

**FOUNDATION INVESTIGATION AND DESIGN REPORT
SEVEN MILE BRIDGE REPLACEMENT
HIGHWAY 11
DISTRICT OF RAINY RIVER, ONTARIO**

G.W.P. 6048-08-00, Site No. 45-76

Geocres Number: 52C-28

Report to

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A Member of MMM Group**

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the location of the Seven Mile Bridge carrying Highway 11 over a section of Rainy Lake approximately 10 km northeast of Fort Frances, in the District of Rainy River, 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, stratigraphic profile, laboratory test results and a written description 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 McCormick Rankin, under the Ministry of Transportation Ontario (MTO) Agreement Number 6010-E-0011.

In the preparation of this report and in addition to the boreholes drilled under the current assignment, reference has been made to information on subsurface conditions and original bridge construction contained in previous reports. The titles of the previous reports are listed as follows:

- Soil Conditions and Engineering Study, Seven Mile Bridge Structure, Highway 120 Alignment, Rainy Lake Causeway, Fort Frances, Ontario, Geocres No. 52C-6, prepared by Geocon Ltd., dated June 15, 1960. (Reference 1).
- Report on Repairs to Seven Mile Bridge, Rainy Lake Causeway, Fort Frances, Ontario, prepared by Fenco, dated December 15, 1961. (Reference 2).

2 SITE DESCRIPTION

The Seven Mile Bridge is located on Highway 11 approximately 10 km northeast of Fort Frances in the District of Rainy River, Ontario. The bridge is part of the causeway crossing Rainy Lake.



The existing bridge is a nine-span structure with a total length of approximately 42.0 m and width of 10.7 m. The structure consists of a wood and concrete deck supported on timber pile bents supplemented by steel H-piles at the abutments and adjacent bents. The span lengths between pile bents are 3.7 m for the end spans adjacent to the abutments and 4.9 m for the remaining spans. Each pile bent consists of six timber piles spaced at 1.7 m, plus the supplemental H-piles.

The bridge approach embankments comprise a causeway constructed by end-dumping of fill in the lake channel. The embankments are approximately 4 m high. Rock protection is visible above the water level on the embankment slopes. A railway causeway and bridge run parallel to the Highway 11 causeway, approximately 30 m to the east.

Photographs in Appendix D show the general nature of the site.

The site lies within a boundary zone between the Wabigoon and Quetico Subprovinces of the Superior Structural Province. The region is characterized by gneissic domes and granitoid intrusions with metavolcanic and metasedimentary rocks occupying the margins. The bedrock is mantled by glaciolacustrine clays and sand and gravel deposits.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out during the period of October 15 to 20, 2011 and consisted of drilling and sampling six boreholes (numbered SMB-01 to SMB-06) through the highway embankment at the abutments and approaches.

Boreholes SMB-02 to SMB-05 were drilled adjacent to the existing abutments and terminated at depths ranging from 11.2 m to 21.9 m (Elev. 319.1 to 329.8). Boreholes were not drilled through the existing bridge, in order to avoid damaging the existing concrete and wood bridge deck. Drill rig access under the bridge was not practical.

Bedrock was proved in Boreholes SMB-02 and SMB-05 by NQ size diamond coring advanced 3.1 m and 3.8 m into bedrock. Boreholes SMB-03 and SMB-04 were terminated upon auger refusal on probable bedrock. A Dynamic Cone Penetration Test (DCPT) was conducted adjacent to each borehole to supplement the data.

Boreholes SMB-01 and SMB-06 were drilled through the approach embankments. Borehole SMB-01 was terminated upon auger refusal on probable bedrock at 11.2 m (Elev. 329.8) and Borehole SMB-06 was terminated in the fill layer at 10.5 m depth (Elev. 330.6).

The approximate locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix H.

The borehole locations were marked in the field and utility clearances were obtained prior to drilling.

The drilling was carried out using a CME 75 truck-mounted drill rig. Hollow stem augers and wash boring methods with NW-casing were used to advance the boreholes. Samples were obtained

at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). NQ coring methods were used to recover core samples from the bedrock.

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

All rock cores were logged, and the Total Core Recovery (TCR), Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined.

Two standpipe piezometers consisting of 19 mm PVC pipe with slotted screen and enclosed in filter sand were installed at this site to permit longer term groundwater level monitoring. The piezometers were subsequently decommissioned in general accordance with MOE Regulation 903. Boreholes without piezometers were backfilled in general accordance with Regulation 903. The installation and completion details of the boreholes and piezometers are summarized in Table 3.1.

