FOUNDATION INVESTIGATION AND DESIGN REPORT
HASTIE RIVER BRIDGE REPLACEMENT
HIGHWAY 613
TOWNSHIP OF BURRISS, DISTRICT OF THUNDER BAY, ONTARIO

G.W.P. 494-00-00, SITE NO: 45-68

Geocres Number: 52C-34

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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 Hastie River Bridge along Highway 613, in the District of Thunder Bay, 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, records of boreholes, 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 Hatch Mott MacDonald, under the Ministry of Transportation Ontario (MTO) Agreement Number 6010-E-0010.

2 SITE DESCRIPTION

The Hastie River Bridge is located on Highway 613, approximately 20 km west of Fort Frances and 4 km north of Highway 11/71, between Hastie/Pyne Road and Booth/Maki Road. The existing bridge comprises a five-span structure supported on timber piles. The bridge is approximately 28 m long and 9 m wide. The existing north and south approach embankments are approximately 2.0 to 2.5 m high above the surrounding ground.

The Hastie River Bridge carries Highway 613 across the Hastie River, which meanders southerly into Rainy River. The banks of the river are heavily vegetated and the surrounding lands are relatively flat comprising farmlands with occasional residential dwellings.

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 Wabigoon Subprovince of the Superior Province of the Canadian Shield. The site is underlain by Neo to Mesoarchean intrusive rocks overlain by glaciolacustrine deposits comprising silt and clay with minor sand. The river channels are underlain by modern alluvial deposits consisting fine sand, silt and clay with detrital organic remains.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out between November 1 and 4, 2013. A total of five boreholes were drilled and sampled. The five boreholes were identified as Boreholes HRB-01, HRB-02, HRB-03, HRB-05 and HRB-06. Boreholes HRB-03 and HRB-04 were planned as the second borehole at each abutment. However, since Boreholes HRB-02 and HRB-05 were drilled beyond 30 m depth, Borehole HRB-03 was terminated within the fill and Borehole HRB-04 was not drilled. The approximate borehole locations are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix G.

The borehole locations were marked in the field and utility clearances were obtained prior to drilling. The coordinates and ground surface elevations of the boreholes were derived from topographic plan provided by HMM.

A truck-mounted CME75 drill rig was used to advance the boreholes using a combination of NW casing/ wash boring technique and NQ coring. Soil samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT).

Rock cores were recovered from Boreholes HRB-02 and HRB-05 using NQ coring techniques. All rock cores were logged and the Total Core Recovery (TCR), Rock Quality Designation (RQD) and Fractures Indices (FI) were determined.

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 and rock core samples for transport to Thurber's laboratory for further examination and testing.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Groundwater conditions observed after completion of drilling were not representative of site conditions as water was used to assist the drilling and rock coring. Two standpipe piezometers were installed to monitor the groundwater level at the site. Completion details of the piezometer and boreholes are summarized in Table 3.1.

Table 3.1 – Borehole Completion Details

Location	Borehole	Piezometer Tip Depth/Elevation (m)	Completion Details
South Approach	HRB-01	None installed	Borehole backfilled with bentonite holeplug to 0.1 m, cement to 0.07 m and asphalt to surface.
South Abutment	HRB-02	30.5 / 320.8	Borehole backfilled with bentonite holeplug to 30.5 m, sand from 30.5 m to 28.7 m, bentonite holeplug from 28.7 m to 0.6 m, sand from 0.6 m to 0.2 m, cement from 0.2 m to 0.1 m, then asphalt and flush mount cover to surface.
	HRB-03	None installed	Borehole backfilled with bentonite holeplug to 0.07m, then asphalt to surface.
North Abutment	HRB-05	32.9 / 318.4	Borehole backfilled with bentonite holeplug to 32.9 m, sand from 32.9 m to 31.1 m, bentonite holeplug from 31.1 m to 0.3 m, sand from 0.3 m to 0.1 m, then asphalt and flush mount cover to surface.

Location	Borehole	Piezometer Tip Depth/Elevation (m)	Completion Details
North Approach	HRB-06	None installed	Borehole backfilled with bentonite holeplug to 0.1 m then asphalt to surface.

Following the final water level readings, the standpipe piezometers were decommissioned in general accordance with MOE Regulation 903.

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.

Bedrock core samples were subjected to geological logging. Point load tests were carried out on selected samples of intact core in the laboratory to evaluate the unconfined compressive strength (UCS) of the bedrock. The UCS values of the rock cores assessed from the point load tests are reported on the Record of Borehole sheets in Appendix A.

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 G. 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 subsurface conditions of the site typically consist of embankment fill materials underlain by a thick deposit of native silty clay which overlies bedrock. A sandy silt layer was encountered within the native silty clay. More detailed descriptions of the individual strata are presented below.

5.1 Asphalt

Asphalt was encountered in all boreholes which were drilled from the highway pavement surface. The asphalt thickness varied from 25 to 50 mm.

5.2 Embankment Fill

Embankment fill comprising gravelly sand to sand overlying sandy silt was encountered beneath the asphalt. Sandy silt fill containing some clay was encountered only in Boreholes HRB-02 and HRB-03. The embankment fill contains occasional cobbles. Borehole HRB-03 was terminated within the fill. Thickness of the sand fill ranged from 1.5 to 2.3 m and, where fully penetrated, thickness of the sandy silt fill was 0.8 m in Borehole HRB-02. The total thickness of the fill encountered was between 1.5 m and 2.3 m, with the lower boundary ranging from elevation 349.0 to 349.9.

SPT N-values recorded within the sand fill typically ranged from 12 to 22 blows for 0.3 m penetration, indicating a compact relative density. An SPT-N value of 50 blows for zero penetration was recorded in Borehole HRB-05, indicating the presence of cobbles. In the sandy silt fill, SPT-N values of 10 and 11 blows for 0.3 m penetration were recorded, indicating a compact relative density.

The measured moisture content ranged from 6 to 19% in the sand fill and from 23 to 28% in the sandy silt fill.

The results of grain size distribution analysis of the fill samples are summarized below. These results are also presented on the Record of Borehole sheets included in Appendix A and are shown on Figures B1a and B1b of Appendix B.

Soil Particles	Gravelly Sand to Sand Fill (%)	Sandy Silt Fill (%)
Gravel	0 to 34	0
Sand	65 to 97	25
Silt	1 to 3	59
Clay		16

5.3 Silty Clay

Native brown to grey silty clay with sand to trace sand was encountered beneath the embankment fill in all boreholes. The upper portion of silty clay deposit contains occasional rootlets and wood fibres. Boreholes HRB-01 and HRB-06 were terminated within the silty clay layer at depths of 6.1 m and 11.3 m, respectively. Borehole HRB-01 was terminated upon refusal on probable cobbles or boulders. The thicknesses of the layer fully penetrated in Boreholes HRB-02 and HRB-05 were 27.9 m and 30.6 m, with the lower boundary at elevation 321.1 and 318.4, respectively.

SPT N-values recorded in the silty clay ranged between 7 and 19 blows per 0.3 m penetration, indicating firm to very stiff consistency. An SPT-N value of 50 blows for zero penetration was recorded at the base of Borehole HRB-01, which indicates the presence of cobbles or boulders. The measured moisture content of the silty clay samples ranged from 19 to 60% and typically from 25 to 40%.

Selected silty clay samples underwent grain size distribution analysis and Atterberg Limits tests. The results of the grain size distribution analyses are shown on Figures B2, B3a and B3b of Appendix B. The results of the Atterberg Limits tests are presented on Figures B5 and B6 of Appendix B. The results are presented on the Record of Borehole sheets included in Appendix A, and summarized in the following tables:

Soil Particles	Silty Clay (%)	Silty Clay, With Sand (%)
Gravel	0	0
Sand	6 to 15	45 to 46
Silt	27 to 48	26 to 31
Clay	38 to 66	24 to 28

Liquid Limit (%)	51 to 71
Plastic Limit (%)	22 to 30

The results of the Atterberg Limits tests indicate that the silty clay is high plastic (CH).

5.4 Sandy Silt

A layer of grey sandy silt with some clay was encountered in Borehole HRB-02 within the silty clay layer. The thickness of the layer was 3.0 m with the lower boundary of the layer at elevation 326.0. The sandy silt is in a compact state based on an SPT N-value of 20 blows per 0.3 m penetration. The moisture content of the sandy silt is 24%.

The result of grain size distribution analysis of the sandy silt sample indicates that the soil contains 29% Sand, 56% Silt and 15% Clay. This result is also presented on the Record of Borehole sheets included in Appendix A and is shown on Figure B4 of Appendix B.

A lower layer of grey sandy silt with trace gravel was encountered immediately above the bedrock in the same Borehole HRB-02 at a depth of 30.2 m or elevation 321.1. The thickness of the layer was 1.8 m. This layer of sandy silt contains cobbles and boulders. Artesian condition was encountered in this lower layer.

5.5 Bedrock

Bedrock was encountered below the silty clay deposit in Boreholes HRB-02 and HRB-05 and proven by coring a minimum of 3.0 m into the bedrock. The depths and elevations of the bedrock surface are summarized in Table 5.1.

Table 5.1 – Depth and Elevation of Bedrock Surface

Borehole	Top of Bedrock	
	Depth (m)	Elevation
HRB-02	32.0	319.3
HRB-05	32.9	318.4

The bedrock was described as slightly weathered to fresh, coarse grained, mottled grey, red and black granite. Total Core recovery (TCR) of all core runs was 100%. The Rock Quality Designation (RQD) of the recovered rock cores ranged from 59% to 95%, indicating fair to excellent rock quality. The Fracture Index (FI) of the rock, expressed as the number of fractures per 0.3 m core, was typically between 0 and 5.

The average unconfined compressive strengths (UCS) of the intact rock cores interpreted from point load tests (PLT) ranged from 151 to 226 MPa, indicating a very strong to extremely strong rock strength.

5.6 Groundwater Conditions

Artesian conditions were observed in Borehole HRB-02 in the sandy silt layer just above the bedrock surface with artesian head up to 1.5 m in the casings extended above the ground surface. The artesian flow dissipated in 20 minutes and the subsequent water level was at the ground surface.

Wash boring method was used to advance the boreholes. Therefore water levels observed in the open boreholes were not reflective of the natural groundwater levels. Standpipe piezometers were installed in two boreholes to monitor the groundwater level after borehole completion. The water levels measured in the piezometers are summarized in Table 5.2.

Table 5.2 – Water Level Measurements

Borehole	Date	Water Level		Comment
		Depth (m)	Elev. (m)	
HRB-02	November 4, 2013	0.0	351.3	In piezometer
HRB-05	November 4, 2013	0.0	351.3	In piezometer

The preliminary GA drawing provided by HMM indicates that the river level was at elevation 348.4 on October 11, 2013, or approximately 3 m below the existing grade. The water levels measured in the piezometers were at the pavement level, which are indicative of the artesian conditions in the soil layers near the bedrock surface.

The above values are short-term readings and seasonal fluctuation of the groundwater level is 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 of the drilled boreholes were established based on topographic survey information provided by HMM.

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

Eastern Ontario Diamond Drilling of 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.

The report was prepared by Mr. Keli Shi, P.Eng., and reviewed by Mr. Murray R. Anderson, P.Eng. and Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations projects.

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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 provides geotechnical recommendations to assist the design team in selecting and designing a suitable foundation system for the proposed replacement bridge.

At present, Highway 613 crosses the Hastie River on a 28 m long five-span structure carrying a bridge deck approximately 9 m wide. The existing road grade on the bridge is at elevation 351.3 m. Based on the existing bridge drawings, the existing abutments and piers are each supported by a single row of five timber piles.

The General Arrangement drawing indicates that the replacement bridge will be constructed in two stages with half of the bridge operational during each stage. The replacement bridge will be a 21 m long single-span bridge carrying an approximately 11.8 m wide bridge deck. The existing road grade will be maintained for the replacement bridge and approach embankments.

Steel sheet pile walls will be installed at both abutments to retain the approach fill. New fill will be placed on the existing 2H: 1V river bank slopes to extend the existing approach embankments to the new abutments. At the embankment centreline, fill heights above the existing ground (river bank slopes) will be about 2.0 m at the South Abutment and 1.0 m at the North Abutment, respectively.

