

**FOUNDATION INVESTIGATION AND DESIGN REPORT
C.P.R. OVERHEAD AT SCHREIBER (MILEAGE 115.85)
HIGHWAY 17
DISTRICT OF THUNDER BAY, ONTARIO**

G.W.P. 6102-10-00, SITE NO: 48 E-16

Geocres Number: 42D-33

Report to:

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TABLE OF CONTENTS

PART 1: FACTUAL INFORMATION

1	INTRODUCTION	1
2	SITE DESCRIPTION	1
3	SITE INVESTIGATION AND FIELD TESTING	2
4	LABORATORY TESTING	3
5	DESCRIPTION OF SUBSURFACE CONDITIONS	3
5.1	Asphalt	3
5.2	Sand to Sand and Gravel Fill	4
5.3	Sand to Sand and Gravel	4
5.4	Water Levels	5
6	MISCELLANEOUS	6

PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7	GENERAL	7
8	ASSESSMENT OF EXISTING FOUNDATIONS	8
8.1	Abutment Foundations	8
8.2	Pier Foundations	9
8.3	Frost Cover	9
9	ABUTMENT BACKFILL AND LATERAL EARTH PRESSURES	9
10	SEISMIC CONSIDERATIONS	11
11	CUT SLOPES	11
12	EROSION PROTECTION	12
13	EXCAVATION AND GROUNDWATER CONTROL	12
14	ROADWAY PROTECTION	13
15	CONSTRUCTION CONCERNS	13
16	CLOSURE	14

Appendices

Appendix A	Record of Borehole Sheets
Appendix B	Laboratory Test Results
Appendix C	Site Photographs
Appendix D	List of SPs and OPSS
Appendix E	Slope Stability Analyses
Appendix F	Borehole Locations and Soil Strata Drawing

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the existing CPR Overhead structure (Mileage 115.85) on Highway 17 in Schreiber, Ontario.

The purpose of this investigation was to explore the subsurface conditions at the site and, based on the data obtained, to provide a borehole location plan, record of borehole sheets, a stratigraphic profile, laboratory test results and written descriptions of the subsurface conditions. A model of the subsurface conditions was developed from the data obtained in the course of the investigation.

Thurber carried out the investigation as a sub-consultant to MMM Group Limited (formerly McCormick Rankin), under the Ministry of Transportation Ontario (MTO) Agreement Number 6010-E-0011.

2 SITE DESCRIPTION

The CPR Overhead grade separation is located on Highway 17 approximately 3 km east of the village of Schreiber and 9 km west of Terrace Bay. The existing structure has three spans, a total length of 51.4 m, a width of 11.6 m, and a skew of approximately 35°. At the crossing, the railway runs in a cut with a depth of approximately 8.0 m below highway grade.

The surrounding lands are generally undeveloped and forested. The terrain comprises high rounded hills, and a large hill with bedrock exposures is located immediately to the northeast of the site.

Photographs in Appendix C show the general nature of the site and the existing structure.

The site lies within the physiographic region known as the Wawa Subprovince of the Superior Province of the Canadian Shield. The area is underlain by igneous to intermediate metavolcanic bedrocks which are overlain by glaciofluvial deposits of sand and gravel.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out between June 12 and 16, 2013. The investigation comprised drilling and sampling four boreholes, identified as Boreholes SCH-01 to SCH-04, to depths between 10.1 m and 30.5 m below existing road grade.

Boreholes SCH-01 and SCH-02 were drilled at the west approach and near the west abutment, respectively. Boreholes SCH-03 and SCH-04 were drilled near the east abutment and at the east approach, respectively. The approximate borehole locations are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix F.

The borehole locations were marked in the field and utility clearances were obtained prior to drilling. The coordinates and ground surface elevations at the boreholes were determined from base plan drawings provided by MMM Group.

A truck-mounted CME75 drill rig was used to advance the boreholes using a combination of hollow-stem augers and wash-boring with NW casing. NQ coring was employed to penetrate cobbles locally. Soil samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). Dynamic cone penetration tests (DCPTs) were undertaken at the base of Boreholes SCH-02 and SCH-03 to depths of 20.1 m and 31.7 m. In addition, DCPTs were carried out adjacent to Boreholes SCH-02 and SCH-03 to refusal at depths of 11.9 m and 7.0 m.

The drilling and sampling operations were supervised on a full time basis by a member of Thurber's technical staff. The supervisor 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 drilling operations. Observations at completion of drilling may not be representative of groundwater conditions as wash-boring methods introduced water into the boreholes. Standpipe piezometers were installed in two boreholes to monitor the groundwater level at the site. The piezometers were decommissioned following monitoring and the boreholes without piezometers were backfilled in general accordance with MOE Regulation 903. Completion details of the piezometers and boreholes are summarized in Table 3.1.

Table 3.1 – Borehole Completion Details

Foundation Unit	Borehole	Piezometer Tip Depth/ Elevation (m)	Completion Details
West Approach	SCH-01	None installed	Borehole backfilled with bentonite holeplug to 0.6 m, concrete from 0.6 m to 0.1 m, then asphalt cold patch to surface.
West Abutment	SCH-02	19.5/ 284.8	Borehole backfilled with sand to 16.2 m, bentonite holeplug from 16.2 m to 0.6 m, sand from 0.6 m to 0.1 m, then asphalt cold patch to surface.
East Abutment	SCH-03	31.7/ 272.6	Borehole backfilled with sand to 28.4 m, bentonite holeplug from 28.4 m to 0.5 m, sand and gravel from 0.5 m to 0.15 m, then asphalt cold patch to surface.
East Approach	SCH-04	None installed	Borehole backfilled with bentonite holeplug to 0.9 m, concrete from 0.9 m to 0.2 m, then asphalt cold patch to surface.

4 LABORATORY TESTING

All recovered soil samples were subjected to Visual Identification (VI) and natural moisture content determination. Selected samples were also subjected to grain size distribution analyses (sieve and hydrometer). The results of this testing program are summarized on the Record of Borehole sheets included in Appendix A and on the figures presented in Appendix B.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets included in Appendix A. Details of the encountered soil stratigraphy are presented in these sheets and on the “Borehole Locations and Soil Strata” drawing included in Appendix F. An overall 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.

The site stratigraphy typically comprises an asphalt layer over sand and gravel fill, overlying native sands and gravels. More detailed descriptions of the individual strata are presented below.

5.1 Asphalt

Asphalt was encountered on the roadway surface in all boreholes drilled. The asphalt was 100mm to 125mm thick.

5.2 Sand to Sand and Gravel Fill

The asphalt layer is underlain by brown sand to sand and gravel fill. The fill extended to a depth of 1.5 m (Elev. 302.7 to 302.8) at the west approach and 2.9 to 3.0 m (Elev. 301.2 to 301.3) at the east approach. Rock coring equipment was used to penetrate cobbles within the fill in Borehole SCH-03.

SPT N-values recorded in the fill ranged from 13 to 62 blows for 0.3 m penetration, indicating a compact to very dense relative density. The moisture content for the granular fill ranged from 4% to 21%.

The results of laboratory grain size analysis testing of two samples of fill are presented on the Record of Borehole sheets included in Appendix A and on the grain size distribution curves shown in Figure B1 of Appendix B. The results are also summarized below.

Gravel %	61 to 63
Sand %	35 to 38
Silt & Clay %	1 to 2

5.3 Sand to Sand and Gravel

A native cohesionless deposit varying in gradation from sand to sand and gravel was encountered beneath the fill in all boreholes. The native deposit contains occasional cobbles and was described as brown to grey. A layer of silt and sand was encountered in the deposit at 28.9 m depth in Borehole SCH-03.

All boreholes were terminated within this deposit at depths of 10.1 m to 30.5 m (Elev. 294.1 to 273.8). DCPT testing was advanced to refusal a further 0.6 m and 1.2 m below the base of Boreholes SCH-02 and SCH-03.

SPT N-values between 11 and 86 blows per 0.3 m penetration were recorded, indicating a compact to very dense relative density. Higher values of 50 blows for 0.0 to 0.15 m of penetration were obtained locally on probable cobbles. The moisture content of the layer ranged from 1% to 26%.

