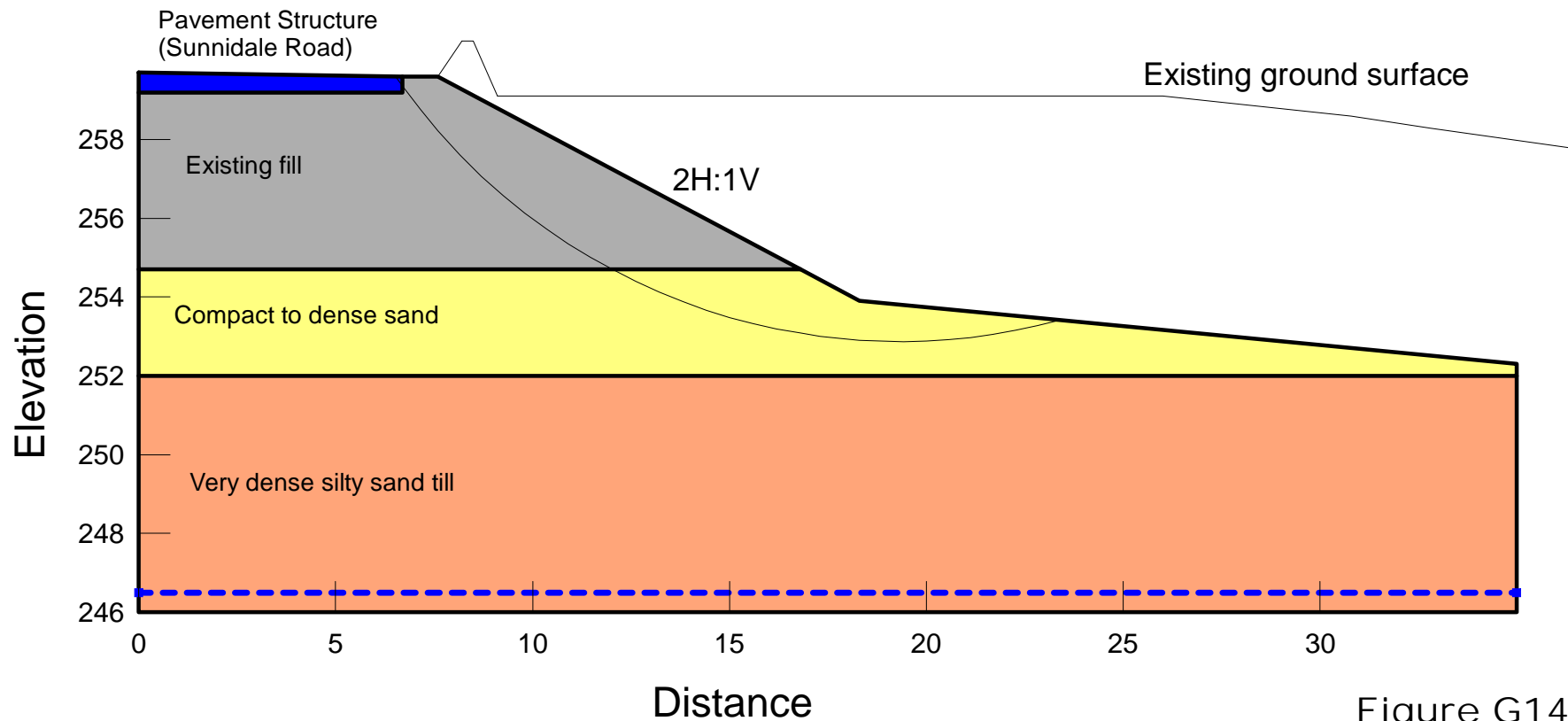
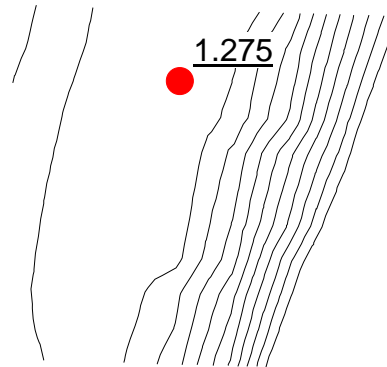


Figure G14

Project Number: 22424
Proposed Highway 400 Widening &
Sunnidale Road Underpass Replacement
East abutment- Forward Slope
Drained Analysis