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

8 STRUCTURE FOUNDATIONS

In general, the soil stratigraphy below the existing embankment fill consists of predominantly a thick deposit of stiff to very stiff silty clay. A sandy silt layer containing some clay was encountered within the silty clay at the South Abutment. Slightly weathered to fresh granitic bedrock was found underlying the silty clay deposit. The water level in the river was at approximately elevation 348.4 m on October 11, 2013, or approximately 3 m below the existing road grade.

Foundation alternatives are presented in the following sections together with the corresponding geotechnical design parameters. A preferred foundation scheme from a geotechnical perspective is recommended.

A comparison of the technical advantages and disadvantages of alternative foundation schemes is presented in Appendix D. Initial consideration was given to spread footings on native soil or engineered fill, driven steel H-piles, and caissons (drilled shafts).

8.1 Spread Footings on Native Soil or Engineered Fill

The use of spread footings founded on native soil to support the abutments is not recommended given the relatively low geotechnical resistance available and potential large consolidation settlement of the foundation soils. Similarly, supporting the abutments on spread footings founded on engineered fill is not feasible.

8.2 Driven Steel H-Piles

The ground conditions at the site are considered to be suitable for the use of steel H-piles.

8.2.1 Axial Resistance

It is recommended that H-piles be driven to refusal on bedrock. Boulders were encountered in the sandy silt layer overlying the bedrock surface in Borehole HRB-02 at the south abutment. Some piles may meet refusal on boulders above the bedrock surface.

The anticipated pile tip elevations and factored geotechnical resistances at ULS for HP 310x110 piles driven to bedrock are presented in Table 8.1. The SLS reaction will not govern for piles driven to bedrock.

Table 8.1 – Anticipated Pile Tip Elevation and Recommended Geotechnical Resistance for H-Piles

Foundation Element	Borehole	Anticipated Tip Elevation (m)	Factored Geotechnical Resistance at ULS (kN)
South Abutment	HRB-02	319.3	2,000
North Abutment	HRB-05	318.4	2,000

8.2.2 Downdrag

Downdrag forces will develop along the length of abutment piles embedded in the embankment fill and silty clay layer due to consolidation of the silty clay under the weight of new approach fill placed behind the abutments.

For design purposes, an unfactored downdrag load of 400 kN per pile is recommended to evaluate the impact of downdrag on the abutment piles.

This downdrag load should be multiplied by a load factor of 1.25 as per CHBDC Commentary Clause C6.8.4 to obtain a factored downdrag load. In accordance with Section 6.8.4 of the CHBDC and Clause C6.8.4 of the Commentary, in the structural design of a pile, the factored downdrag load should be added to the factored permanent loads to assess the effects of downdrag. The factored dead and downdrag load should not exceed the factored structural resistance of a pile.

In geotechnical analysis of downdrag, live load effects should not be considered.

The location of the neutral plane for a pile or group of piles should be determined by using unfactored loads and unfactored geotechnical parameters.

8.2.3 Pile Tips

Pile tip protection is recommended for H-piles to prevent pile damage when setting the piles on bedrock or if cobbles and boulders are encountered. The tips of all driven H-piles must be fitted with pile tip protection from an approved manufacturer such as Titus Steel (Standard H-point) or approved equivalent.

8.2.4 Pile Installation

Pile installation should be in accordance with OPSS 903. The appropriate pile driving note is "Piles to be driven to bedrock".

To facilitate pile installation, embankment fill through which piles will be driven must not contain oversize material, i.e. no particles exceeding 75 mm in size.

8.2.5 Lateral Pile Resistance

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

$$k_s = n_h \cdot z / D \quad (kN/m^3)$$

$$p_{ult} = 3 \cdot \gamma' \cdot z \cdot K_p \quad (kPa)$$

Where
- z = depth of embedment of pile (metre)
- D = pile width or diameter (metre)
- n_h = coefficient related to soil density (kN/m^3)
- γ' = effective unit weight (kN/m^3)
- K_p = coefficient of passive 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:

$$k_s = 67 \cdot S_u / D \quad (kN/m^3)$$

$$p_{ult} = 9 \cdot S_u \quad (kPa)$$

Where
- S_u = undrained shear strength (kPa)
- D = pile width or diameter (metre)

The above equations and recommended parameters in Table 8.2 may be used to analyze the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis must not exceed the ultimate lateral resistance.

Table 8.2 – Soil Parameters for Lateral Pile Resistance

Soil Unit	Elevation (m)		γ' (kN/m^3)	n_h (kN/m^3)	K_p	S_u (kPa)
	Top	Bottom				
South Abutment (Borehole HRB-02)						
Fill	350.1	349.0	21	2,500	3.0	-
Silty Clay	349.0	329.1	9	-	-	60
Sandy Silt	329.1	326.0	10	2,000	3.5	-
Silty Clay	326.0	321.1	9	-	-	60
Sandy Silt	321.1	319.3	10	2,000	3.5	-
North Abutment (Borehole HRB-05)						
Fill	350.1	349.0	21	2,500	3.0	-
Silty Clay	349.0	318.4	9	-	-	60

The spring constant, K_s, for analysis may be obtained by the expression, $K_s = k_s \times L \times D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m^3), D is the pile width (m) and L is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance, P_{ult}, may be obtained from the expression, $P_{ult} = p_{ult} \times L \times D$. This represents the ultimate load at which the pile fails and will not support any additional load at greater displacements. It is recommended, however, that the total lateral resistance assumed in one pile be limited to no more than 120 kN at ULS and 35 kN at SLS.

The coefficient of subgrade reaction and ultimate lateral resistance may have to be reduced, based on the pile spacing. The reduction factors to be used for a pile group oriented perpendicular or parallel to the direction of loading are provided in Table 8.3. Intermediate values may be obtained by linear interpolation.

Table 8.3 – Subgrade Reaction Reduction Factors for Pile Spacing

Condition	Pile Spacing Centre to Centre	Reduction Factor
Pile group oriented ***perpendicular*** to direction of loading	4D	1.0
	1D	0.5
Pile group oriented ***parallel*** to direction of loading	8D	1.0
	6D	0.7
	4D	0.4
	3D	0.25

8.3 Caissons (Drilled Shafts)

Given the depths of bedrock and the high groundwater table at this site, the use of caissons or drilled shafts is not considered to be a cost-effective option and has not been developed herein.

8.4 Recommended Foundation

From a geotechnical perspective and based on the subsurface conditions, steel H-piles driven to bedrock are considered to be the most cost effective foundation option at this site.

8.5 Frost Cover

The depth of frost penetration at this site is approximately 2.4 m. The base of pile caps must be provided with a minimum of 2.4 m of earth cover as protection against frost action.

8.6 Impact on Existing Foundations

Piles will be driven adjacent to the existing bridge for construction of the replacement bridge.

The existing timber piles are likely frictional piles founded in the silty clay deposit. Potential for settlement exists for the existing bridge due to excess pore pressure generation and dissipation in the silty clay in response to the pile driving.

Therefore, it is recommended that the structural designer select appropriate settlement monitoring points on the existing structure and specify a monitoring program for the duration of pile driving. Based on results of the monitoring program, the Contractor should be prepared with appropriate equipment on site to maintain the grade of the existing bridge in operation, which includes but is not limited to lifting and shimming the bridge.

9 EXCAVATION AND DEWATERING

It is recommended that all excavation for construction of pile caps or footings at the abutments be maintained above the river level, approximately elevation 348.4 m on October 11, 2013. Excavation above this level will be carried out primarily within the existing granular fill and approximately 0.5 m in the stiff silty clay.

All excavations must be carried out in accordance with the requirements of the Occupational Health and Safety Act (OHSA) and in accordance with OPSS 902. For the purposes of the OHSA, the approach fill within the depth of excavation may be classified as Type 3 soil above the groundwater level and Type 4 soil below the groundwater level. The stiff silty clay within the depth of excavation may be classified as Type 3 soil. Flatter slopes may be required at locations where water seepage affects surficial stability.

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. It is anticipated that a hydraulic excavator will be suitable. Provision must be made for the handling of pavement materials, potential obstructions in the fill, and cobbles and boulders.

Roadway protection will be required for the staged construction at this site. Sheet piles or soldier pile & lagging walls are two options for roadway protection. The Contractor must select the wall type and design taking into account the soil conditions encountered in the boreholes and the lateral earth pressure parameters given in later sections of this report.

The roadway protection should be designed and constructed in accordance with OPSS 539. In general, the lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539. The design of roadway protection is the responsibility of the Contractor, and the shoring system must be designed by a Professional Engineer.

10 SHEET PILE WALLS

The current design proposes the installation of steel sheet pile walls adjacent to the pile foundations in lieu of conventional abutment walls. The sheet piles will provide containment and resistance to lateral earth pressures from the approach fill. The alignment of the proposed sheet pile walls should be carefully selected to avoid existing timber bents and piles.

Lateral stability of the sheet pile walls should be checked by the wall designer using the parameters presented in Table 10.1. The coefficients of passive earth pressure (K_p) are provided for horizontal ground surface in front of the sheet pile wall. For sloping ground in front of the sheet pile wall, the recommended values for the coefficients of passive earth pressure (K_p) should be reduced.

Table 10.1 – Soil Parameters for Sheet Pile Analysis

Foundation Element	Reference Borehole	Soil Unit	Elevation (m)		γ' (kN/m^3)	K_a	K_p
			Top	Bottom			
South Abutment	HRB-02	Fill	351.0*	349.0	21	0.33	3.0
		Silty Clay	349.0	330.0	9	0.39	2.6
North Abutment	HRB-05	Fill	351.0*	349.0	21	0.33	3.0
		Silty Clay	349.0	330.0	9	0.39	2.6

* Top of sheet pile elevation varies.

Driving of the sheet pile through the existing approach fill may encounter cobbles. Removal of any such obstructions may be required to install the sheeting. Any visible obstructions such as boulders and rock protection along the sides of the embankment should be removed prior to driving the sheet piles. No tip protections are required for these sheet piles.

Design of the permanent sheet pile walls must consider environmental conditions such as road salts or fluctuating water levels that may cause corrosion and reduce the service life of the structure. The native soils in front of the sheet piles should be protected from river erosion so that the sheet piles do not lose lateral support.

Backfill to the sheet pile walls should be in accordance with OPSS 902 and should consist of Granular A, Granular B Type II or Granular B Type III material. All granular material should meet the specifications of OPSS.PROV 1010. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS 501.

11 APPROACH EMBANKMENTS

Based on the latest GA drawing, the existing grade will be maintained at the both abutments. The new approach embankments will be retained by sheet pile walls to approximately 6 m behind the abutment walls. The foundation soils governing stability of the approach embankments consist of primarily stiff silty clay.

Global stability analyses were carried out to assess the stability of the forward slopes with the proposed sheet pile wall configuration. The stability analyses were carried out using the commercially available slope stability program GEO-SLOPE, applying the Morgenstern-Price method. The geotechnical model and results of the analyses are shown on Figures 1 to 4 in Appendix F. The computed factors of safety are summarized in Table 11.1.

Table 11.1 - Computed Factors of Safety for Approach Embankments

Abutment	Condition	Factor of Safety	Figure (Appendix F)
South	Short term - undrained	3.75	1
	Long term - drained	1.55	2
North	Short term - undrained	3.49	3
	Long term - drained	1.54	4

The computed factors of safety generally meet or exceed the minimum values of 1.3 and 1.5 normally accepted for this type of analysis under short and long term conditions, respectively. Global stability analyses carried out for the proposed sheet-pile enclosed approach embankments indicated that the foundation soils are considered to provide adequate stability with the tip of the sheet piles driven to Elev. 345.3 m and 344.0 m at the south and north abutments, respectively. The depth of penetration may need to be greater to provide lateral stability.

Settlement induced by the additional embankment fill is expected to be less than 40 mm at the both abutments. The settlements will be due to recompression of the silty clay deposit and will be essentially complete at the end of fill placement for the approach embankments. Any settlement noted at the approach fill should be brought up to design grade prior to placing the approach slab or pavement.

Embankment construction should be in accordance with OPSS.PROV 206. It is recommended that embankment fill consist of granular materials. All granular material should meet the specifications of OPSS.PROV 1010. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS 501. The backfill to the abutment walls should be in accordance with OPSS 902. Granular backfill should be placed to the extents shown in OPSD 3101.150.