Grain size distribution curves for tested samples of the deposit are shown on Figures B2 to B5 of Appendix B. The results are summarized on the Record of Borehole sheets included in Appendix A, and in the following table:

Soil Particles	Sand (%)	Sand & Gravel (%)	Silt & Sand (%)
Gravel	0 to 10	27 to 63	0
Sand	82 to 94	35 to 69	54
Silt	6 to 12	2 to 8	44
Clay			2

5.4 Water Levels

The water levels in the boreholes were measured upon completion of drilling. Wash boring methods were used to advance the boreholes and therefore these levels may not be stabilized. Standpipe piezometers were installed in two boreholes to measure the groundwater level after drilling. The measured water levels are summarized in Table 5.1.

Table 5.1 – Water Level Measurements

Borehole	Date	Water Level		Comment
		Depth (m)	Elev. (m)	
SCH-01	June 12, 2013	9.4	294.8	In open borehole
SCH-02	June 17, 2013	15.2	289.1	In piezometer upon completion
SCH-03	June 13, 2013	24.6	279.7	In open borehole
	June 14, 2013	24.6	279.7	In piezometer
SCH-04	June 15, 2013	9.4	294.7	In open borehole

The above values 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 the spring snowmelt or after periods of heavy rainfall.

6 MISCELLANEOUS

Borehole locations were selected and established in the field by Thurber Engineering Ltd. The coordinates and the ground surface elevations at the boreholes were determined from base plan information provided by MMM Group.

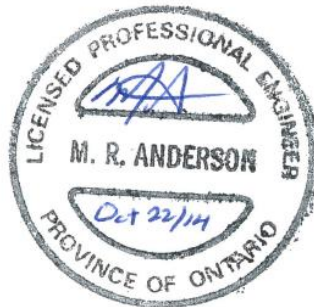
Thurber obtained utility clearances for the borehole locations prior to drilling.

Eastern Ontario Diamond Drilling Ltd from Hawkesbury, Ontario supplied a truck mounted CME75 drill rig and conducted the drilling, sampling and in-situ testing operations. The drilling operations were supervised by Mr. George Azzopardi.

Overall supervision of the field program was conducted by Mr. Mark Farrant, P.Eng. Interpretation of the data and preparation of the report were carried out by Ms. Mei Cheong, P.Eng.

The report was reviewed by Mr. Murray Anderson, P.Eng. and 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 RECOMMENDATIONS

7 GENERAL

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical recommendations for the proposed rehabilitation of the existing CPR Overhead structure (Mileage 115.85) on Highway 17 in Schreiber, Ontario.

The existing CPR Overhead grade separation was constructed in the 1940s and consists of a three span structure with a total length of 51.6 m and a width of 11.6 m. The structure is placed on a skew of approximately 35°. At the crossing, the railway runs in a cut with a depth of approximately 8.0 m below highway grade. Road grade at the abutments is approximate Elev. 304.4 and rail grade is near Elev. 295.7.

Archive drawings indicate that the structure is supported on spread footings founded on native sands and gravels. The abutment footings are approximately 13.5 m long, 1.8 m wide and 1.2 m thick, with a design top of footing at Elev. 301.2. The abutment footings are perched above a 1.5H:1V cut slope. The piers consist of two columns each supported on a pad footing approximately 2.7 m by 2.6 m in plan and 1.2 m thick, with a design top of footing at Elev. 295.3.

Based on the preliminary General Arrangement drawing provided by MMM Group (February 2014), replacement of the existing deck slab and steel girders with a new deck comprising precast prestressed concrete box girders is planned. The rehabilitation will include modification of the tops of the abutments, wingwalls and piers, and placement of a precast sleeper slab at the east approach. Highway grade revisions or widening are not planned.

The discussion and recommendations presented in this report are based on the information provided by MMM Group and on the factual data obtained in the course of the investigation.

8 ASSESSMENT OF EXISTING FOUNDATIONS

8.1 Abutment Foundations

The archive drawings indicate that the existing abutment footings are 1.8 m wide and founded at Elev. 300.0, with a vertical embedment depth of 2.0 m below a 1.5H:1V cut slope face. The borehole data indicates that the footings are founded on native compact to very dense sands and gravels.

Based on the existing footing width, the depth of embedment below the slope face, and the borehole information, the following geotechnical resistances are recommended to assess the capability of the existing abutment footings to carry the revised loads from the rehabilitated structure:

Factored Geotechnical Resistance at Ultimate Limit States (ULS) = 400 kPa

Geotechnical Reaction at Serviceability Limit States (SLS) = 275 kPa

The resistance values provided are for a vertical, concentric load. Where eccentric or inclined loads are applied, the resistance used in design must be reduced in accordance with the CHBDC Clause 6.7.3 and Clause 6.7.4.

The geotechnical reaction at SLS given above is based on an estimated total settlement not exceeding 25 mm. This settlement is expected to occur essentially as the foundation loads are applied.

We understand that the foundation loads applied on the abutment footings by the existing structure are 435 kPa at factored ULS and 270 kPa at SLS. Deck replacement would increase the loads to 490 kPa at factored ULS (a 13% increase) and 310 kPa at SLS (a 15% increase), exceeding the recommended resistance and reaction values.

To decrease the foundation loading at ULS to a level below the resistance value recommended above (400 kPa), placement of 1.2 m of EPS behind the abutment is planned, resulting in a foundation load of 325 kPa at factored ULS. The reduced loading at SLS with the EPS backfill will be 265 kPa, essentially unchanged from the current condition, and therefore the performance of the existing foundations will not be impacted.

Flattening of the slope in front of the abutments by placement of fill to improve the geotechnical resistance was also considered. However, the space between the abutments and piers is reportedly insufficient to accommodate a flatter inclination.

The lateral resistance of the footings may be computed using an unfactored friction coefficient of 0.55 for concrete on the native sand and gravel. This is an “ultimate” value and requires a degree of sliding movement to occur to fully mobilize the resistance.

8.2 Pier Foundations

The archive drawings indicate that the existing pier footings are 2.7 m by 2.6 m in plan and founded at Elev. 294.1. The borehole data indicates that the footings are founded on native dense to very dense sands and gravels.

Based on the existing footing dimensions and the borehole information, the following geotechnical resistances are recommended to assess the capability of the existing pier footings to carry the revised loads from the rehabilitated structure:

Factored Geotechnical Resistance at Ultimate Limit States (ULS) = 600 kPa

Geotechnical Reaction at Serviceability Limit States (SLS) = 400 kPa

The resistance values provided are for a vertical, concentric load. Where eccentric or inclined loads are applied, the resistance used in design must be reduced in accordance with the CHBDC Clause 6.7.3 and Clause 6.7.4.

The geotechnical reaction at SLS given above is based on an estimated total settlement not exceeding 25 mm. This settlement is expected to be essentially immediate as the foundation loads are applied.

We understand that foundation loads under the existing structure are 355 kPa at factored ULS and 300 kPa at SLS. Deck replacement will increase the loads to 475 kPa at factored ULS (a 34% increase) and 420 kPa at SLS (a 40% increase). Considering the available resistance and reaction values, the increase in vertical load is not expected to impact the performance of the existing foundations.

The lateral resistance of the footings may be computed using an unfactored friction coefficient of 0.55 for concrete on the native sand and gravel. This is an “ultimate” value and requires a degree of sliding movement to occur to fully mobilize the resistance.

8.3 Frost Cover

The depth of frost penetration at this site is 2.2 m. As the foundations soils consist of non-frost susceptible sands and gravels, and the existing foundations appear to be performing well, supplementary measures to provide additional frost protection are not considered necessary.

9 ABUTMENT BACKFILL AND LATERAL EARTH PRESSURES

Any new backfill to the abutment walls should be in accordance with OPSS 902. All granular material should consist of Granular A or Granular B Type II or Type III meeting the specifications of OPSS.PROV 1010. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS 501.

Lateral earth pressures acting on the abutments walls may be assumed to be distributed triangularly and to be governed by the characteristics of the wall backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

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

Where:

p_h = horizontal pressure on the wall at depth h (kPa)

K = earth pressure coefficient (see Table 9.1)

γ = unit weight of retained soil (see Table 9.1)

h = depth below top of fill where pressure is computed (m)

q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill and the existing material adjacent to the wall. Typical values are given in Table 9.1.

Table 9.1 – Earth Pressure Coefficients (K)

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, Type III or Existing Granular Fill $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)
Active (Unrestrained Wall)	0.27	0.38*	0.31	0.46*
At Rest (Restrained Wall)	0.43	-	0.47	-
Passive	3.7	2.1	3.3	1.7

* For wing walls.