Table 3.1 – Borehole and Piezometer Completion Details

Location	Borehole	Piezometer Tip Depth/ Elevation (m)	Completion Details
East Approach	SMB-01	None installed	Borehole backfilled with bentonite to 1.5 m, auger cuttings to 0.15 m, then asphalt to surface.
East Abutment	SMB-02	13.9 /327.1	Sand from 13.9 to 10.9 m, bentonite from 10.9 m to 0.9 m, then sand and gravel to surface.
	SMB-03	None installed	Borehole backfilled with bentonite to 1.5 m, auger cuttings to 0.15 m, then asphalt to surface.
West Abutment	SMB-04	None installed	Borehole backfilled with bentonite to 1.5 m, auger cuttings to 0.15 m, then asphalt to surface.
	SMB-05	21.9 /319.1	Sand from 21.9 m to 17.9 m, bentonite from 17.9 m to 1.5 m, sand from 1.5 m to 0.15 m, then asphalt to surface.
West Approach	SMB-06	None installed	Borehole backfilled with bentonite to 1.5 m, auger cuttings to 0.15 m, then asphalt to surface.

4 LABORATORY TESTING

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

Point load tests were carried out on selected samples of intact bedrock to assist in evaluation of the compressive strength of the bedrock. The results of point load tests on the rock core samples are presented on the Record of Borehole sheets in Appendix A.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A for details of the encountered soil stratigraphy. Stratigraphic profiles and cross-sections are presented on the “Borehole Locations and Soil Strata” drawings in Appendix H. 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. It must be recognized that soil conditions may vary between and beyond borehole locations.

In general terms, the stratigraphy encountered at this site consists of a pavement structure overlying sand and gravel embankment fill, underlain by discontinuous deposits of silty clay and sand and silt till, overlying bedrock. The fill contained rock fill locally in the west approach.

More detailed descriptions of the individual strata are presented below.

5.1 Pavement Structure

All boreholes were drilled through the pavement structure on Highway 11 and encountered a surficial 90 to 100 mm thick layer of asphalt. Granular fill was encountered below the asphalt.

5.2 Sand and Gravel Fill

Sand and gravel embankment fill was encountered in all boreholes drilled during the current investigation. In Boreholes SMB-01 to SMB-05, the lower boundary of the sand and gravel fill was encountered at depths ranging from 7.1 to 15.0 m (Elev. 326.0 to 333.9), deeper at the west abutment. Borehole SMB-06 was terminated within the sand and gravel fill at 10.5 m depth (Elev. 330.6).

The sand and gravel fill contains trace to some silt and occasional cobbles and boulders. Boulder sizes of 250 to 350 mm, locally 1300 mm for one boulder in Borehole SMB-06, were noted during drilling. Coring was required to advance through the boulders in some cases.

SPT N-values recorded in the fill ranged from 3 to 92 blows per 0.3m of penetration, indicating a very loose to very dense relative density. Typically the N-values were in the order of 8 to 28 blows per 0.3 m, indicating a loose to compact condition. Values of 85 blows per 0.23 m to 100 blows per 0.05 m of penetration were obtained where cobbles and boulders were encountered in the fill.

Moisture contents of samples of the sand and gravel fill typically ranged from about 3% to 10% above the observed water level and 10% to 24% below the water level.

The results of grain size distribution analyses conducted on samples of the sand and gravel fill are presented on the Record of Borehole sheets and on Figures B1 to B3 of Appendix B. The results of the laboratory tests are summarized as follows:

Gravel %	23 to 88
Sand %	11 to 64
Silt and Clay %	1 to 13

5.3 Silty Clay

Native grey silty clay containing trace sand was encountered below the fill in Boreholes SMB-01 and SMB-03, at depths of 7.1 and 9.4 m (Elev. 333.9 and 331.6), respectively. The silty clay layer was 2.8 and 1.8 m thick. In Borehole SMB-01, the lower boundary of the clay was encountered at 9.9 m depth (Elev. 331.1). Borehole SMB-03 was terminated below the silty clay at 11.2 m depth (Elev. 329.8), upon auger refusal on probable bedrock.

In Borehole SMB-01, SPT N-values of 1 and 9 blows per 0.3 m of penetration were recorded in the silty clay, indicating a stiff to very soft consistency. An undrained shear strength of 96 kPa (stiff) was measured in this layer. An SPT N-value of 24 blows per 0.3 m of penetration was recorded in Borehole SMB-03, indicating a very stiff consistency.