12 SCOUR AND EROSION PROTECTION

Erosion protection should be provided along any soil surfaces that may be in contact with the river flow. In particular, erosion protection must be provided in front of the sheet pile walls to prevent undermining of the sheet pile walls at the abutments.

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

13 LATERAL EARTH PRESSURES

Earth pressures acting on the structure may be assumed to be distributed triangularly and to be governed by the characteristics of the abutment 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 = coefficient of lateral earth pressure (see Table 13.1)

γ = unit weight of retained soil (see Table 13.1)

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. Typical values are given in Table 13.1.

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

The factors in Table 13.1 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 Type III or at a depth of 1.7 m for Granular A or Granular B Type II.

Table 13.1 – Coefficients of Lateral Earth Pressure (K)

Conditions	**OPSS Granular A or Granular B Type II $\phi = 35°, \gamma = 22.8$ kN/m³**		**OPSS Granular B Type I or Type III $\phi = 32°, \gamma = 21.2$ kN/m³**	
	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	-	3.3	-

* For wing walls.

14 SEISMIC CONSIDERATIONS

The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.00
- Acceleration Related Seismic Zone 0
- Zonal Acceleration Ratio 0.00
- Peak Ground Acceleration 0.036 g

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

The foundation soils at the site are assessed as not being prone to liquefaction.

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.

For the design of retaining walls under seismic loading, the coefficients of horizontal earth pressure in Table 14.1 may be used:

Table 14.1 – Earth Pressure Coefficient for Earthquake Loading

Conditions	Granular A or Granular B Type II ϕ = 35°; γ = 22.8 kN/m³		OPSS Granular B Type I or Type III ϕ = 32°; γ = 21.2 kN/m³	
	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)	Horizontal Surface Behind Wall	Sloping Backfill (2H:1V)
Active (K_{AE})*	0.28	0.42	0.32	0.51
Passive (K_{PE})	3.6	-	3.2	-
At Rest (K_{OE})**	0.47	-	0.52	-

* After Mononobe & Okabe, passive case assumes a horizontal surface in front of the wall.
** After Woods (1973).

15 CONSTRUCTION CONCERNS

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

- There is a risk that pile driving for the replacement bridge may cause settlement of the existing bridge. It is recommended that settlement monitoring of the existing bridge be carried out for the duration of pile driving. The Contractor should be prepared with appropriate equipment on site to maintain the grade of the existing bridge within acceptable tolerances.
- Installation of the sheet piles retaining approach embankments may encounter resistance in the fill due to the presence of cobbles. The Contractor must allow for removal of any such obstructions.
- The Contractor's selection of construction equipment and methodology must include assessment of the capability of the clay subgrade to support the proposed construction equipment and any temporary structures or fill (i.e. as a pad for crane support). Site conditions may limit the type of equipment suitable for use. The design and safety of any temporary works is the responsibility of the Contractor.

16 CLOSURE

Engineering analysis and preparation of the foundation design report were carried out by Mr. Keli Shi, P.Eng. The report was reviewed by Mr. Murray R. Anderson, 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

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No HRB-01

METRIC

WP#	494-00-00	LOCATION	Hastie River Bridge N 5 390 852.8 E 255 350.8	ORIGINATED BY	GA
HWY	613	BOREHOLE TYPE	NW Casing	COMPILED BY	AN
DATUM	Geodetic	DATE	2013.11.01 - 2013.11.01	CHECKED BY	MC

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
351.4									
0.0	**ASPHALT:** (25mm)								
	Gravelly **SAND** Compact Brown Wet (FILL)	1	SS	20		351	o		
		2	SS	16	∇		o		34 65 1 (SI+CL)
349.9 1.5	Silty **CLAY**, trace to some sand Firm to Very Stiff Brown to Grey Moist	3	SS	7		350	o		
	Occasional iron oxide staining	4	SS	8		349	o		
		5	SS	12		348	o		
						347			
		6	SS	15			├o──┤		0 10 48 42
						346			
345.3 6.1	END OF BOREHOLE AT 6.1m UPON REFUSAL. BOREHOLE OPEN TO 6.1m AND WATER LEVEL AT 0.9m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.1m, CEMENT TO 0.07m, THEN ASPHALT TO SURFACE.	7	SS	50/ 0.0					

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: O UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB VANE; 20 40 60 80 100
PLASTIC LIMIT w_P; NATURAL MOISTURE CONTENT w; LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

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Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No HRB-02

METRIC

WP#	494-00-00	LOCATION	Hastie River Bridge N 5 390 867.4 E 255 353.3	ORIGINATED BY	GA
HWY	613	BOREHOLE TYPE	NW Casing/NQ Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2013.11.02 - 2013.11.02	CHECKED BY	MC

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
351.3					
0.0	**ASPHALT:** (40mm)				
	SAND, trace gravel, Compact, Brown, Wet (FILL)	1	SS	17	
		2	SS	15	8 89 3 (SI+CL)
349.8 / 1.5	Sandy **SILT**, some clay, Compact, Brown, Moist (FILL)	3	SS	11	
349.0 / 2.3	Silty **CLAY**, some sand, Stiff to Very Stiff, Brown to Grey, Moist	4	SS	11	0 15 47 38
		5	SS	8	
		6	SS	15	
		7	SS	13	
		8	SS	17	
		9	SS	12	0 11 32 57

Elevation scale: 351, 350, 349, 348, 347, 346, 345, 344, 343, 342

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m^3

Continued Next Page

$+^3$, $\times^3$: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No HRB-02

METRIC

WP# 494-00-00 LOCATION Hastie River Bridge N 5 390 867.4 E 255 353.3 ORIGINATED BY GA
HWY 613 BOREHOLE TYPE NW Casing/NQ Coring COMPILED BY AN
DATUM Geodetic DATE 2013.11.02 - 2013.11.02 CHECKED BY MC

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
	Continued From Previous Page													
	Silty **CLAY**, trace to some sand Stiff to Very Stiff Grey Moist						341							
			10	SS	9									
							340	>>2.0						
			11	SS	13		339							
							338							
			12	SS	14		337							
			13	SS	14		336				0	7	27	66
							335							
			14	SS	13		334							
			15	SS	8		333							
							332							

Continued Next Page

+3, ×3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No HRB-02

METRIC

WP#	494-00-00	LOCATION	Hastie River Bridge N 5 390 867.4 E 255 353.3	ORIGINATED BY	GA
HWY	613	BOREHOLE TYPE	NW Casing/NQ Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2013.11.02 - 2013.11.02	CHECKED BY	MC

Continued From Previous Page

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty **CLAY**, trace to some sand Firm Grey Wet	16	SS	7	331		
329.1 22.3	Sandy **SILT**, some clay Compact Grey Moist	17	SS	20	328		0 29 56 15
326.0 25.3	Silty **CLAY**, trace to some sand Stiff to Very Stiff Grey Moist	18	SS	12	325		
		19	SS	19	322		

Elevation scale: 331, 330, 329, 328, 327, 326, 325, 324, 323, 322

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m3

Continued Next Page

+3, ×3: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No HRB-02

METRIC

WP# 494-00-00 | LOCATION Hastie River Bridge N 5 390 867.4 E 255 353.3 | ORIGINATED BY GA

HWY 613 | BOREHOLE TYPE NW Casing/NQ Coring | COMPILED BY AN

DATUM Geodetic | DATE 2013.11.02 - 2013.11.02 | CHECKED BY MC

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page						
321.1 / 30.2	Sandy **SILT**, trace gravel Grey Boulder at 30.8m (200mm) Artesian pressure encountered (1.5m above ground surface dropping to surface level after 20mins)				321 320		
319.3 / 32.0	**BEDROCK**, slightly weathered to fresh, strong, coarse grained, grey/white to red/black: (GRANITE) Sand seam at 32.2m Horizontal joint at 32.2m, 32.5m, 33.0m Sub-vertical joint (150mm) at 32.2m Sub-vertical joint at 32.4m, 32.7m Highly broken zone (150mm) at 33.1m Horizontal joint at 33.7m Sub-vertical joint at 33.8m, 33.9m, 34.2m	1 2	RUN RUN		319 318 317	FI 2 4 1 >5 0 2 1 1 0 0	RUN #1 TCR=100% SCR=80% RQD=59% UCS=222MPa (Average) RUN #2 TCR=100% SCR=100% RQD=95% UCS=226MPa (Average)
316.3 / 35.1	END OF BOREHOLE AT 35.1m. BOREHOLE OPEN TO 35.1m AND WATER LEVEL AT 2.7m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Nov 04, 13 At Ground Surface						

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100
PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

+³, ×³: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

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Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No HRB-03

METRIC

WP# 494-00-00
LOCATION Hastie River Bridge N 5 390 866.5 E 255 361.4
ORIGINATED BY GA
HWY 613
BOREHOLE TYPE NW Casing
COMPILED BY AN
DATUM Geodetic
DATE 2013.11.04 - 2013.11.04
CHECKED BY MC

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
351.3 / 0.0	**ASPHALT:** (25mm)										
	SAND Compact Brown Wet (FILL)	1	SS	18		351	~15				
		2	SS	12			~17				
349.8 / 1.5	Sandy **SILT**, some clay Loose to Compact Brown Moist (FILL)	3	SS	10	∇ (at 2.1m)	350	~22	0	25	59	16
349.2 / 2.1	END OF BOREHOLE AT 2.1m. BOREHOLE OPEN TO 2.1m AND WATER LEVEL AT 2.1m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.07m, THEN ASPHALT TO SURFACE.										

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

+ ^{3}, × ^{3}: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

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THURBER

RECORD OF BOREHOLE No HRB-05

METRIC

WP# 494-00-00 | LOCATION Hastie River Bridge N 5 390 900.6 E 255 364.1 | ORIGINATED BY GA

HWY 613 | BOREHOLE TYPE NW Casing/NQ Coring | COMPILED BY AN

DATUM Geodetic | DATE 2013.11.03 - 2013.11.03 | CHECKED BY MC

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
351.3										
0.0	**ASPHALT:** (50mm)									
0.0	**SAND**, trace silt, occasional cobbles Compact Brown Wet (FILL)	1	SS	22	351	~12				
		2	SS	13		~19	0	97	3 (SI+CL)	
	Cobble	3	SS	50/ 0.0	350					
349.0 2.3	Silty **CLAY**, with sand Stiff to Very Stiff Brown to Grey Moist Occasional wood fibres	4	SS	9	349	~34				
		5	SS	10	348	~22	0	45	31	24
		6	SS	10	347	~35				
		7	SS	12	346 / 345	~33				
		8	SS	15	344 / 343	~28				
		9	SS	14	342	~32				

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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+³, ×³: Numbers refer to Sensitivity

20 / 15 / 5 / 10 (%) STRAIN AT FAILURE

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RECORD OF BOREHOLE No HRB-05

METRIC

WP#	494-00-00	LOCATION	Hastie River Bridge N 5 390 900.6 E 255 364.1	ORIGINATED BY	GA
HWY	613	BOREHOLE TYPE	NW Casing/NQ Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2013.11.03 - 2013.11.03	CHECKED BY	MC

Continued From Previous Page

Silty **CLAY**, some sand
Stiff to Very Stiff
Grey
Moist

Sample NUMBER	TYPE	"N" VALUES	Approx. Elevation (m)	Water Content (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
10	SS	13	340.3	~35	
11	SS	14	338.8	~23 (wP ~20, wL ~46)	0 11 34 55
12	SS	12	337.3	~35	
13	SS	16	335.7	~34	
14	SS	14	334.2	~37	
15	SS	11	332.7	~34	

Elevation scale: 341, 340, 339, 338, 337, 336, 335, 334, 333, 332

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

20 / 15 / 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No HRB-05

METRIC

WP#	494-00-00	LOCATION	Hastie River Bridge N 5 390 900.6 E 255 364.1	ORIGINATED BY	GA
HWY	613	BOREHOLE TYPE	NW Casing/NQ Coring	COMPILED BY	AN
DATUM	Geodetic	DATE	2013.11.03 - 2013.11.03	CHECKED BY	MC

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ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty **CLAY**, some sand Stiff Grey Moist	16	SS	14	331	
					330	
					329	
		17	SS	14	328	
					327	
					326	
	Varved	18	SS	13	325	0 12 40 48
					324	
					323	
		19	SS	10	322	