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

The factors in Table 9.1 are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.16 in the Commentary to the Canadian Highway Bridge Design Code.

In accordance with Clause 6.9.3 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 2.0 m for Granular B Type I or at a depth of 1.7 m for Granular A or Granular B Type II.

10 SEISMIC CONSIDERATIONS

The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.0
- Acceleration Related Seismic Zone 0
- Zonal Acceleration Ratio 0.0
- Peak Horizontal Acceleration 0.011g

The soil profile type at this site has been classified as Type I. Therefore, according to Clause 4.4.6.1, Table 4.4 of the CHBDC, a Site Coefficient “S” (ground motion amplification factor) of 1.0 should be used in seismic design.

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading. The coefficients of horizontal earth pressure for seismic loading presented in Table 10.1 may be used:

Table 10.1 – Earth Pressure Coefficients for Earthquake Loading

Condition	Earth Pressure Coefficient (K)	
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ$ $\gamma = 22.8 \text{ kN/m}^3$	OPSS Granular B Type I or Existing Granular Fill $\phi = 32^\circ$, $\gamma = 21.2 \text{ kN/m}^3$
Active (K_{AE})*	0.28	0.31
Passive (K_{PE})	3.7	3.2
At Rest (K_{OE})**	0.44	0.49

* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.

** After Woods

The potential for liquefaction of the foundation soils was assessed using the Seed and Idriss (1971) method for cohesionless soils. The foundation soils at the site are not prone to liquefaction.

11 CUT SLOPES

The existing cut slopes in front of the perched abutments are approximately 8.0 m high and inclined at about 1.5H:1V. The borehole data indicates that the slopes are formed within 1.5 to 3.0 m of compact sand and gravel fill overlying compact to very dense native sand to sand and gravel. The existing slopes appear to be performing well. No revisions of the highway grade or cut slope inclinations are planned.

The stability of the existing cut slope was assessed using geotechnical parameters evaluated from the borehole data and the commercially available slope stability program GEO-SLOPE, applying the Morgenstern-Price method. The geotechnical model and results of the analysis are presented in Figure E1 of Appendix E. The computed factor of safety against instability of the existing slope is 1.2, which is below the value of 1.3 normally accepted for this type of analysis.

The stability analysis was repeated assuming a minimum 1.2 m thick layer of EPS was placed behind the abutment walls to reduce the foundations loads as discussed in Section 8.1. The results are shown on Figure E2 in Appendix E. The computed factor of safety for this configuration is 1.4, exceeding the accepted value of 1.3 for long term conditions.

Flattening of the slope in front of the abutments by placement of fill to achieve an inclination of 2H:1V as per MTO requirements for earth slopes was also considered as a method to improve the slope stability. However, we understand that the space between the abutments and piers is insufficient to accommodate a flatter inclination and encroachment into the railway corridor must be avoided.

Based on the results of the stability analyses and the past performance of the slopes, the stability of the existing cut slopes is considered to be adequate provided the proposed rehabilitation works include EPS in the abutment backfill. The existing slopes should be examined and re-established where past material loss due to surface erosion or downward creep is evident.

12 EROSION PROTECTION

The archive drawings indicate that the slopes in front of the existing abutments were inclined at 1.5H:1V and covered with rip rap. It is recommended that rock protection be reinstated on the slopes where required.

A vegetation cover should be established on all other exposed earth surfaces to protect against surficial erosion, in general accordance with OPSS 804.

13 EXCAVATION AND GROUNDWATER CONTROL

Excavation into the granular backfill and potentially the native sand and gravel will be required to carry out modifications to the existing abutments and wingwalls, and to place EPS in the approaches. The existing fill contains cobbles.

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the granular fill and native sand and gravel are classified as Type 3 soils.

The 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. Excavations should be inspected regularly for evidence of instability if they have been left open for extended periods of time and

following periods of heavy rain or thawing. If required, remedial actions must be taken to ensure the stability of the excavation and the safety of workers.

Works at the abutments are not expected to require excavation below the groundwater level. The Contractor should provide pumps if any seepage water or surface water collects in the excavation.

The excavation and backfilling behind the abutment walls must be carried out in accordance with OPSS 902.

Excavation at the abutments must not undermine the existing abutment footings.

14 ROADWAY PROTECTION

The bridge construction will be carried out in stages in order to keep at least one highway lane operational. Roadway protection should be provided in accordance with OPSS 539 and designed for Performance Level 2.

Conventional steel soldier pile and timber lagging walls or continuous sheet pile wall are two options to provide temporary support to the roadway during excavation. However, the existing embankment fill contains occasional cobbles which may interfere with installation of soldier piles or sheet piles. The ultimate choice of a roadway protection system should be left to the Contractor.

The 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.

The design of roadway protection should be the responsibility of the Contractor. All shoring systems should be designed by a Professional Engineer experienced in such designs.

15 CONSTRUCTION CONCERNS

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

- The existing embankment fill contains cobbles which may interfere with installation of soldier piles or sheet piles for roadway protection.

16 CLOSURE

Engineering analysis and preparation of the report were carried out by Ms. Mei Cheong, P.Eng, and Mr. Murray Anderson, 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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Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

NOTE: Hierarchy of Soil Strength Prediction

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



4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

5. LEGEND FOR RECORDS OF BOREHOLES

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

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

 Water Level
 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			

RECORD OF BOREHOLE No SCH-01

1 OF 2

METRIC

W.P. 6102-10-01 LOCATION CPR Overhead at Schreiber N 5 406 581.9 E 288 228.7 ORIGINATED BY GA
 HWY 17 BOREHOLE TYPE Hollow Stem Augers/NW Casing COMPILED BY AN
 DATUM Geodetic DATE 2013.06.12 - 2013.06.12 CHECKED BY MEF

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						
								20 40 60 80 100						
304.2														
0.0	ASPHALT: (125mm)													
0.1	SAND and GRAVEL Dense Brown Dry (FILL)		1	SS	31		304							
303.4														
0.8	SAND, trace gravel Compact Brown Dry (FILL)		2	SS	23		303							
302.7														
1.5	SAND and GRAVEL Dense Brown Dry		3	SS	35		302							46 49 5 (SI+CL)
	Occasional cobbles		4	SS	50/ 0.150									
301.2														
3.0	SAND, trace to some gravel, occasional cobbles Very Dense Brown Wet		5	SS	86		301							
	Gravel layer at 4.6m		6	SS	65		300							49 47 4 (SI+CL)
							299							
298.1														
6.1	SAND and GRAVEL Compact Brown Wet		7	SS	18		298							
							297							
			8	SS	24		296							
	Grey/Brown		9	SS	18		295							48 44 8 (SI+CL)

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SCH-01

2 OF 2

METRIC

W.P. 6102-10-01 LOCATION CPR Overhead at Schreiber N 5 406 581.9 E 288 228.7 ORIGINATED BY GA
 HWY 17 BOREHOLE TYPE Hollow Stem Augers/NW Casing COMPILED BY AN
 DATUM Geodetic DATE 2013.06.12 - 2013.06.12 CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	Continued From Previous Page																
294.1 10.1	END OF BOREHOLE AT 10.1m. BOREHOLE OPEN TO 10.1m AND WATER LEVEL AT 9.4m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.6m, CONCRETE TO 0.1m THEN ASPHALT PATCH TO SURFACE.																