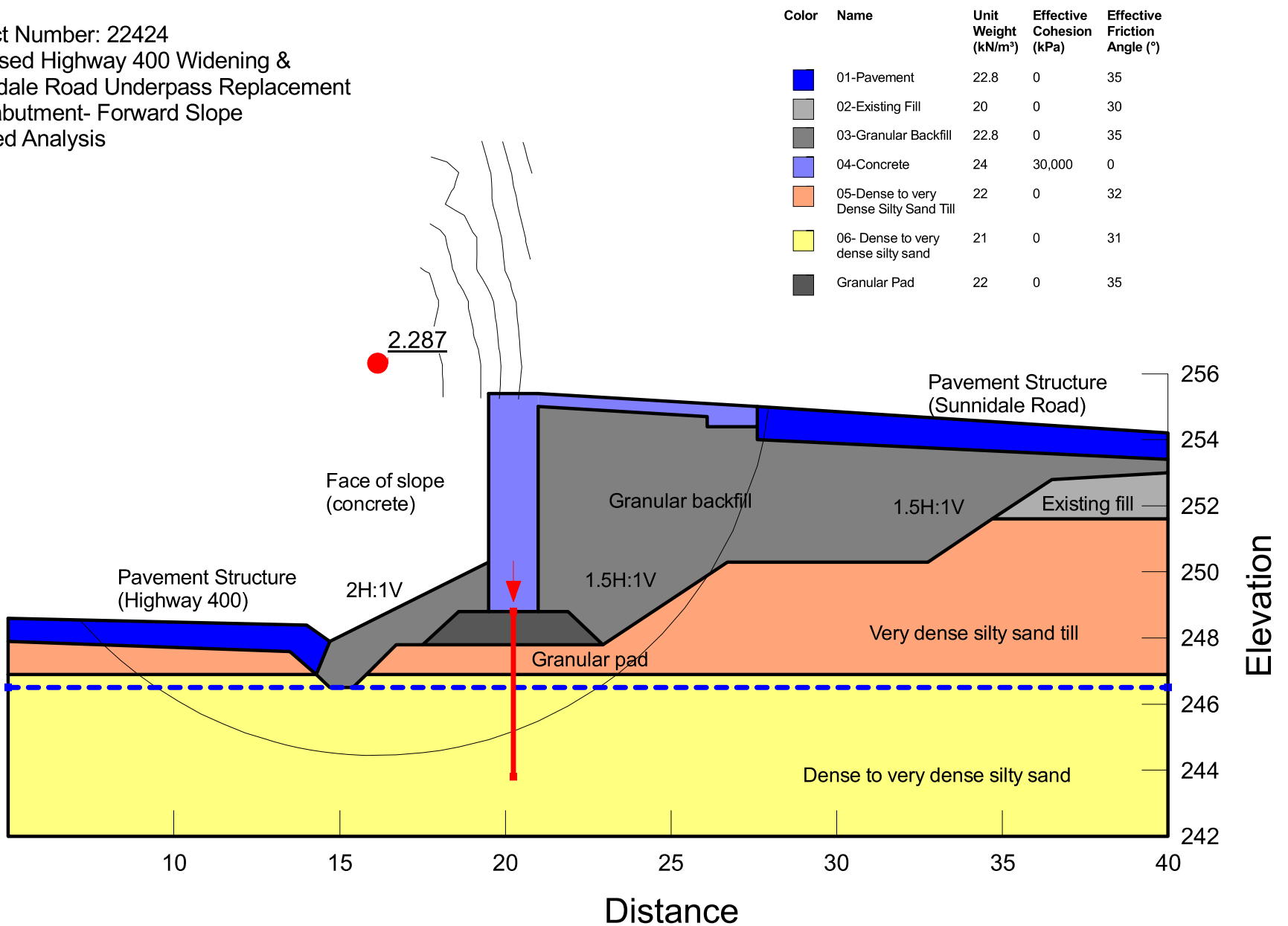


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Project Number: 22424
Proposed Highway 400 Widening &
Sunnidale Road Underpass Replacement
East abutment- Forward Slope
Seismic Analysis PGA=0.064g