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3

Continued Next Page

+ ^{3}, × ^{3}: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

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THURBER

RECORD OF BOREHOLE No HRB-05

METRIC

WP# 494-00-00	LOCATION Hastie River Bridge N 5 390 900.6 E 255 364.1		ORIGINATED BY GA
HWY 613	BOREHOLE TYPE NW Casing/NQ Coring		COMPILED BY AN
DATUM Geodetic	DATE 2013.11.03 - 2013.11.03		CHECKED BY MC

Continued From Previous Page

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty **CLAY**, trace sand, varved Stiff to Firm Grey Moist				321			
					320			
		20	SS	7	319			0 6 94 (SI+CL)
318.4 / 32.9	**BEDROCK**, fresh, coarse grained, strong, grey/white/black: (GRANITE)						FI 5 >5	RUN #1 TCR=100% SCR=69% RQD=68% UCS=151MPa (Average)
	Highly broken zone (400mm) at 33.0m	1	RUN		318		0	
	Sub-vertical joint at 33.8m						2	
	Horizontal joint at 33.1m, 33.5m, 34.4m, 35.0m, 35.6m				317		2 2	RUN #2 TCR=100% SCR=98% RQD=93% UCS=220MPa (Average)
	Silt seam (50mm) at 34.4m						1	
	Sub-horizontal joint at 35.7m	2	RUN		316		0 1 1	
315.3 / 36.0	END OF BOREHOLE AT 36.0m. BOREHOLE OPEN TO 36.0m AND WATER LEVEL AT 0.3m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Nov 04, 13 At Ground Surface							

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

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

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No HRB-06

METRIC

WP#	494-00-00	LOCATION	Hastie River Bridge N 5 390 914.7 E 255 367.0	ORIGINATED BY	GA
HWY	613	BOREHOLE TYPE	NW Casing	COMPILED BY	AB
DATUM	Geodetic	DATE	2013.11.01 - 2013.11.01	CHECKED BY	MC

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
351.3 / 0.0	**ASPHALT:** (25mm)										
	SAND, some gravel, trace silt; Compact; Brown; Wet; (FILL)	1	SS	19		351					
		2	SS	13		350		14	84	2 (SI+CL)	
349.8 / 1.5	Silty **CLAY**, with sand to some sand, occasional rootlets, occasional black oxide staining; Stiff to Very Stiff; Brown to Grey; Moist	3	SS	10							
		4	SS	8		349		0	46	26	28
		5	SS	12		348					
						347					
		6	SS	15	▽	346					
		7	SS	16		345					
						344					
		8	SS	14		343	wP–w–wL	0	10	30	60
		9	SS	14		342					

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Continued Next Page

+ ³, × ³: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

ONTMT4S 5121.GPJ 2012TEMPLATE(MTO).GDT 7/17/14

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No HRB-06

METRIC

WP#	494-00-00	LOCATION	Hastie River Bridge N 5 390 914.7 E 255 367.0	ORIGINATED BY	GA
HWY	613	BOREHOLE TYPE	NW Casing	COMPILED BY	AB
DATUM	Geodetic	DATE	2013.11.01 - 2013.11.01	CHECKED BY	MC

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Continued From Previous Page										
	Silty **CLAY**, trace to some sand Stiff Grey Moist						341				
			10	SS	10						
340.0 11.3	END OF BOREHOLE AT 11.3m. BOREHOLE OPEN TO 11.3m AND WATER LEVEL AT 5.2m. BOREHOLE BACKFILLED WITH BENTONITE HOLEPLUG TO 0.1m, THEN ASPHALT TO SURFACE.										

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

ONTMT4S 5121.GPJ 2012TEMPLATE(MTO).GDT 7/17/14

Appendix B

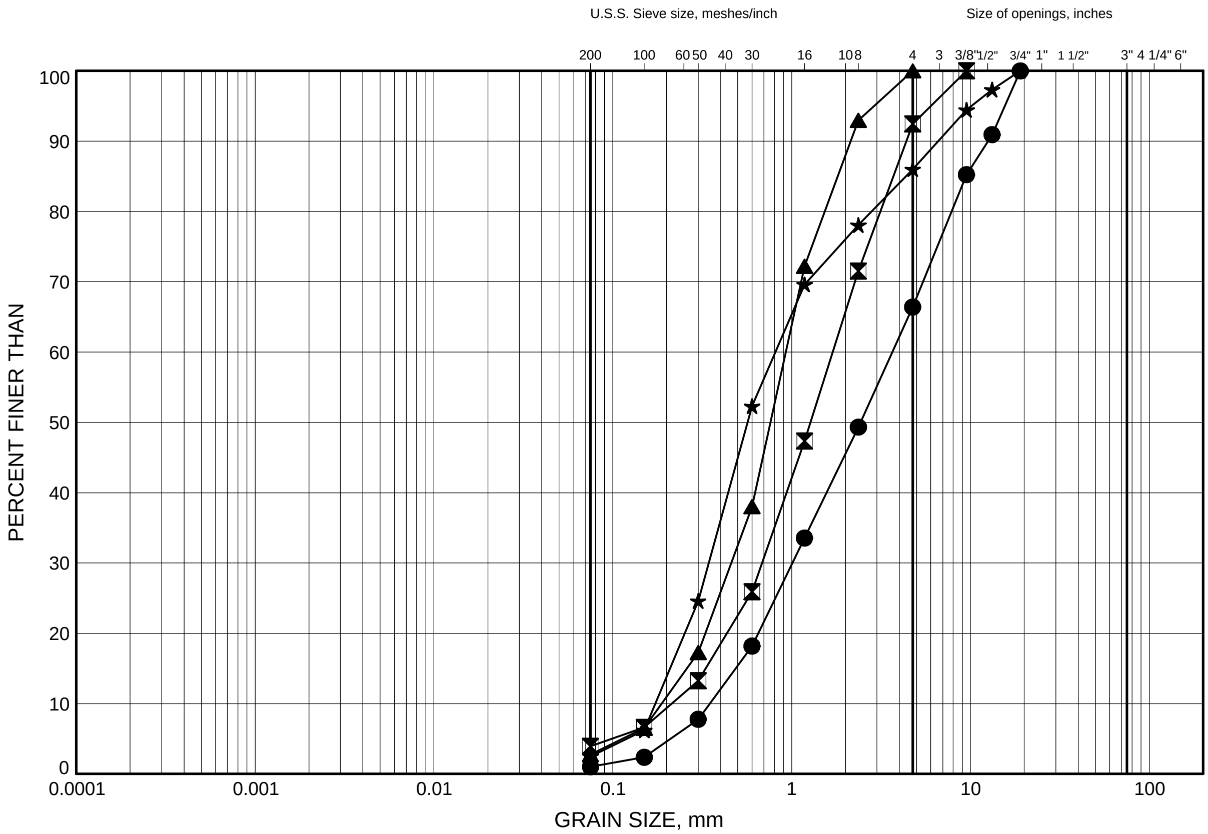
Laboratory Test Results

Hastie River Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B1a

Gravelly SAND to SAND (FILL)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	HRB-01	1.07	350.33
⧗	HRB-02	1.07	350.26
▲	HRB-05	1.07	350.22
★	HRB-06	1.07	350.26

Date July 2014

WP# 494-00-00

THURBER

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Chkd. KS

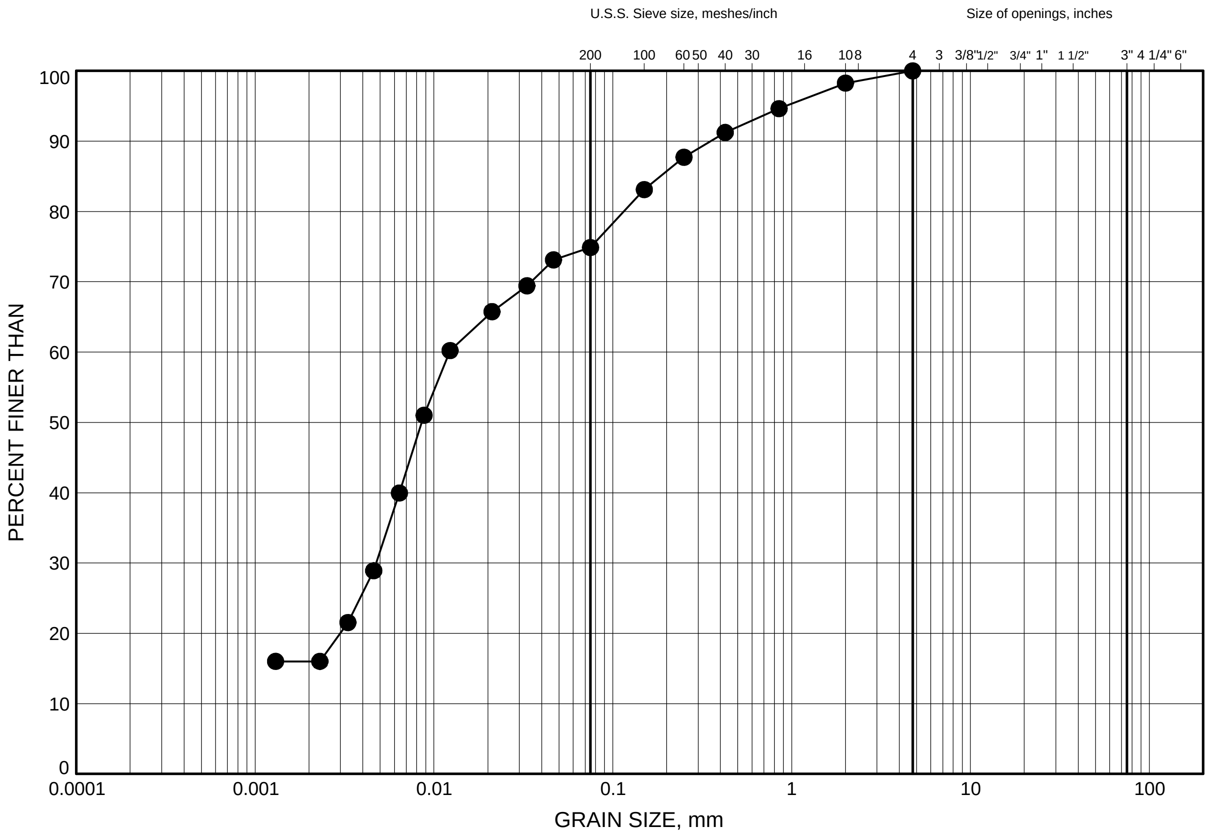
GRAIN SIZE DISTRIBUTION - THURBER 5121.GPJ 7/11/14

Hastie River Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B1b

Sandy SILT, Some Clay (FILL)