The moisture contents of samples of the silty clay ranged from 45% to 76%.

Grain size distribution curves for samples of the silty clay are presented in Figure B4, Appendix B. The results are also summarized on the Record of Borehole sheets included in Appendix A. Atterberg Limits test results from one sample are plotted on Figure B6 of Appendix B. The results of the laboratory tests are summarized as follows:

Gravel %	0
Sand %	1 to 3
Silt %	35 to 47
Clay %	50 to 64
Liquid Limit	61
Plastic Limit	24

The above results show that the silty clay is generally of high plasticity with a group symbol of CH.

5.4 Sand and Silt Till

Grey sand and silt till containing trace to some gravel was contacted below the silty clay at 9.9 m depth (Elev. 331.1) in Borehole SMB-01 and below the fill at 15.0 and 14.0 m depth (Elev. 326.0 and 327.0) in Boreholes SMB-04 and SMB-05, respectively. The till graded to fine sand with some gravel in Borehole SMB-05. The thickness of the sand and silt till ranged from 1.1 to 4.1 m.

Boreholes SMB-01 and SMB-04 were terminated below the sand and silt till at 11.2 and 16.1 m depth (Elev. 329.8 and 324.9), upon auger refusal on probable bedrock. In

Borehole SMB-05, the lower boundary of the till was encountered on bedrock at 18.1 m depth (Elev. 322.9).

SPT N-values recorded in the sand and silt till ranged from 18 to 44 blows per 0.3 m of penetration, indicating a compact to dense relative density. An N-value of 100 blows per 0.075 m was obtained at the base of the till on bedrock in Borehole SMB-04.

The moisture contents of the till ranged from 11% to 36%.

The results of grain size distribution analyses conducted on two samples of the sand and silt till are presented on the Record of Borehole sheets and on Figure B5 of Appendix B. The results are summarized as follows:

	Sand and Silt	Sand
Gravel %	12	15
Sand %	35	79
Silt %	44	6
Clay %	9	

Till deposits inherently contain cobbles and boulders, and these should be anticipated during construction.

5.5 Bedrock and Refusal

The soils described above are underlain by bedrock consisting of grey monzodiorite. Bedrock was proved by coring in Boreholes SMB-02 and SMB-05. Boreholes SMB-01, SMB-03 and SMB-04 were terminated upon auger refusal on probable bedrock. Table 5.1 summarizes the depths and elevations to the top of bedrock and auger refusal encountered in the boreholes.

Table 5.1 – Depths and Elevations of Top of Bedrock and Auger Refusal

Foundation Unit	Borehole	Top of Bedrock/Auger Refusal on Probable Bedrock	
		Depth (m)	Elevation (m)
East Approach	SMB-01	11.2	329.8
East Abutment	SMB-02	11.0*	330.0
	SMB-03	11.2	329.8
West Abutment	SMB-04	16.1	324.9
	SMB-05	18.1*	322.9

*Bedrock proved by coring.

Total Core Recovery (TCR) in the bedrock was generally 100%. A recovery of 87% was obtained in the final run from Borehole SMB-02. The measured RQD of the rock cores ranged from 82% to 100%, indicating a good to excellent rock quality.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, ranged from 0 to 2 with one value of 5.

The unconfined compressive strength of the rock interpreted from point load tests conducted on the recovered rock cores ranged from 151 to 197 MPa, indicating a very strong rock. The interpreted unconfined compressive strengths are indicated on the Record of Borehole sheets in Appendix A.

5.6 Water Levels

Water levels in the boreholes were measured upon completion of drilling. Water was added into the boreholes during wash-boring and coring operations, and therefore the water levels may not have been stabilized. Standpipe piezometers were installed in Boreholes SMB-02 and SMB-05 to monitor water levels after completion of drilling. The water levels measured in the piezometers and open boreholes are summarized in Table 5.2.