RECORD OF BOREHOLE No SCH-02

1 OF 3

METRIC

W.P. 6102-10-01 LOCATION CPR Overhead at Schreiber N 5 406 569.2 E 288 236.9 ORIGINATED BY GA
 HWY 17 BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2013.06.15 - 2013.06.16 CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
304.3								20 40 60 80 100						
0.0	ASPHALT: (125mm)							20 40 60 80 100						
0.1	SAND Compact Brown Wet (FILL)		1	SS	26		304							
	Occasional gravel		2	SS	18		303							
302.8														
1.5	SAND, trace gravel Compact Brown Wet		3	SS	11		302							
			4	SS	16		301							
			5	SS	25		300							
	Gravel layer at 3.9m		6	SS	18		299							
298.2														
6.1	SAND and GRAVEL, trace silt Dense to Compact Brown Wet		7	SS	38		298							
			8	SS	25		297							
			9	SS	29		296							
							295							

Continued Next Page

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

METRIC

[illegible]

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No SCH-02

3 OF 3

METRIC

W.P. 6102-10-01 LOCATION CPR Overhead at Schreiber N 5 406 569.2 E 288 236.9 ORIGINATED BY GA
 HWY 17 BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2013.06.15 - 2013.06.16 CHECKED BY MEF

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									
284.2	Continued From Previous Page																
20.1	END OF BOREHOLE AT 20.1m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Jun. 17/13 15.2 289.1																

RECORD OF BOREHOLE No SCH-03

1 OF 4

METRIC

W.P. 6102-10-01 LOCATION CPR Overhead at Schreiber N 5 406 517.9 E 288 255.6 ORIGINATED BY GA
 HWY 17 BOREHOLE TYPE NW Casing/Coring COMPILED BY AN
 DATUM Geodetic DATE 2013.06.12 - 2013.06.13 CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
304.3								20 40 60 80 100						
0.0	ASPHALT: (100mm)							20 40 60 80 100						
0.1	SAND and GRAVEL, with cobbles (coring required) Compact Brown Wet (FILL)		1	SS	15		304							61 38 1 (SI+CL)
			2	SS	19		303							
302.0														
2.3	SAND, trace gravel Compact Brown Wet (FILL)		3	SS	13		302							
301.3														
3.0	SAND, trace gravel Compact Brown Wet		4	SS	18		301							0 91 9 (SI+CL)
			5	SS	16		300							
							299							
298.2														
6.1	SAND and GRAVEL, trace silt Dense Brown Wet		6	SS	44		298							
							297							
	Grey		7	SS	36		296							
	Occasional cobbles cored from 9.1m to 10.6m		8	SS	49		295							

Continued Next Page

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

RECORD OF BOREHOLE No SCH-03

2 OF 4

METRIC

W.P. 6102-10-01 LOCATION CPR Overhead at Schreiber N 5 406 517.9 E 288 255.6 ORIGINATED BY GA
HWY 17 BOREHOLE TYPE NW Casing/Coring COMPILED BY AN
DATUM Geodetic DATE 2013.06.12 - 2013.06.13 CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W _P	W	W _L			WATER CONTENT (%)	GR	SA	SI	CL
Continued From Previous Page																						
293.6							294															
10.7	SAND, trace gravel Dense Grey Wet		9	SS	39		293															
292.1																						
12.2	SAND and GRAVEL, trace to some silt Very Dense Grey Wet		10	SS	40		292															

Continued Next Page

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

RECORD OF BOREHOLE No SCH-03

3 OF 4

METRIC

W.P. 6102-10-01 LOCATION CPR Overhead at Schreiber N 5 406 517.9 E 288 255.6 ORIGINATED BY GA
HWY 17 BOREHOLE TYPE NW Casing/Coring COMPILED BY AN
DATUM Geodetic DATE 2013.06.12 - 2013.06.13 CHECKED BY MEF

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
	Continued From Previous Page		15	SS	34		284							
	SAND, trace gravel, trace silt to silty Dense Grey Wet						283							
							282							
			16	SS	49		281							0 88 12 (SI+CL)
							280							
							279							
			17	SS	16		278							
							277							
							276							
	Silt and sand layer		18	SS	15		275							0 54 44 2

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SCH-03

4 OF 4

METRIC

W.P. 6102-10-01 LOCATION CPR Overhead at Schreiber N 5 406 517.9 E 288 255.6 ORIGINATED BY GA
 HWY 17 BOREHOLE TYPE NW Casing/Coring COMPILED BY AN
 DATUM Geodetic DATE 2013.06.12 - 2013.06.13 CHECKED BY MEF

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 (%)					
	Continued From Previous Page													
273.8 30.5	End of sampling at 30.4m and start DCPT						274 273							
272.6 31.7	END OF BOREHOLE AT 31.7m. BOREHOLE OPEN TO 31.7m AND WATER LEVEL AT 24.6m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Jun. 14/13 24.6 279.7													



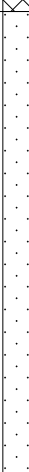
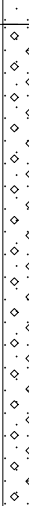
ONTMT4S 1197.GPJ 2012TEMPLATE(MTO).GDT 3/5/14

RECORD OF BOREHOLE No SCH-04

1 OF 2

METRIC

W.P. 6102-10-01 LOCATION CPR Overhead at Schreiber N 5 406 505.6 E 288 264.9 ORIGINATED BY GA
 HWY 17 BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2013.06.14 - 2013.06.15 CHECKED BY MEF

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							WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE							w _p w w _L		
304.1							20	40	60	80	100	20	40	60			
0.0	ASPHALT: (125mm)																
0.1	SAND and GRAVEL Dense to Very Dense Brown/Grey Wet (FILL)		1	SS	49								○				
			2	SS	31									○			
			3	SS	57										○		
301.8																63 35 2 (SI+CL)	
2.3	GRAVEL, trace sand Very Dense Grey Wet (FILL)		4	SS	62								○				
301.2																	
2.9	SAND, trace gravel, trace silt Compact Grey Wet		5	SS	25								○				
			6	SS	15										○		
298.0																	
6.1	SAND and GRAVEL, trace silt Dense Grey Wet		7	SS	37								○				
			8	SS	50										○		
			9	SS	33								○				

Continued Next Page

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

RECORD OF BOREHOLE No SCH-04

2 OF 2

METRIC

W.P. 6102-10-01 LOCATION CPR Overhead at Schreiber N 5 406 505.6 E 288 264.9 ORIGINATED BY GA
 HWY 17 BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2013.06.14 - 2013.06.15 CHECKED BY MEF

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 (%)					
294.0 10.1	Continued From Previous Page END OF BOREHOLE AT 10.1m. BOREHOLE OPEN TO 10.1m AND WATER LEVEL AT 9.4m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.9m, CONCRETE TO 0.2m THEN ASPHALT PATCH TO SURFACE.													

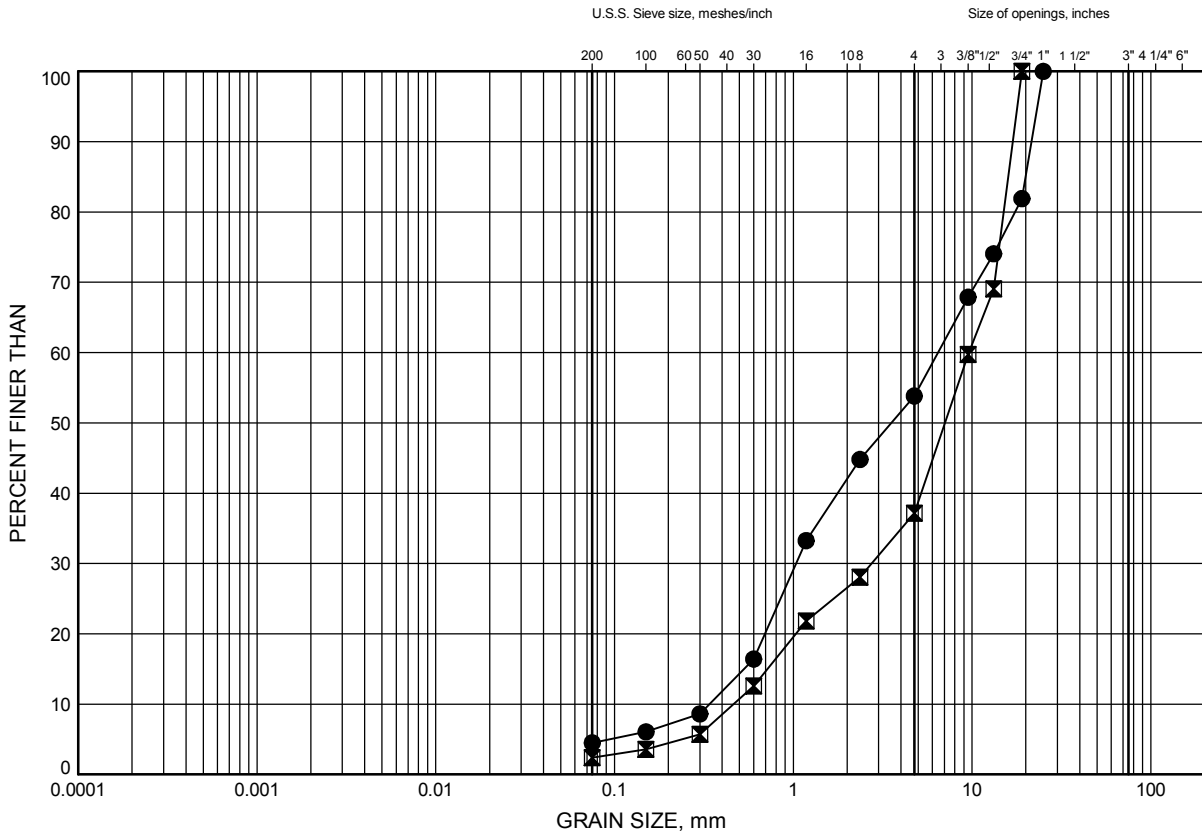
Appendix B

Laboratory Test Results

CPR Overhead at Schreiber
GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND & GRAVEL FILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SCH-01	1.83	302.37
⊠	SCH-04	1.83	302.27