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	HRB-03	1.83	349.50

Date July 2014

WP# 494-00-00

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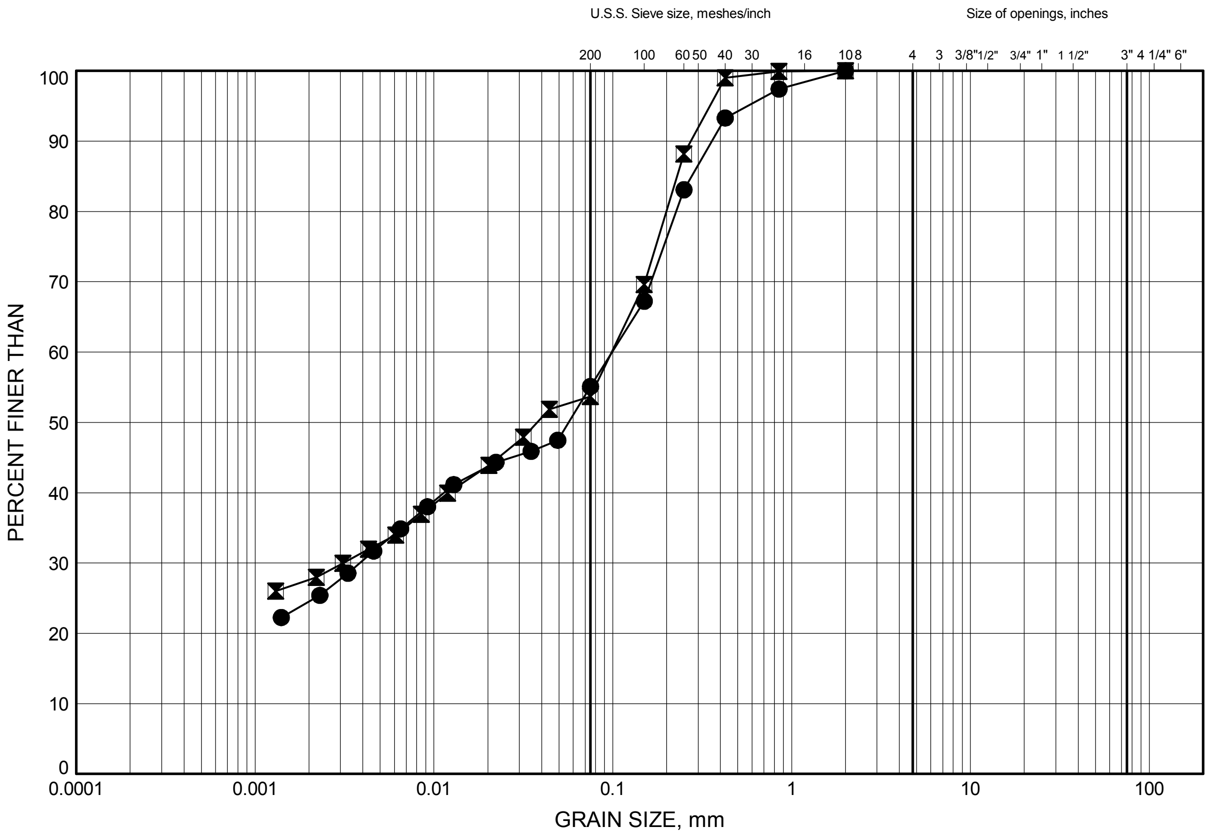
Chkd. KS

Hastie River Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B2

Silty CLAY, With Sand



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	HRB-05	3.35	347.93
⧗	HRB-06	2.59	348.73

GRAIN SIZE DISTRIBUTION - THURBER 5121.GPJ 7/11/14

Date July 2014

WP# 494-00-00

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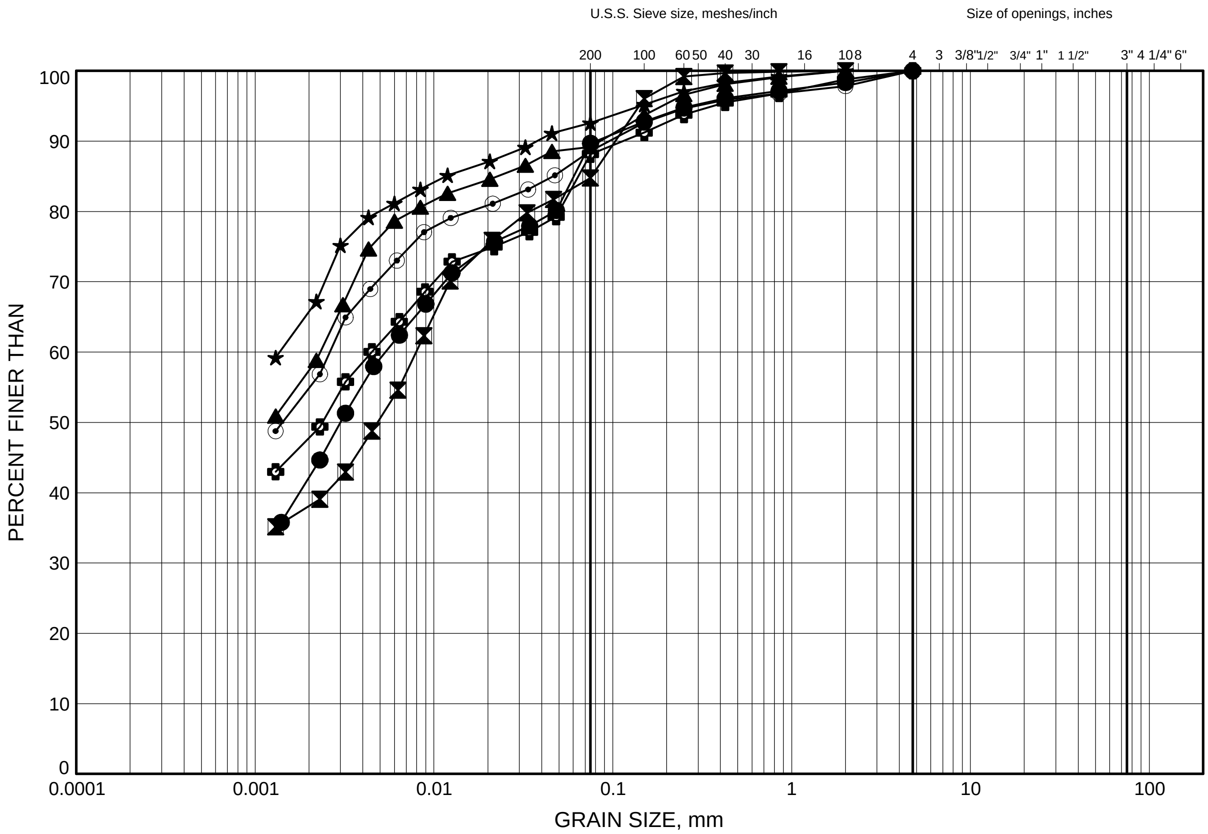
Hastie River Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B3a

Silty CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	HRB-01	4.88	346.52
⧗	HRB-02	2.59	348.73
▲	HRB-02	9.45	341.87
★	HRB-02	15.54	335.78
⊙	HRB-05	12.50	338.79
✿	HRB-05	26.21	325.07

Date July 2014

WP# 494-00-00

THURBER

Prep'd AN

Chkd. KS

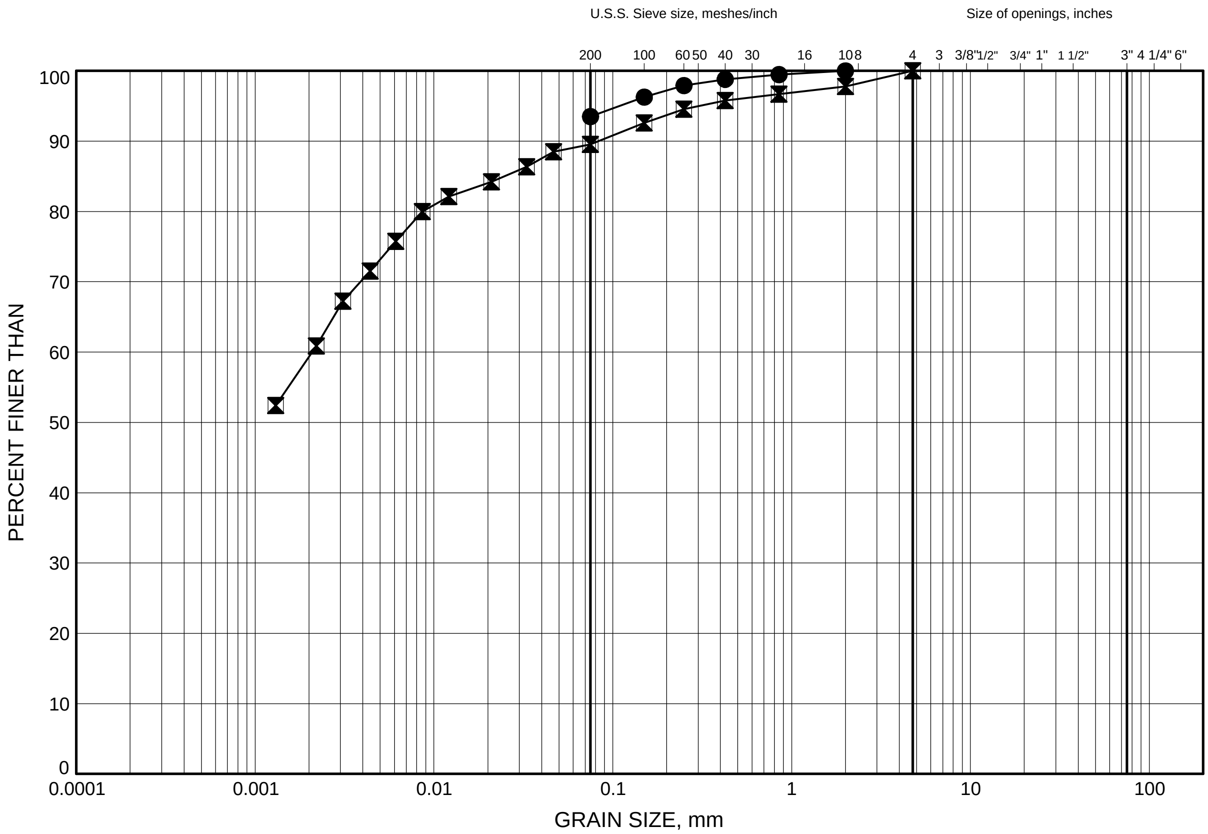
GRAIN SIZE DISTRIBUTION - THURBER 5121.GPJ 7/11/14

Hastie River Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B3b

Silty CLAY



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	HRB-05	32.31	318.98
⧗	HRB-06	7.92	343.40

Date July 2014

WP# 494-00-00

THURBER

Prep'd AN

Chkd. KS

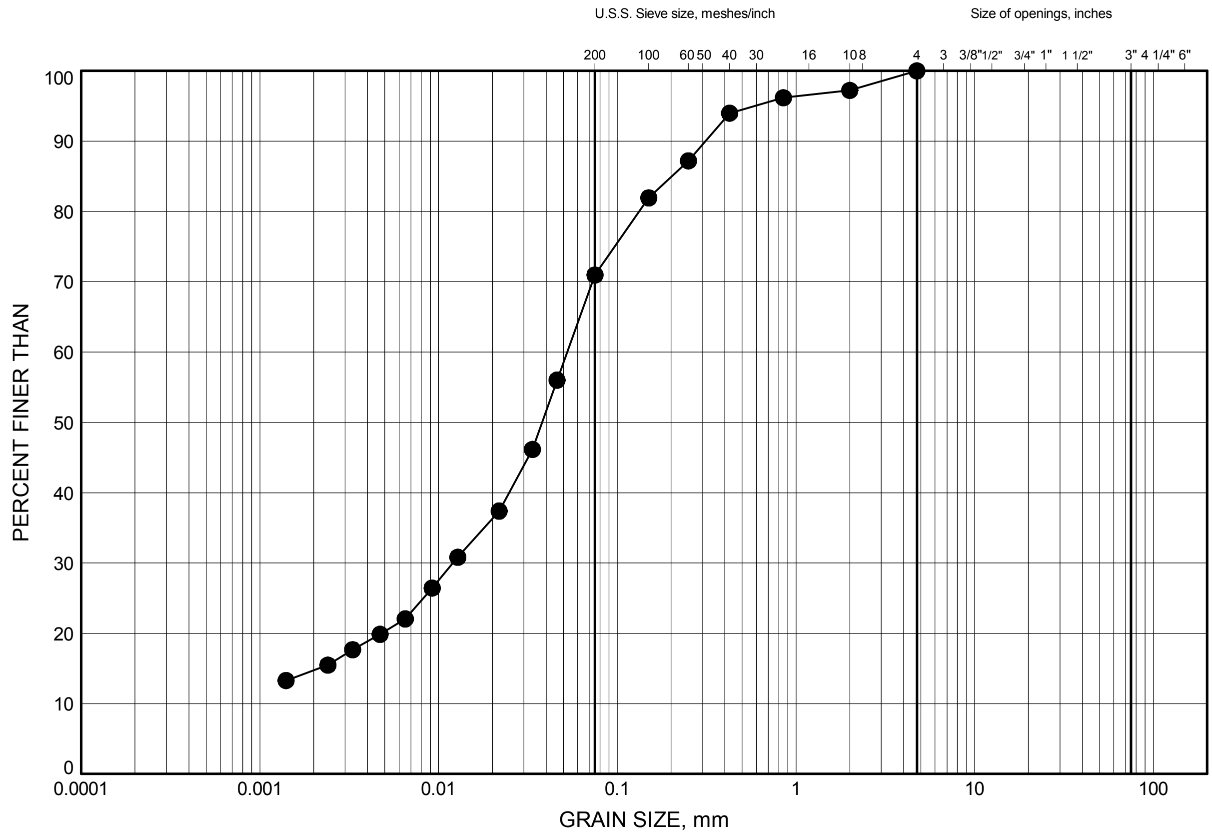
GRAIN SIZE DISTRIBUTION - THURBER 5121.GPJ 7/11/14