Table 5.2 – Water Level Measurements

Foundation Unit	Borehole	Date	Water Level (m)		Comments
			Depth	Elevation	
East Approach	SMB-01	October 15, 2011	4.9	336.1	In open borehole
East Abutment	SMB-02	October 16, 2011	4.2	336.8	In piezometer
		October 20, 2011	3.9	337.1	
		October 27, 2011	4.3	336.7	
West Abutment	SMB-03	October 20, 2011	3.9	337.1	In open borehole
	SMB-04	October 17, 2011	4.5	336.5	In open borehole
	SMB-05	October 16, 2011	4.5	336.5	In piezometer
		October 20, 2011	4.0	337.0	
		October 27, 2011	4.0	337.0	
West Approach	SMB-06	October 18, 2011	3.8	337.3	In open borehole

The water level in the lake is indicated on the preliminary GA drawing provided by McCormick Rankin to be at Elev. 336.7 in February 2011.

The groundwater levels will be controlled by the water level in the lake. The recorded values are short-term readings and fluctuations of the lake and groundwater levels are to be expected. The groundwater level may also be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

5.7 Previous Investigation (Reference 1)

A foundation investigation was conducted at this site in 1960 (Reference 1) shortly following construction of the causeway, approach embankments and bridge. The field investigation consisted of nine boreholes drilled through the approach embankments and three probes drilled adjacent to the embankment toe. The borehole logs, laboratory test results, and Borehole Plan and Soil Stratigraphy drawing from the previous investigation are reproduced in Appendix C.

The approximate locations of the previous boreholes (BH-1 to BH-9) and probes (P10 to P12) are included on the Boreholes Locations and Soil Strata drawing in Appendix H. The previous information was used to supplement the current borehole data during preparation of the soil profiles and cross-sections presented on this drawing. It must be noted that earthworks were continued on site subsequent to drilling of the 1960 boreholes, and the current site stratigraphy may vary from that recorded in the previous boreholes.

In general, the subsurface conditions documented during the 1960 investigation are consistent with the conditions encountered during the current investigation, comprising the following:

- A 300 to 900 mm thick layer of rock fill was encountered surficially in Boreholes 1, 2, 3, 5 and 6.
- Loose to compact granular fill was encountered below the rock fill or surficially in all boreholes. The granular fill extended to depths of 1.2 to 11.9 m (Elev. 335.7 to 328.0).
- In Boreholes 7 to 9 drilled on the west approach, a 2.4 to 5.2 m thick layer of rock fill was encountered below the granular fill. The lower boundary of the rock fill was at depths of 4.9 to 11.6 m (Elev. 332.5 to 328.0).
- Native soft to firm, grey, varved silty clay was contacted below the granular fill and rock fill in Boreholes 2 to 9, as well as surficially in Probes 10 to 12. The thickness of the silty clay ranged from 0.3 m in Borehole 3 to 14.0 m in Probe 11. In general, the thickness of the clay increased towards the perimeter of the embankments, indicating that the clay had been largely, but incompletely, displaced during fill placement. The lower boundary of the clay layer varied between depths of 8.5 and 14.3 m (Elev. 329.9 and 324.2).
- Compact to dense, grey sandy till was encountered below the fill and silty clay in all boreholes and probes except Probe 10. The thickness of the sandy till layer ranged from 0.3 to 3.7 m.
- Bedrock or probable bedrock/boulders was contacted below the sandy till in Boreholes 1 to 4 and 6 to 8 at the following depths and elevations:

Table 5.3 – Depths and Elevations of Top of Bedrock (Previous Investigation)

Location	Borehole	Top of Bedrock/Auger Refusal	
		Depth (m)	Elevation (m)
East Approach	1	10.1*	330.6
	2	9.4*	328.9
	3	10.8	329.6
West Approach	4	16.8*	323.1
	6	13.4	325.1
	7	17.9	321.0
	8	15.4	322.0

*Bedrock proved by coring

- The water level observed in the boreholes during the investigation (March/April 1960) was at elevation 336.9.

The report also documents that the causeway and approach embankments were constructed by end-dumping of rock fill and granular material into the lake during November 1958 and March 1959. During construction, “several minor spreading-type failures” were reported, and lateral failures occurred on both sides of the south approach embankment.

Timber piles for bridge construction were driven during April and May 1959. Settlement and rotational movement of the two or three pile bents at each end of the bridge was observed shortly after pile installation. Consequently, steel H-piles were driven at the first two bents at each end of the bridge to supplement the timber piles.

6 MISCELLANEOUS

Borehole locations were selected and established in the field by Thurber Engineering Ltd. MMM Group Limited surveyed the borehole locations and provided the co-ordinates and the ground surface elevations.

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

Eastern Ontario Diamond Drilling Ltd. from Hawkesbury, Ontario supplied a truck mounted CME 75 drill rig and conducted the drilling, sampling and in-situ testing operations.