Date March 2014
W.P. 6102-10-01

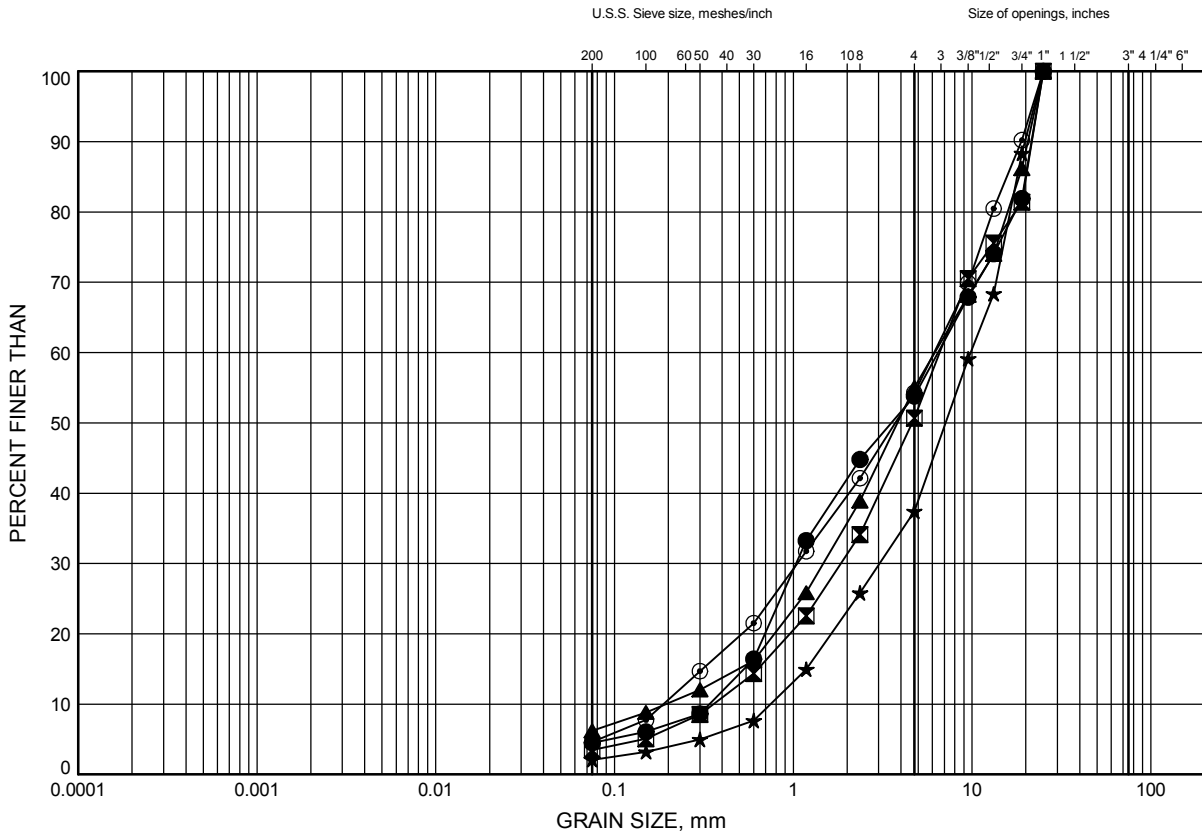


Prep'd AN
Chkd. MC

CPR Overhead at Schreiber
GRAIN SIZE DISTRIBUTION

FIGURE B2

SAND & GRAVEL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SCH-01	1.83	302.37
⊠	SCH-01	4.88	299.32
▲	SCH-02	6.40	297.90
★	SCH-02	10.97	293.33
⊙	SCH-03	12.50	291.80

Date February 2014
Project 6102-10-01

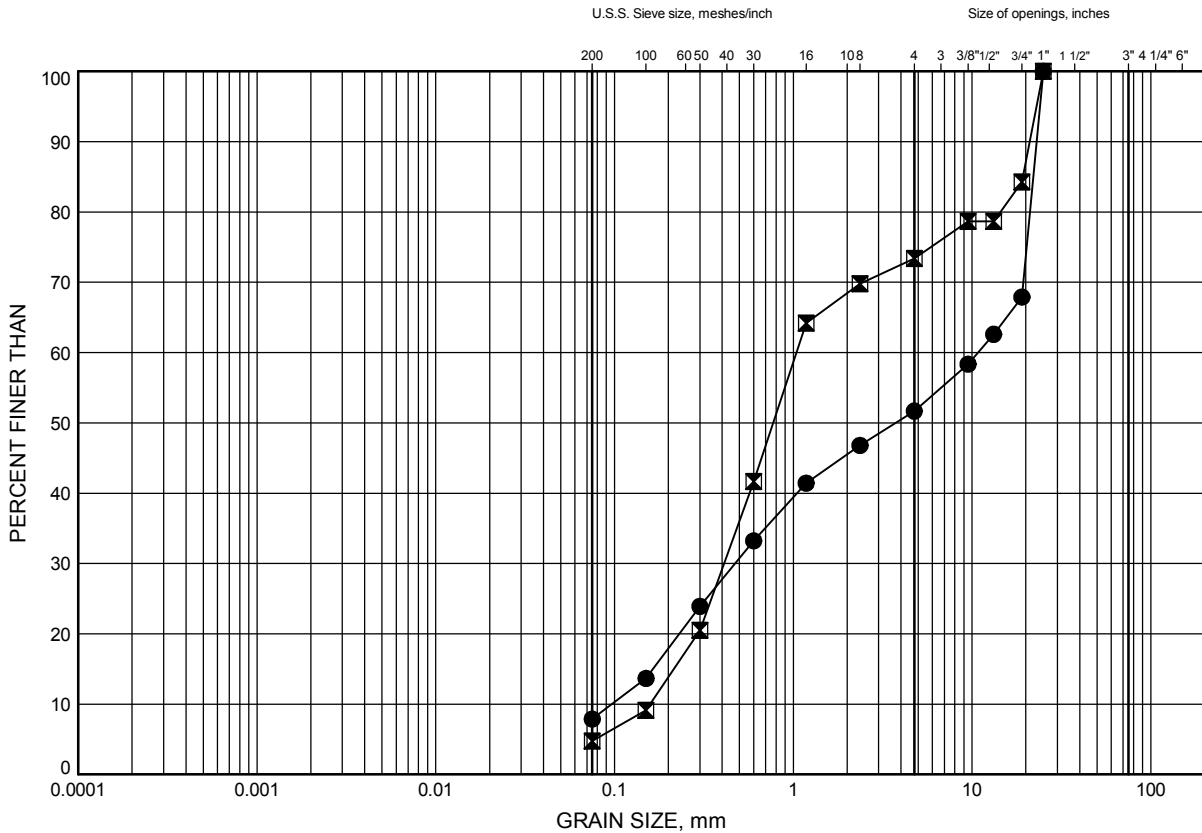


Prep'd AN
Chkd. MC

CPR Overhead at Schreiber
GRAIN SIZE DISTRIBUTION

FIGURE B3

SAND AND GRAVEL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SCH-01	9.45	294.75
⊠	SCH-04	6.40	297.70