Hastie River Bridge

GRAIN SIZE DISTRIBUTION

FIGURE B4

Sandy SILT, Some Clay



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	HRB-02	23.16	328.16

GRAIN SIZE DISTRIBUTION - THURBER 5121.GPJ 7/11/14

Date July 2014

WP# 494-00-00

THURBER

Prep'd AN

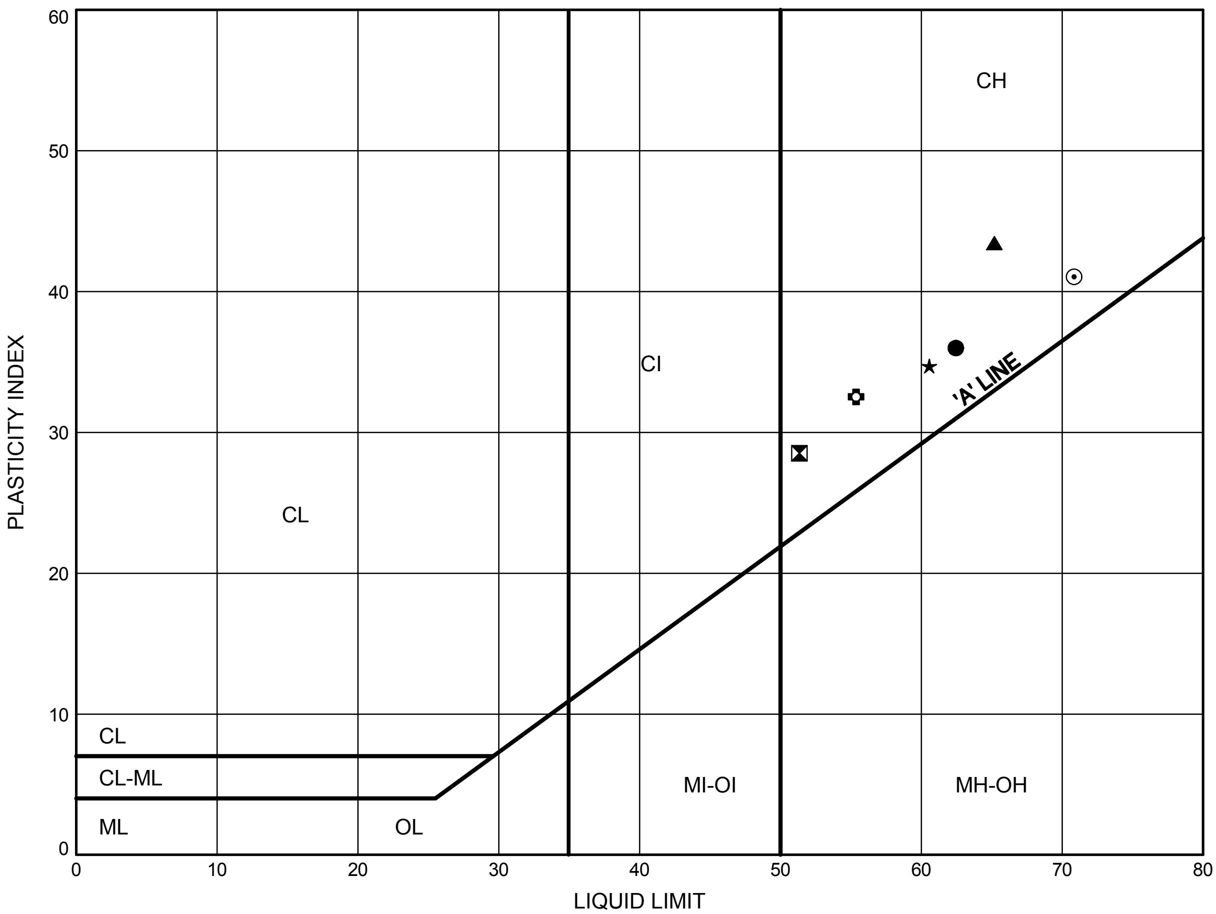
Chkd. KS

Hastie River Bridge

ATTERBERG LIMITS TEST RESULTS

FIGURE B5

Silty CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	HRB-01	4.88	346.52
⧗	HRB-02	2.59	348.73
▲	HRB-02	9.45	341.87
★	HRB-02	15.54	335.78
⊙	HRB-02	20.12	331.21
✿	HRB-05	12.50	338.79

THURBALT 5121.GPJ 7/18/14

Date July 2014
WP# 494-00-00

THURBER

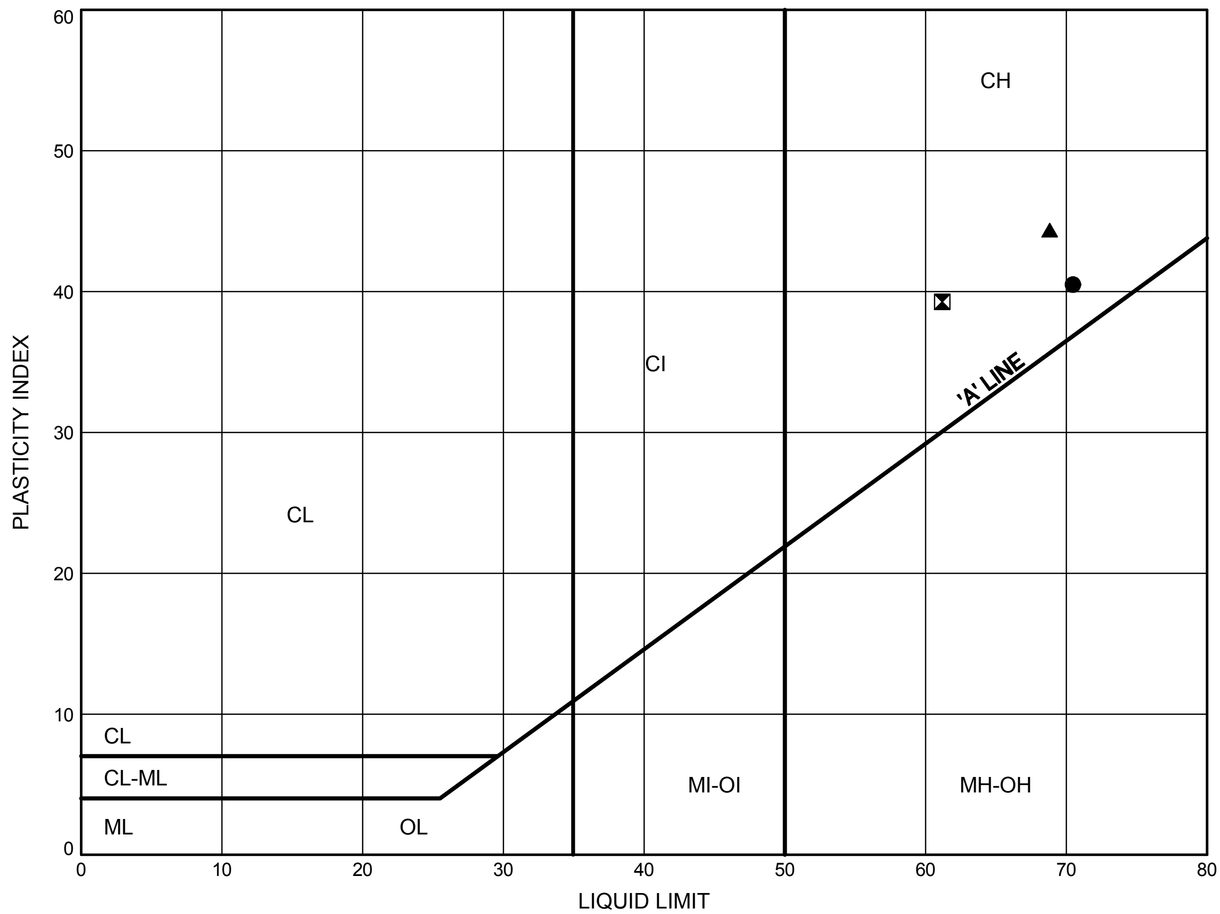
Prep'd MFA
Chkd. KS

Hastie River Bridge

ATTERBERG LIMITS TEST RESULTS

FIGURE B6

Silty CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	HRB-05	26.21	325.07
⧗	HRB-06	7.92	343.40
▲	HRB-06	10.97	340.35

Appendix C

Site Photographs

Photograph 1 – West elevation of the bridge, looking north

Photograph 2 – East elevation of the bridge, looking north

Photograph 3 – Upstream, looking west

Photograph 4 – Downstream, looking east

Appendix D

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Footings on Native Soil	Footings on Engineered Fill	Driven H-Piles	Caissons (Drilled Shaft)
Advantages i. Ease of construction. ii. Lower cost than deep foundations. ***Disadvantages:*** i. Relatively low geotechnical resistance is likely to be inadequate. ii. Large consolidation settlement likely due to the presence of deep compressible deposit. iii. Dewatering may be required, depending on depth of excavation and groundwater level at time of construction.	***Advantages:*** i. Generally less costly than deep foundation elements. ii. Allows use of perched abutments. iii. Higher geotechnical resistance than on native soil. ***Disadvantages:*** i. Cost of engineered fill placement. ii. Not feasible due to thick clay deposit overlying competent granular soil. iii. Dewatering may be required, depending on depth of excavation and groundwater level at time of construction.	***Advantages:*** i. High geotechnical resistance available for piles founded on bedrock. ii. Installation of piles could continue in freezing weather. iii. Allows integral abutment design. iv. Foundation construction may require less volume of excavation than footings. ***Disadvantages:*** i. Higher unit costs than footings. ii. Pile installation may encounter cobbles and boulders and meet refusal above bedrock surface.	***Advantages:*** i. High resistance is available for caissons founded on bedrock. ii. Construction of caissons could continue in freezing weather. ***Disadvantages:*** i. Higher cost than spread footings. ii. Cohesionless deposit below groundwater level at south abutment. iii. Deep bedrock level at abutments. iv. Possibility of cobbles and boulders being encountered during augering. v. Difficulty in cleaning and inspecting bases.
NOT RECOMMENDED	**NOT RECOMMENDED**	**RECOMMENDED**	**NOT RECOMMENDED**

Appendix E

List of Standard Specifications and Special Provisions

1) The following Standard Specifications and Special Provisions are referenced in this report:

OPSS.PROV 206
OPSS 501
OPSS 539
OPSS 804
OPSS 902
OPSS 903
OPSS.PROV 1010

OPSD 3101.150

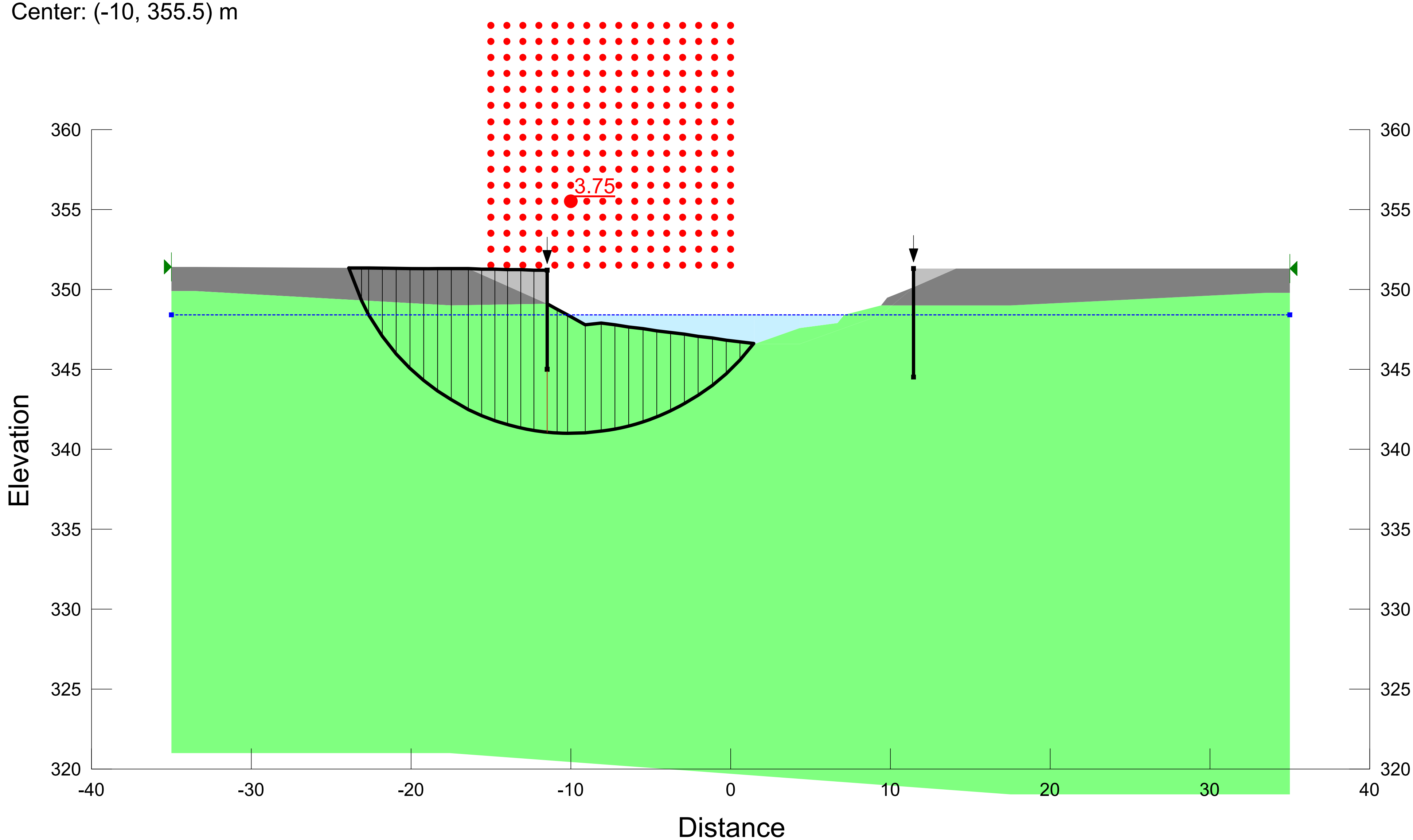
Appendix F

Select Runs of Slope Stability Analysis

Title: Highway 613, Hastie River Bridge
Comments: Abutment Stability Assessment
Name: South Abutment.TSA1

New FILL	21 kN/m³	0 kPa	32 °	1
Existing FILL	20 kN/m³	0 kPa	31 °	1
CLAY1 (TSA)	18 kN/m³	50 kPa	0 °	1

Method: GLE, Half-Sine
Minimum Slip Surface Depth: 1 m
Seismic: 0
Center: (-10, 355.5) m



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Last Solved Date: 2014-07-18, 9:15:22 AM
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Figure 1

Title: Highway 613, Hastie River Bridge
Comments: Abutment Stability Assessment
Name: South Abutment.ESA1

New FILL	21 kN/m³	0 kPa	32 °	1
Existing FILL	20 kN/m³	0 kPa	31 °	1
CLAY1 (ESA)	18 kN/m³	0 kPa	27 °	1

Method: GLE, Half-Sine
Minimum Slip Surface Depth: 1 m
Seismic: 0
Center: (-9, 353.5) m

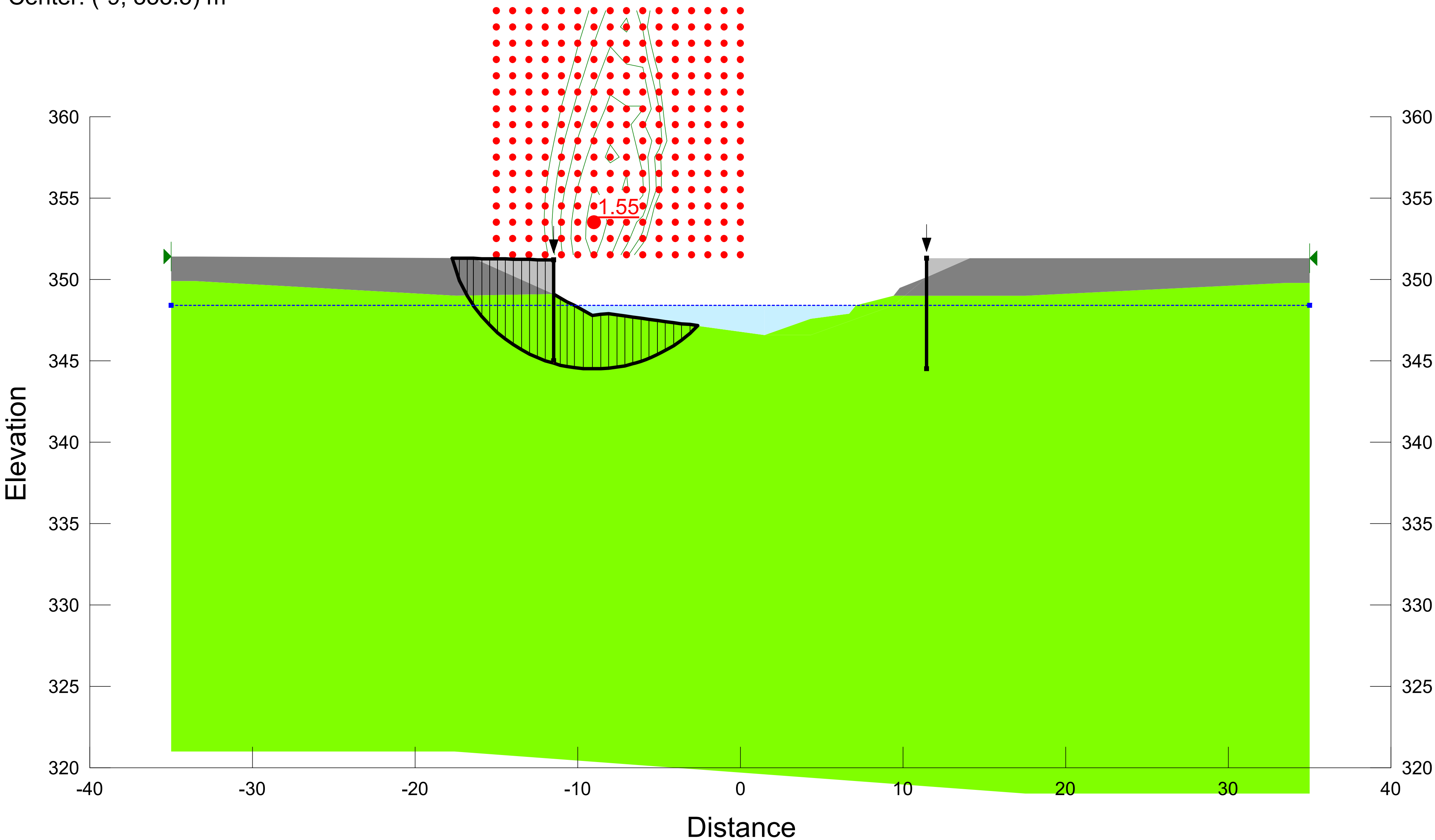

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Figure 2

Title: Highway 613, Hastie River Bridge
Comments: Abutment Stability Assessment
Name: North Abutment.TSA1

New FILL	21 kN/m³	0 kPa	32 °	1
Existing FILL	20 kN/m³	0 kPa	31 °	1
CLAY1 (TSA)	18 kN/m³	50 kPa	0 °	1

Method: GLE, Half-Sine
Minimum Slip Surface Depth: 1 m
Seismic: 0
Center: (10, 353.5) m

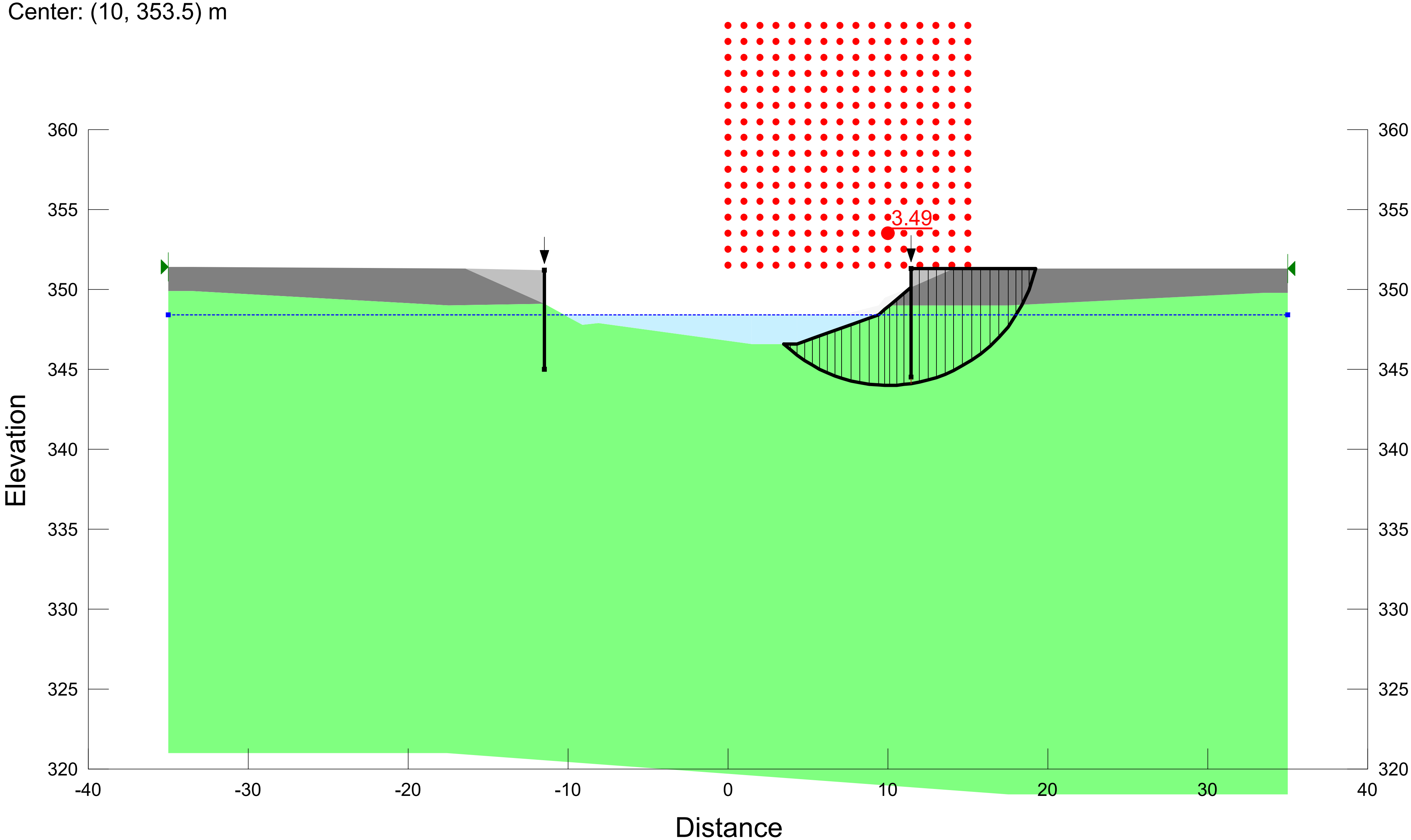

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Figure 3

Title: Highway 613, Hastie River Bridge
Comments: Abutment Stability Assessment
Name: North Abutment.ESA1

New FILL	21 kN/m³	0 kPa	32 °	1
Existing FILL	20 kN/m³	0 kPa	31 °	1
CLAY1 (ESA)	18 kN/m³	0 kPa	27 °	1

Method: GLE, Half-Sine
Minimum Slip Surface Depth: 1 m
Seismic: 0
Center: (8, 356.5) m