The drilling and sampling operations were supervised in the field on a full time basis by Mr. Jason Mei of Thurber Engineering Ltd.

Routine laboratory testing was carried out by Thurber Engineering Ltd.

Overall planning and supervision of the field program was conducted by Mr. Mark Farrant, P. Eng.

Interpretation of the data and preparation of the report was carried out by Ms. R. Palomeque Reyna, 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.

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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 for the proposed replacement of the existing Seven Mile Bridge on Highway 11 approximately 10 km northeast of Fort Frances in the District of Rainy River, Ontario.

The Seven Mile Bridge is part of the Highway 11 causeway crossing Rainy Lake. The existing bridge is a nine-span structure with a total length of approximately 42.0 m and width of 10.7 m. The structure consists of a wood and concrete deck supported on timber pile bents supplemented by steel H-piles at the abutments and adjacent bents. The span lengths between pile bents are 3.7 m for the end spans adjacent to the abutments and 4.9 m for the remaining spans. Each pile bent consists of six timber piles spaced at 1.7 m, plus the supplemental H-piles.

The bridge approach embankments were constructed by placement of fill partially across a section of the lake, advancing from both shorelines towards the bridge location. The embankments are approximately 4 m high.

Based on the preliminary General Arrangement (GA) drawing provided by McCormick Rankin, a new single-span bridge is proposed to replace the existing bridge. The new structure will consist of a precast concrete box girder deck supported on 324 mm diameter steel pipe piles filled with concrete and socketed into bedrock. The bridge will have a span of 35.0 m and a width of 12.4 m. Steel sheet piling will be installed behind the pipe piles to retain the approach fill.

The new bridge will be constructed along the same alignment as the existing structure, and no grade raise is planned. A maximum 1.8 m thickness of fill will be placed within the wedge-shaped area enclosed by the sheet pile abutments. To enable widening of the highway platform, low retaining walls are planned along both sides of the west approach and the south side of the east approach.

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

8 SITE BACKGROUND

Information regarding site conditions and events occurring during construction of the existing bridge and causeway in 1958 to 1961 are provided in the existing reports obtained from MTO Geocres files (References 1 and 2). Of particular note to the selection and design of foundations for the replacement bridge, the following site background is documented:

- The approach embankments were constructed by end-dumping in the lake during November 1958 and March 1959. During construction, “several minor spreading-type failures” were reported, and lateral failures occurred on both sides of the south approach embankment.
- Timber piles for bridge construction were driven during April and May 1959. The timber piles were driven generally into the underlying till stratum or to bedrock. The reported length of installed piles ranged from 8.8 to 15.8 m.
- Settlement and rotational movement of the two or three pile bents at each end of the bridge was observed shortly after pile installation. Between May 1959 and January 1961, the maximum recorded settlement of individual piles was about 150 mm. Maximum translational movements of the pile bents due to rotation of the piles occurred at the south end bent and the second and third bents from the north end, where approximately 280 mm of movement towards the south was measured at each bent.
- Repairs to the bridge were carried out after evaluation of the pile and embankment movements. To supplement the timber piles, five steel H-piles were driven to bedrock at the first two bents at the north end of the bridge as well as at the second bent from the south end. At the south abutment bent, two rows of six H-piles were driven to bedrock, with the front row battered at 1H:6V.

9 STRUCTURE FOUNDATIONS

The borehole information and documented site history indicates that the existing approach embankments were constructed by end-dumping of rock fill and granular fill onto the lake bed, largely but incompletely displacing the soft silty clay deposit underlying the site. The rock fill was encountered surficially as well as below the granular fill in boreholes located west of the west abutment. The granular fill consists of sand and gravel with occasional cobbles and boulders.

A layer of compact to dense sand and silt till was encountered below the fill and clay. Bedrock and probable bedrock were encountered at depths of 9.4 to 11.2 m (Elev. 328.9 to 330.6) at the east approach, and at depths of 13.4 to 18.1 m (Elev. 321.0 to 325.1) at the west approach.

The preliminary GA drawing indicates that the water level in the lake was at Elevation 336.7 in February 2011. Water level measured in the piezometers during the current investigation ranged between Elevations 336.5 and 337.1, approximately 3.9 to 4.5 m below the existing road grade.