Date February 2014
 Project 6102-10-01

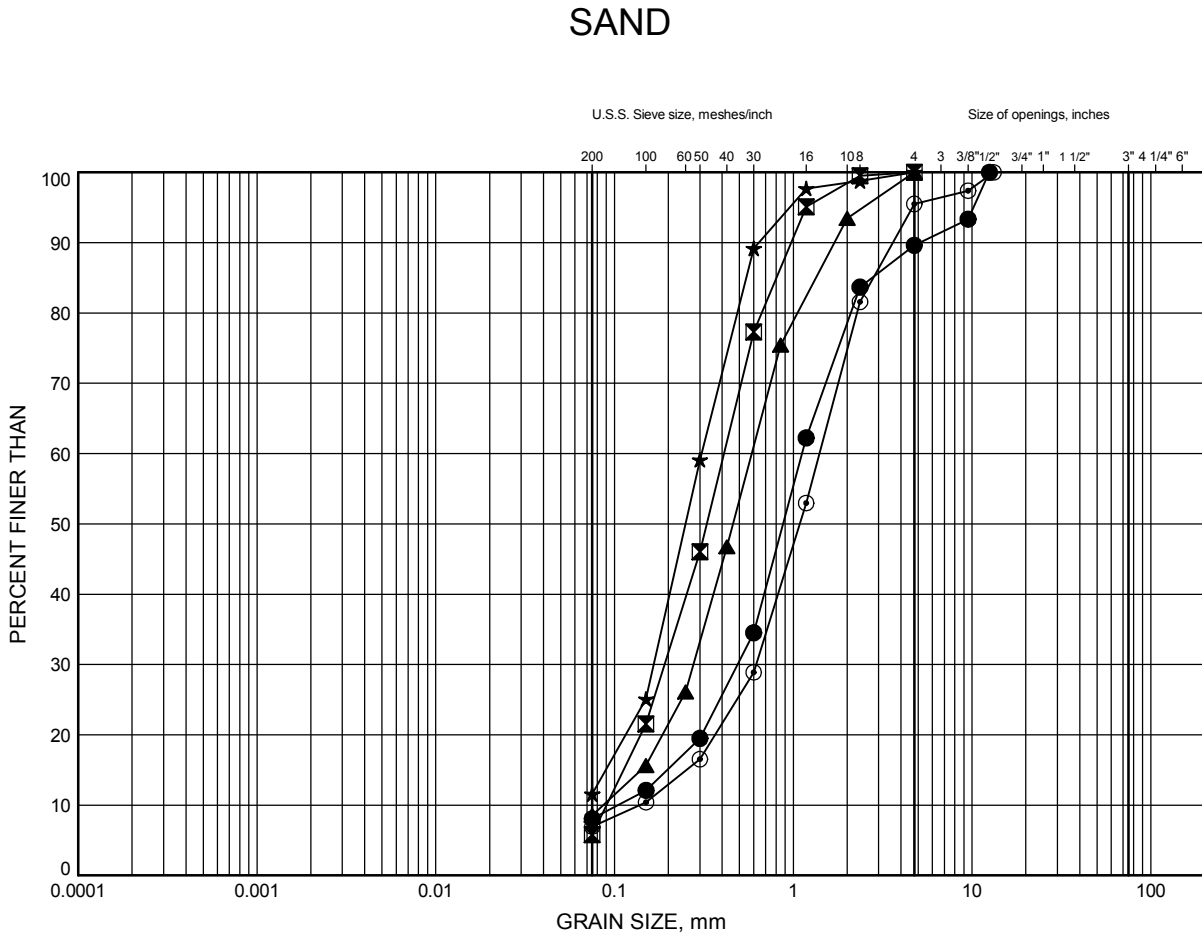


Prep'd AN
 Chkd. MC

CPR Overhead at Schreiber

GRAIN SIZE DISTRIBUTION

FIGURE B4



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SCH-02	2.59	301.71
⊠	SCH-02	17.07	287.23
▲	SCH-03	3.35	300.95
★	SCH-03	23.16	281.14
⊙	SCH-04	3.35	300.75

Date March 2014
W.P. 6102-10-01



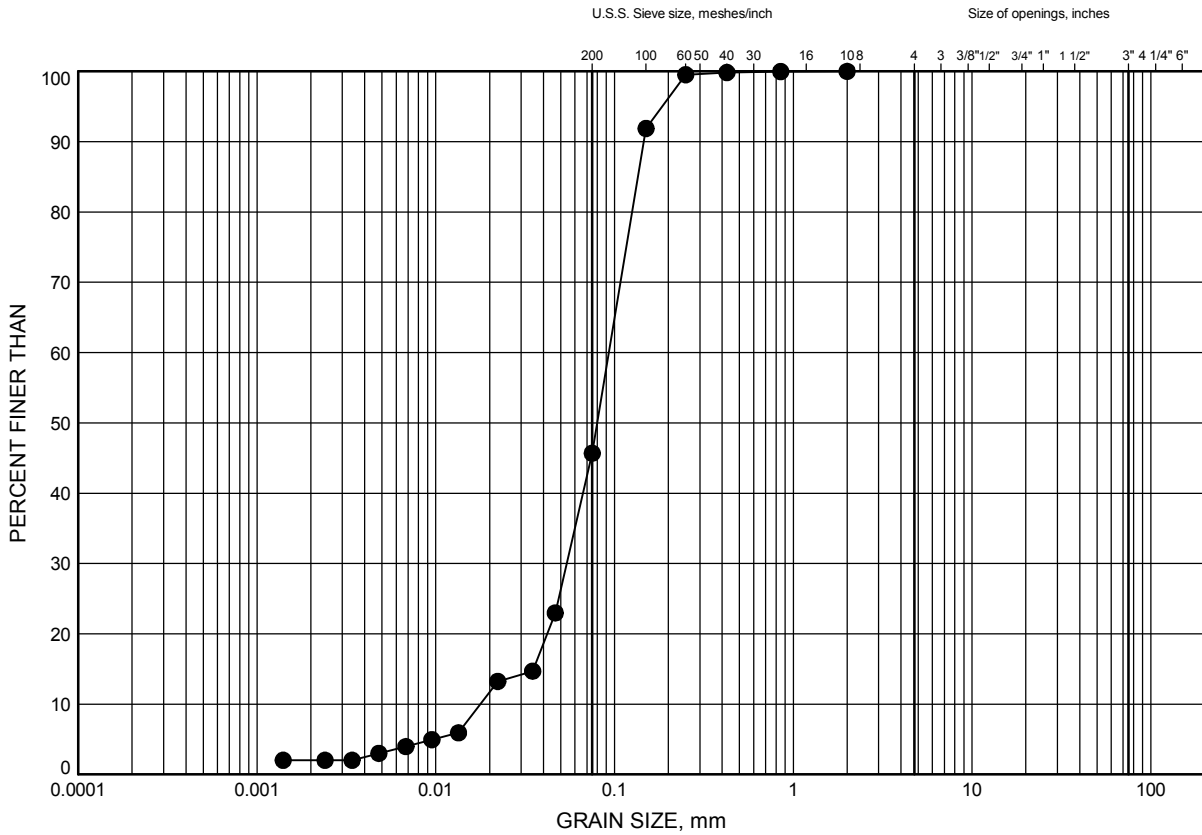
Prep'd AN
Chkd. MC

CPR Overhead at Schreiber

GRAIN SIZE DISTRIBUTION

FIGURE B5

SILT & SAND



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SCH-03	29.26	275.04

Date February 2014

Project 6102-10-01



Prep'd AN

Chkd. MC

Appendix C

Site Photographs



Photograph 1 – North elevation of CPR Overhead, looking south



Photograph 2 – South elevation of the CPR Overhead, looking north



Photograph 3 - West abutment



Photograph 4 – West pier, looking east

Appendix D

List of SPs and OPSS

List of Special Provisions and OPSS Documents Referenced in this Report

- OPSS 501
- OPSS 539
- OPSS 804
- OPSS 902
- OPSS.PROV.1010

Appendix E

Slope Stability Analyses

File Name: Abutment_v4.gsz
 Directory: H:\19\1351\197 NWR 32 Rehabs\Reports & Memos\CP Overhead Schreiber\03 Analysis\
 Name: Existing 1-1.5
 Description: Existing bridge with SLS load
 Created By: Mei Cheong
 Method: Morgenstern-Price
 Minimum Slip Surface Depth: 2 m

Name: Existing Fill Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 35 ° Piezometric Line: 1
 Name: Sand Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 34 ° Piezometric Line: 1
 Name: Sand and Gravel Unit Weight: 21 kN/m³ Cohesion: 0 kPa Phi: 36 ° Piezometric Line: 1
 Name: Sand 2 Unit Weight: 21.5 kN/m³ Cohesion: 0 kPa Phi: 35 ° Piezometric Line: 1