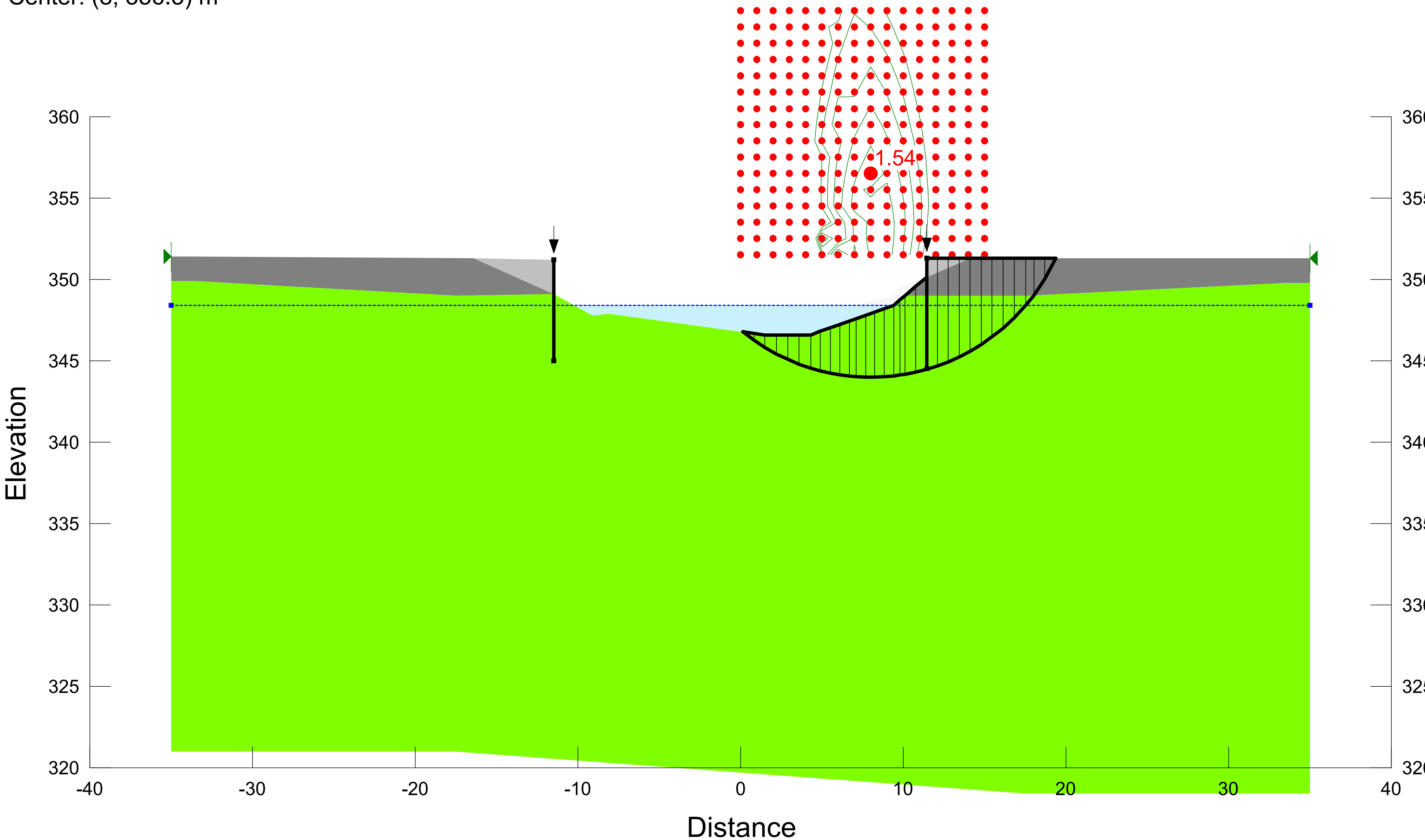

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Figure 4

Appendix G

Borehole Locations and Soil Strata Drawing

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

CONT No

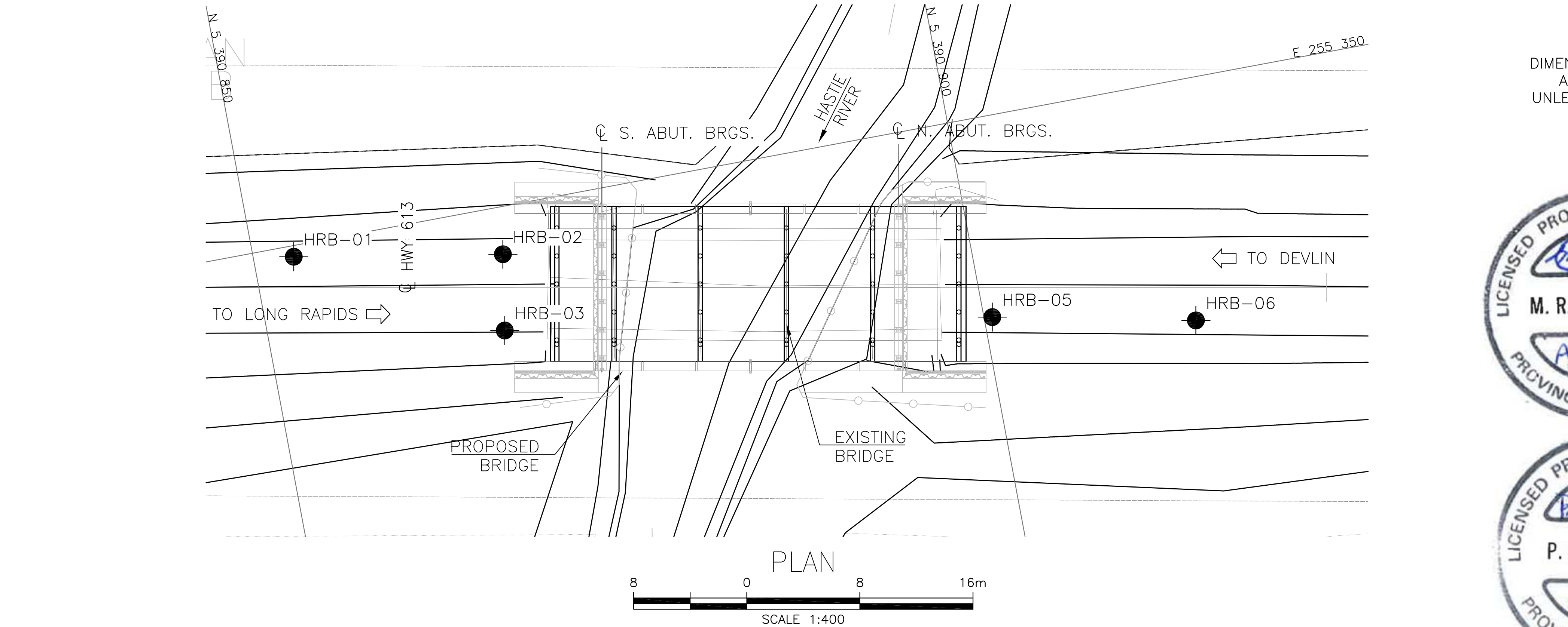
WP No 494-00-01

HIGHWAY 613
HASTIE RIVER BRIDGE
STRUCTURAL REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET 35

Hatch Mott MacDonald

THURBER ENGINEERING LTD.

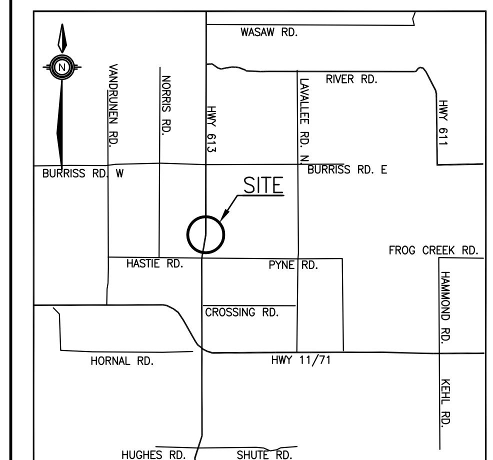


PLAN

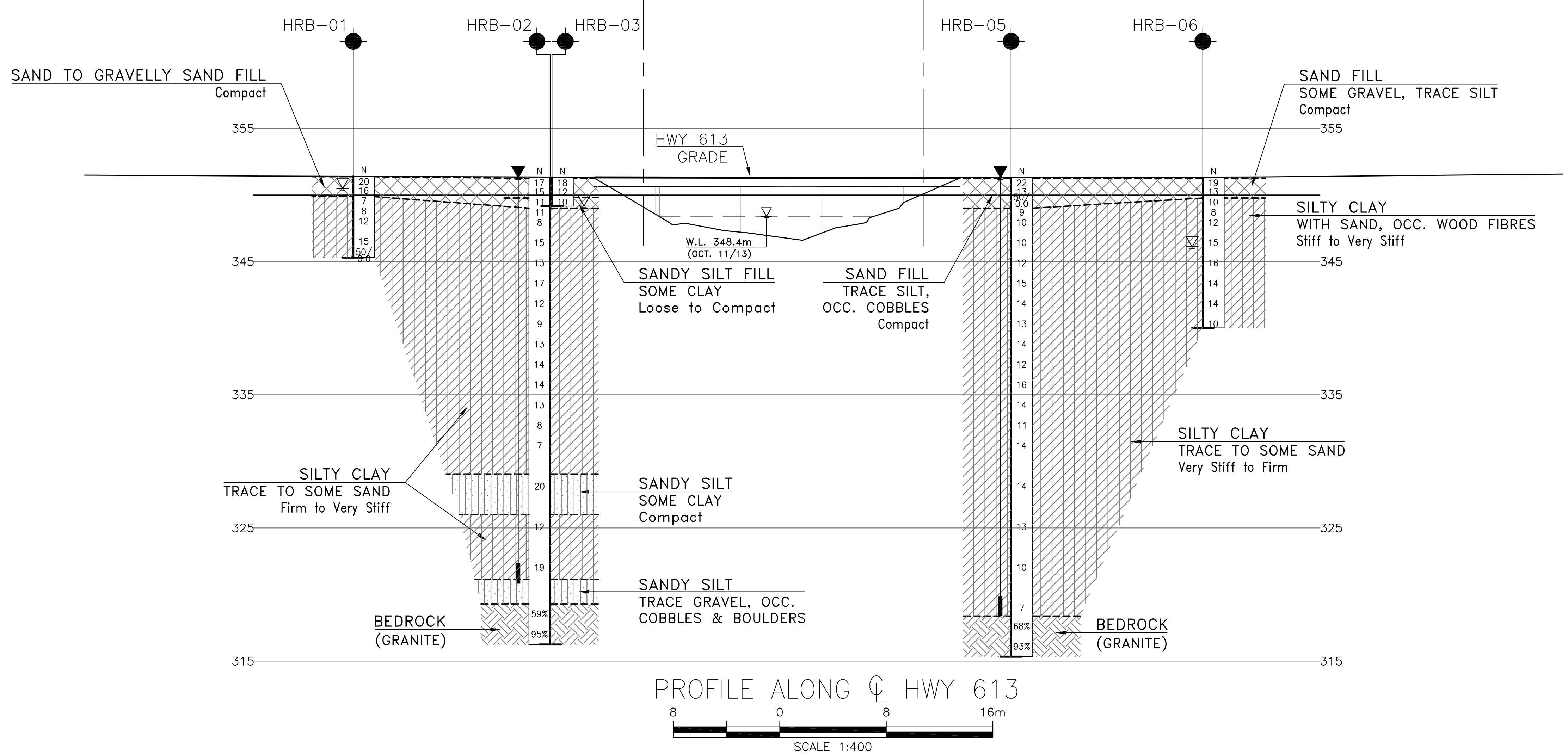
SCALE 1:400

LICENSED PROFESSIONAL ENGINEER
M. R. ANDERSON
Aug 8/14
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Aug 8/14
PROVINCE OF ONTARIO



KEYPLAN



PROFILE ALONG ℄ HWY 613

SCALE 1:400

LEGEND

●	Borehole
⊕	Borehole and Cone
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
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
HRB-01	351.4	5 390 852.8	255 350.8
HRB-02	351.3	5 390 867.4	255 353.3
HRB-03	351.3	5 390 866.5	255 358.7
HRB-05	351.3	5 390 900.6	255 364.1
HRB-06	351.3	5 390 914.7	255 367.0

-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. 52C-34

REVISIONS	DATE	BY	DESCRIPTION

DESIGN KS	CHK PKC	CODE CAN/CSA S6-06	LOAD CL-626-ONT	DATE	AUG 2014
DRAWN AN	CHK KS	SITE 45-68	STRUCT	DWG 2	

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