Based on the existing site conditions, initial consideration was given to the following foundation types:

- Spread footings on native soils, fill or bedrock
- Driven steel H-piles founded on bedrock.
- Drilled-in pipe piles socketed into bedrock
- Augered Caissons (drilled shafts)

A comparison of the foundation alternatives based on advantages and disadvantages of each is presented in Appendix E.

9.1 Spread Footings on Native Soils, Fill or Bedrock

Consideration was given to supporting the structure on spread footings founded on native soils, existing embankment fill or bedrock. However these options are not recommended for the following reasons:

- Competent native soils are not present within a reasonable depth and construction would require cofferdam enclosures to enable foundation excavation extending below the lake water level.
- The existing fill materials are variable in relative density, are underlain by incompletely displaced soft clay, and are considered unsuitable for support of spread footings.
- Bedrock is not located within a reasonable depth for footing construction.

In light of the above factors, the spread footing option was not further developed.

9.2 Steel H-piles

Consideration may be given to supporting the proposed single-span bridge on steel H-piles driven to refusal on bedrock.

The piles are expected to meet refusal on bedrock at the depths (below existing road grade) and elevations given in Table 9.1.

Table 9.1 – Anticipated Pile Tip Elevations on Bedrock

Foundation Unit	Reference Boreholes	Anticipated Pile Refusal on Bedrock	
		Depth below existing road grade (m)	Elevation (m)
East Abutment	SMB-02, SMB-03, BH-1, BH-2	11.5	329.5
West Abutment	SMB-04, SMB-05, BH-4, BH-6	18.0	323.0

The axial, factored geotechnical resistance at Ultimate Limit States (ULS) for a 310x110 H-Pile section driven to refusal on bedrock is 2,000 kN.

The SLS condition will not govern for piles founded on the bedrock.

The factored structural resistance of the piles at ULS must be checked by the structural designer as per Section 6.8.8 of the CHBDC.

Cobbles and boulders were encountered within the embankment fill during the current investigation. Boulders and rock slabs are present on the embankment slopes and a surficial layer of rock fill was documented on the boreholes logs from the previous investigation. The cobbles, boulders and rock fill may interfere with pile installation and some piles may meet refusal on boulders or rock fill above the bedrock surface.

9.2.1 Pile Tips

The tips of all H-piles should be fitted with pile tip protection from an approved manufacturer such as Titus Steel (Standard H-point) or approved equivalent. Pile tip protection is recommended to prevent pile damage when driving through fill containing cobble and boulders and when setting the piles on bedrock.

9.2.2 Pile Installation

Pile installation should be in accordance with OPSS 903. The foundation drawing should include the note “Piles to be driven to bedrock”.

If the proposed bridge design requires that the deviation at the top of the pile be limited to tolerances more stringent than those specified in Clause 903.07.05.01 of OPSS 903, a driving template or other means may be required to achieve the specified maximum deviation.

The possibility exists that cobbles, boulders and/or rock fill may be encountered while driving piles through the existing embankment fill. The Contract Documents should contain a NSSP alerting Bidders to:

- the presence of the cobbles, boulders and rock fill within the embankment fill;

- The possibility of some piles being deflected or meeting refusal on large boulders or rock fill above the bedrock surface; and
- The need to remove, dislodge or otherwise penetrate these obstructions to advance the piles to bedrock while meeting the specified deflection tolerances.

An NSSP addressing these issues is included in Appendix F.

Oversize materials (e.g. greater than 75 mm nominal diameter) must not be used in any additional fill through which the piles will be driven.

9.2.3 Lateral Resistance for Piles

The lateral resistance of the piles within cohesionless fill may be assessed using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) calculated as follows:

$$k_s = n_h z / D \quad (\text{kN/m}^3)$$

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

where

$$z = \text{depth of embedment of pile in metres}$$

$$D = \text{pile width in metres}$$

$$n_h = \text{value from Table 9.2}$$

$$\gamma = \text{unit weight (Table 9.2)}$$

$$K_p = \text{passive earth pressure coefficient (Table 9.2)}$$

For cohesive soils, the lateral resistance of the piles may be calculated as follows:

$$k_s = 67 S_u / D \quad (\text{kN/m}^3)$$

$$p_{ult} = 9 S_u \quad \text{at and below a depth of } 3D, \\ \text{reduced to zero at the ground surface (kPa)}$$

where

$$D = \text{pile width in metres}$$

$$S_u = \text{undrained shear strength (kPa)}$$

Based on the previous and the present investigations, parameters for lateral pile resistance are shown in Table 9.2. The unit weights provided in the table for soils below the groundwater level are buoyant (effective) unit weights for use in the lateral resistance calculations.