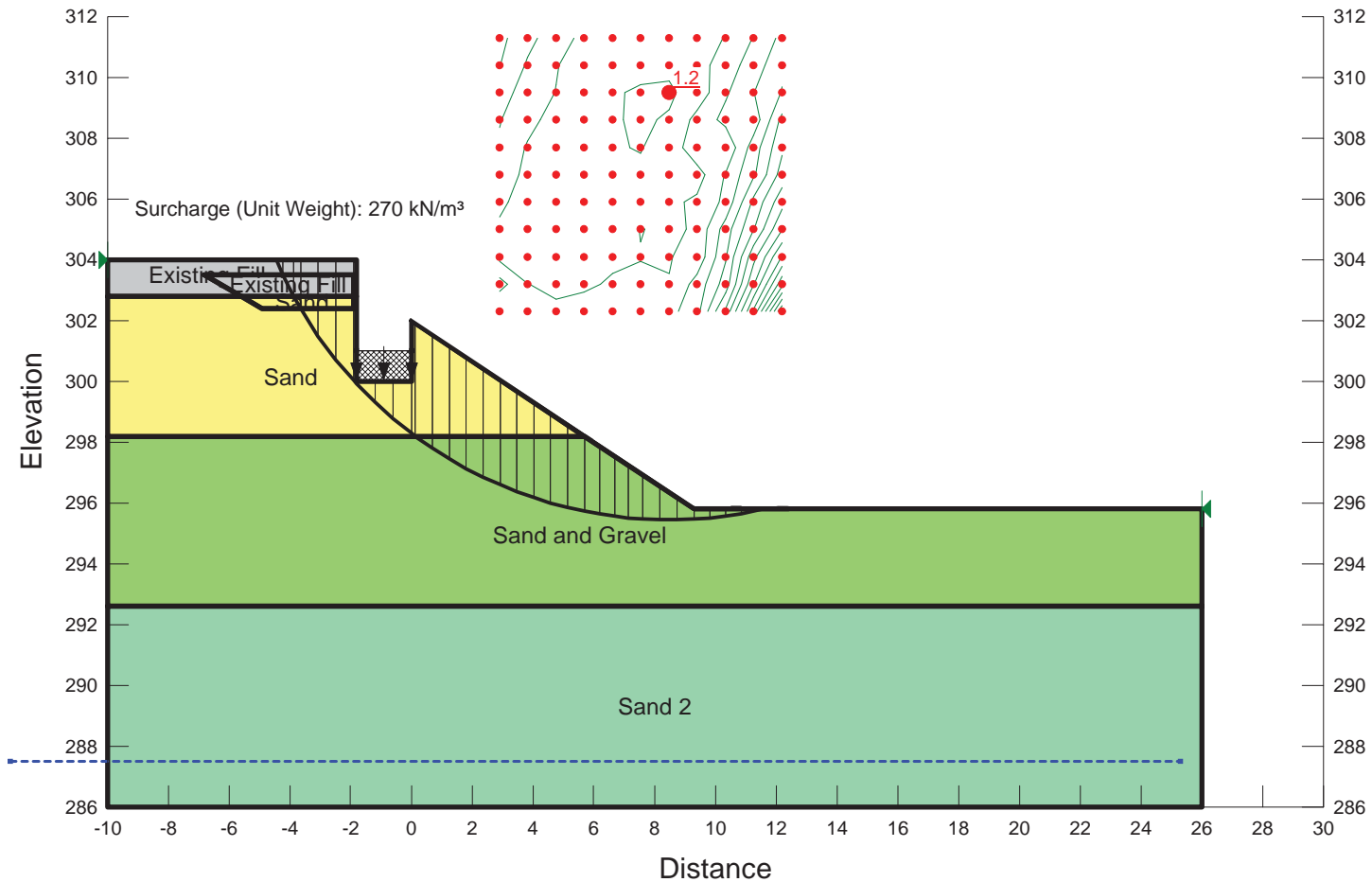


FIGURE E1

File Name: Abutment_v4.gsz

Directory: H:\19\1351\197 NWR 32 Rehabs\Reports & Memos\CP Overhead Schreiber\03 Analysis\

Name: New deck 3-1.5 (3)

Description: New bridge SLS load with EPS

Created By: Mei Cheong

Method: Morgenstern-Price

Minimum Slip Surface Depth: 2 m

Name: Existing Fill	Unit Weight: 21 kN/m ³	Cohesion: 0 kPa	Phi: 35 °	Piezometric Line: 1
Name: Sand	Unit Weight: 21 kN/m ³	Cohesion: 0 kPa	Phi: 34 °	Piezometric Line: 1
Name: Sand and Gravel	Unit Weight: 21 kN/m ³	Cohesion: 0 kPa	Phi: 36 °	Piezometric Line: 1
Name: Sand 2	Unit Weight: 21.5 kN/m ³	Cohesion: 0 kPa	Phi: 35 °	Piezometric Line: 1
Name: EPS (2)	Unit Weight: 1 kN/m ³	Cohesion: 50 kPa	Phi: 0 °	Piezometric Line: 1

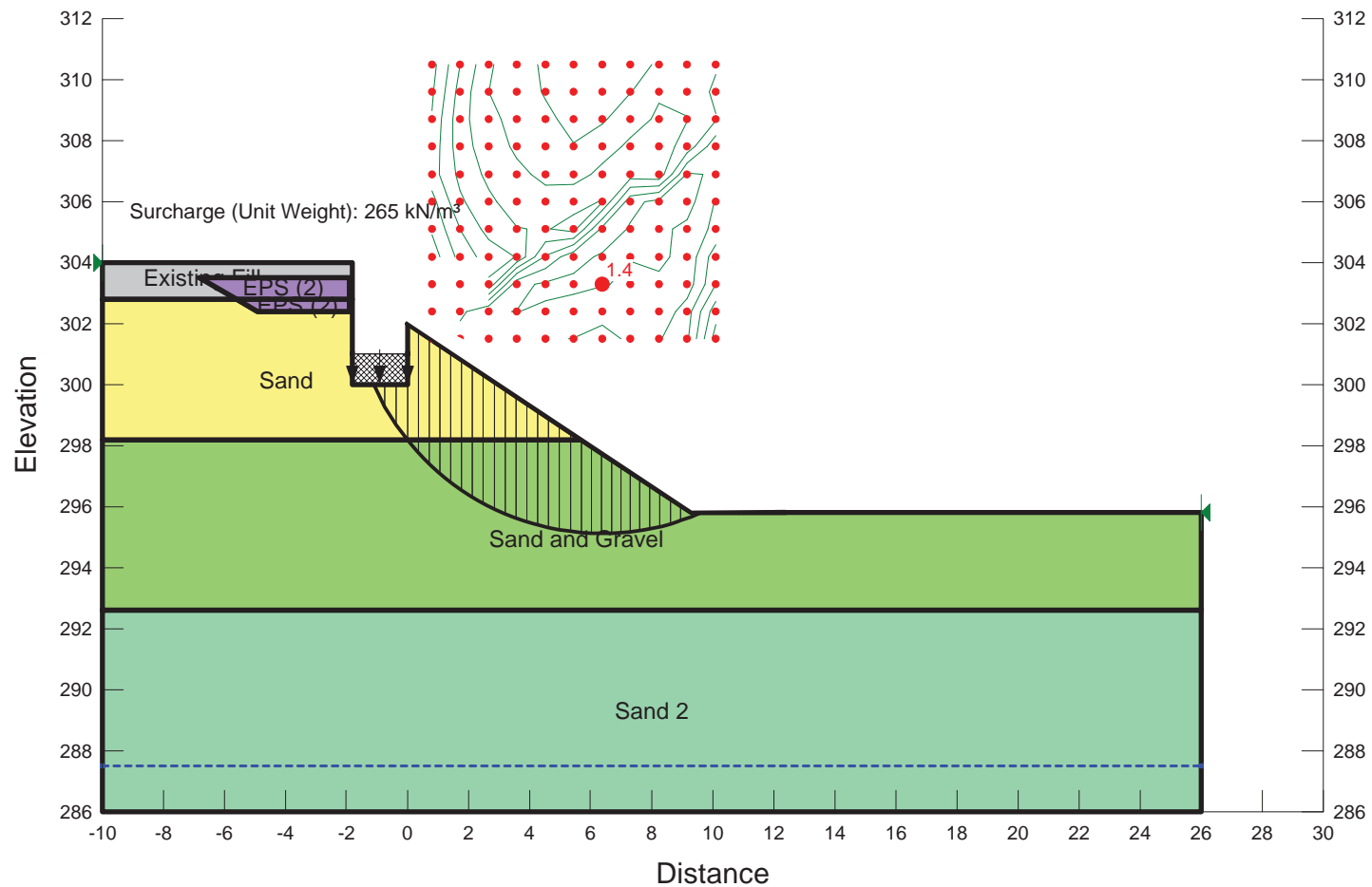
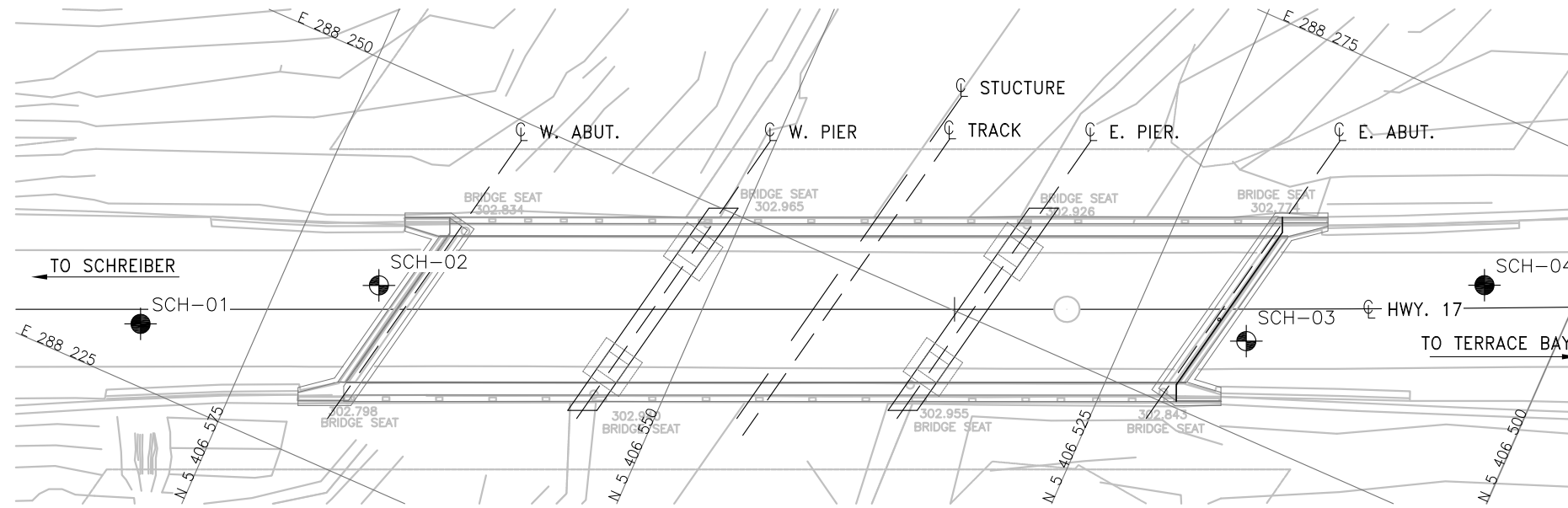