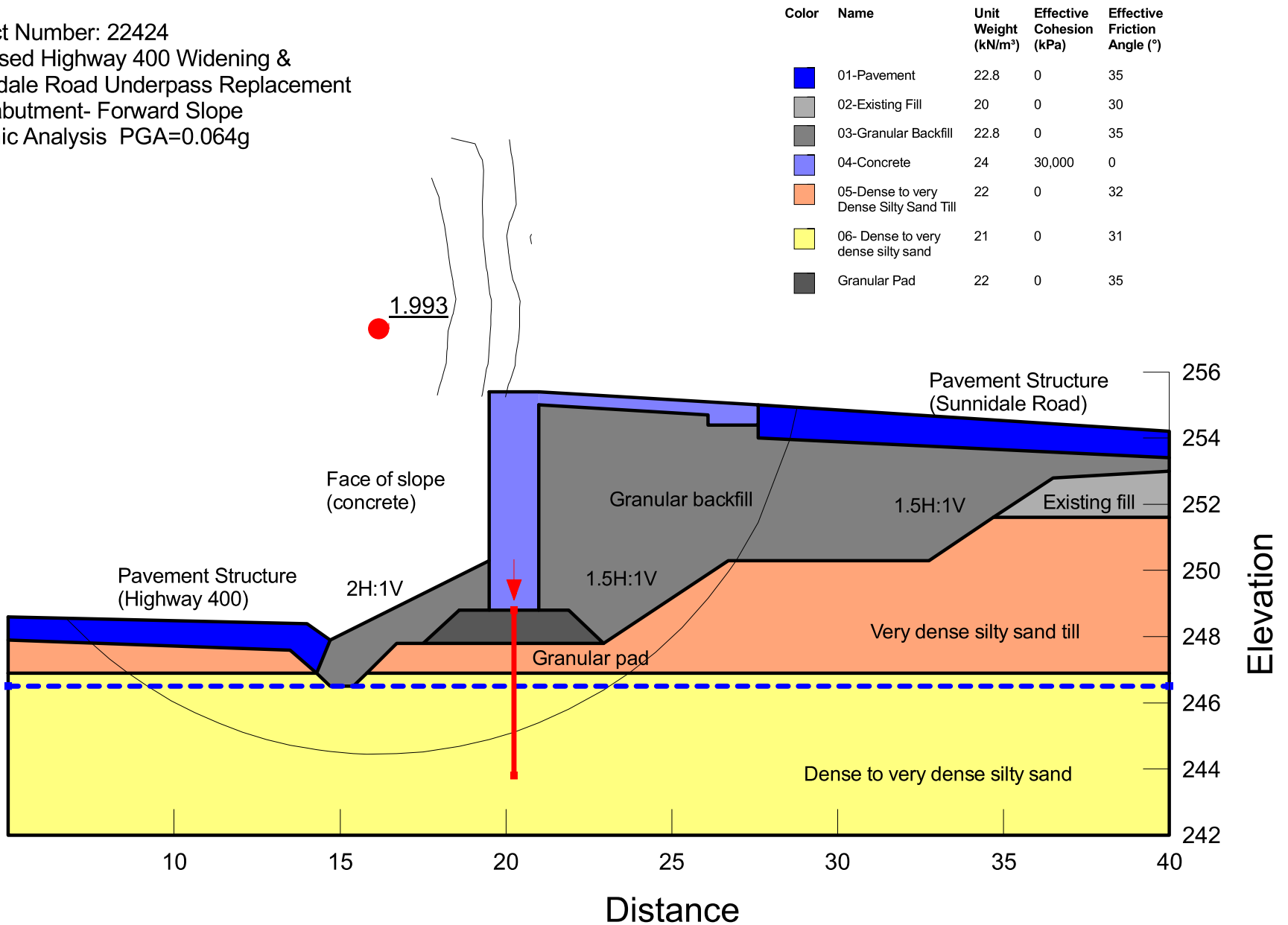


Figure G16

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Appendix H

List of OPSS Documents Nssp Wording



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 a NSSP on Pile Installation

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. Pile driving must be controlled according to the criteria specified for the site. 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.

Pre-augering shall be carried out at the west abutment pile locations to the elevation specified elsewhere in the contract prior to driving piles. Each pre-augered hole shall not be greater than 200 mm in diameter and reverse augering shall be carried out as the auger is retrieved to leave as much soil cuttings as possible inside the hole.

3. Suggested Text for NSSP on “Installation of Caissons”

All caissons shall be installed in accordance with OPSS 903. The native soil deposits generally increase in strength with depth and contain very dense zones throughout. Caisson installation through glacially derived soil deposits may encounter cobbles and/or boulders. The caisson installation equipment shall be capable of dislodging and removing any obstructions and penetrating very dense/hard layers.

Construction of caissons will require the use of temporary steel liners to support the caisson sidewalls and to provide seepage cut-off where required. Any accumulated water will need to be pumped out from the hole prior to placing concrete. Concrete shall be placed with a minimum delay after each caisson is drilled, cleaned and inspected. If accumulated water in the caisson hole cannot be removed, tremie techniques shall be used to place concrete inside the caisson hole.

4. Suggested Text for NSSP on Groundwater Control

The foundation excavations and earth cuts are expected to near or below the groundwater level. 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 may be required where the excavations extend below the groundwater table in order to maintain a dry excavation base. 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 fill surfaces by limiting construction traffic.



5. Suggested Text for NSSP on “Impact on Adjacent Structure”

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:

- The operating lanes of the Highway 400 during excavation and foundation construction at the new pier and abutments.
- Protection of the existing structure foundations, back slopes at median, and utilities (if present at this site) during excavation and pile driving.
- Protection of existing approach fills.

6. Suggested Text for NSSP on “Decommissioning of Temporary Protection Systems”

Temporary protection system (TPS) installed for the purposes of shoring and/or groundwater control for construction of the centre pier shall be left in place. It is recommended that this TPS be decommissioned by cutting to at least 1.2 m below the final grade as per OPSS.PROV 539 requirements. Decommissioning procedures shall 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 shall also be left in place and similarly decommissioned.