Table 9.2 - Parameters for Lateral Pile Resistance

Location	Elevation	n_h (kN/m ³)	S_u kPa	K_p	Unit Weight (kN/m ³)	Soil Conditions
West Abutment	338.0 to 337.0	6,000	-	3.3	21	Sand and Gravel FILL, compact
	337.0 to 329.0	4,000	-	3.3	11*	Sand and Gravel FILL, loose to compact
	329.0 to 326.5	-	50	2.7	10*	Silty clay, very soft to stiff
	326.5 to 323.0	4,000	-	3.0	11*	Sand and silt till, compact
East Abutment	338.0 to 337.0	6,000	-	3.3	21	Sand and Gravel FILL, compact
	337.0 to 332.0	4,000	-	3.3	11*	Sand and Gravel FILL, loose to compact
	332.0 to 330.5	-	50	2.7	10*	Silty clay, soft to firm
	330.5 to 329.5	4,000	-	3.0	11*	Sand and silt till, compact

*Buoyant unit weight below the water table.

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

The spring constant, K , for analysis may be obtained by the expression, $K = k_s L D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m³), 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 on any one segment of pile, P_{ult} , may be obtained from the expression, $P_{ult} = p_{ult} L D$. 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 a HP 310x110 pile driven to refusal on bedrock be limited to no more than 120 kN at ULS and 50 kN at SLS.

For lateral soil/pile group interaction analysis, the modulus of subgrade reaction (k_s) may have to be reduced based on pile spacing.

Where a pile group is oriented *perpendicular* to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Pile Spacing Perpendicular to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
4 D^*	1.00
1 D^*	0.50

* D is the width of the pile, and spacing is measured centre to centre

Where a pile group is oriented *parallel* to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Pile Spacing Parallel to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
8 D	1.00
6 D	0.70
4 D	0.40
3 D	0.25

Intermediate values may be obtained by interpolation.

9.3 Drilled-in Pipe Piles

The replacement bridge may be supported on drilled-in steel pipe piles socketed into bedrock and filled with concrete. It is recommended that the piles be advanced a minimum 1.0 m into bedrock to confirm that any fractured rock and cobbles/ boulders at the bedrock surface are penetrated and to fix the pile tip in place. Due to the presence of cobbles, boulders and rock fill in the embankment material, driven pipe piles are not recommended.

The capacity of concrete-filled pipe piles socketed into bedrock will be dictated by the structural resistance of the composite pile section and will not be governed by the geotechnical resistance of the bedrock. The factored geotechnical resistance recommended for selected pipe pile sections are presented in Table 9.3.

Table 9.3 – Factored Geotechnical Resistance of Drilled-in Pipe Piles

Pipe Pile Section		Factored Geotechnical Resistance (kN)
Outer Diameter (mm)	Wall Thickness (mm)	
324	12.7	2,000
406	12.7	2,800
508	12.7	4,000
610	12.7	5,500

The resistance values presented above assume a steel yield strength of 245 MPa and a concrete compressive strength of 35 MPa. The resistances have been reduced to account for the possibility that residual crushed rock may remain in the rock socket. The depth of the socket may need to be greater than 1.0 m to address the lateral resistance requirement, base fixity requirement and shear and moment demand for each pile.

9.3.1 Socket Lateral Resistance

The ultimate passive force that can be mobilized by the embedded portion of a pipe pile socket within rock is constant with depth and is given by:

$$P_p = 6 c D L$$

Where $c = 2,000 \text{ kPa}$ (equivalent Mohr-Coulomb cohesion based on Hoek and Brown rock mass classification)

$L = \text{Depth of socket in rock, m}$

$D = \text{Socket diameter, m}$

Lateral resistance of the section of pipe pile located above the bedrock surface may be computed using the parameters in Section 9.2.3.

9.3.2 Drilled-in Pipe Pile Installation

Installation of pipe piles must follow OPSS 903 specifications.

The method of installation of the pipe piles is the responsibility of the Contractor. One option for installing pipe piles is to drill them in using a concentric drilling method such as the Symmetrix system. The Contractor's drilling method must be capable of dislodging, removing or penetrating obstructions such as cobbles, boulders or rock fill in the granular fill and overburden soils. Care must be exercised while drilling into the bedrock; the drilling methodology must be capable of advancing the pile without disturbing or fracturing the bedrock at the base of the pile. Blasting to facilitate rock removal is not permitted.