FIGURE E2

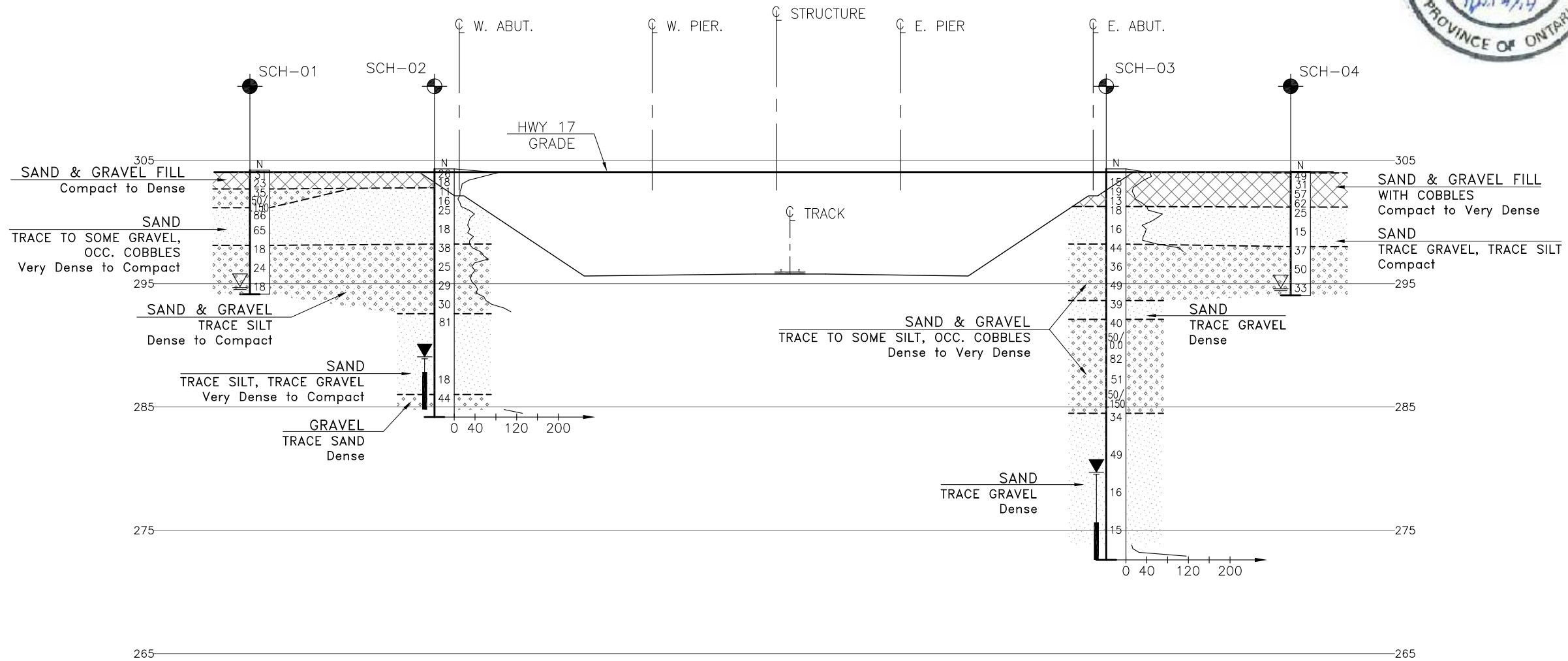
Appendix F

Borehole Locations and Soil Strata Drawing



8 0 8 16m

SCALE 1:400



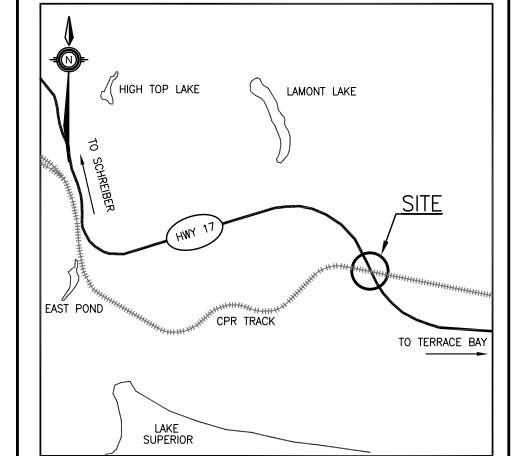
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN



CONT No 2014-6007 ()
WP No 6102-10-01

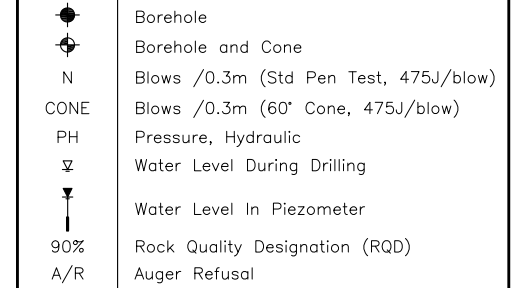
HIGHWAY 17
C.P.R. OVERHEAD
AT SCHREIBER
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET
13



KEYPLAN

LEGEND

[illegible]

-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.

GEOCRES No. 42D-33

REVISIONS									
	DATE	BY	DESCRIPTION						
DESIGN	MC		CHK	MC	CODE		LOAD	DATE	APR 2014
DRAWN	AN		CHK		SITE	48E-16	STRUCT	DWG	2