Since the rock cutting shoe at the tip of a pipe pile will be slightly larger in diameter than the outside diameter of a pipe pile, there will be a small gap between the rock socket wall and the pipe pile. It is recommended that the annular space between the pipe pile and socket wall be grouted to the bedrock surface to achieve fixity.

During and subsequent to installation, the pipe pile will be partially filled with water and it may not be practical to dewater the pipe prior to concreting. Tremie concreting will be required for concreting these pipe piles.

A NSSP addressing the above issues is included in Appendix F.

9.4 Drilled Shafts/Caissons

Caissons socketed into bedrock may be employed to support the structural loads this site. The bedrock surface is anticipated at the depths and elevations given in Table 9.1.

The vertical geotechnical resistance computed for 0.9 m, 1.2 m and 1.5 m diameter sockets with lengths of 1.5 and 3.0 m below the bedrock surface are presented in Table 9.4.

Table 9.4 – Recommended Resistance Values for Caisson Design

Caisson Diameter (m)	Socket Length below Bedrock Surface (m)	Factored Geotechnical Resistance at ULS (kN)
0.91	1.5	3,000
	3.0	6,000
1.2	1.5	3,500
	3.0	7,500
1.5	1.5	4,500
	3.0	9,500

The vertical geotechnical resistances were computed using the method outlined in the Canadian Foundation Engineering Manual, 4th Edition, Section 18.6.4. The resistance values are based on shaft resistance within the bedrock socket only; end-bearing resistance has been ignored in anticipation of difficulties cleaning and inspecting the caisson base below the water level.

The SLS condition will not govern for caissons socketed into the rock.

The selection of a suitable socket depth will be governed by axial loads, lateral load and maximum shear and moment demand on each caisson. The depth of rock socket should not be less than 1.5 m and the axial load, shear and moment demands may require a deeper depth of rock socket.

The assessment of rock socket depth allows for the presence of some weathered and broken up rock just below the bedrock surface. Since the elevation of the bedrock surface is variable across the site and there is evidence of cobbles and boulders above the bedrock, it is critical to determine in the field during inspection of caisson installation that the entire depth of socket is formed in bedrock and not partly in cobbles and boulders and partly in bedrock. This issue is addressed in an NSSP included in Appendix F.

9.4.1 Caisson Socket Lateral Resistance

The ultimate passive force that can be mobilized by the embedded portion of a caisson socket within rock is constant with depth and is given in Section 9.3.1 of this report.

9.4.2 Caisson Socket Installation

Caisson installation must be in accordance with OPSS 903.

Caisson installation will require excavation through sand and gravel fill with cobbles, boulders and rock fill, soft to stiff silty clay and cohesionless sand and silt till below the groundwater table. Sockets must be constructed in the underlying bedrock. The installation of caissons at this site must consider the following issues:

- The installation method must prevent collapse of caisson sidewalls and washing of cohesionless soils into the rock socket. Provision of a permanent (left-in-place) steel liner is recommended for this purpose.
- The caisson installation equipment must be able to advance through cobbles, boulders and rock fill within the existing embankment fill.
- Sealing the liner into the bedrock may be difficult with the overlying fill and may result in washing of cohesionless soils into the rock socket.
- The strength and hardness of the bedrock at this site must be considered when selecting equipment to excavate the rock socket.

Selection of the methods and equipment employed to address the above issues is the responsibility of the Contractor. The contract documents must contain a statement to alert bidders of the above facts. The wording for an NSSP addressing this issue is included in Appendix F. This NSSP must be included in the tender documents.

9.5 Downdrag

Downdrag forces will develop along the length of piles embedded in the embankment fill and very soft to firm silty clay layer due to consolidation of the clay under the weight of additional fill placed in the embankments.

For design purposes, the following unfactored downdrag loads are recommended to evaluate the impact of downdrag on the abutment piles:

Table 9.5– Estimated Downdrag Loads

Pile Section	Pile Size / Outer Diameter (mm)	Unfactored Downdrag Load (kN)
H-Pile	310 x 110	410
Pipe Pile	324	335
	405	420
	508	525
	610	630

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. 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.

As indicated in Clause C6.8.4 of the Commentary, the factored dead and downdrag load should not exceed the factored structural resistance of a pile.

9.6 Recommended Foundation

From a geotechnical perspective and based on the subsurface conditions, the preferred foundation type for supporting the replacement bridge at this site is drilled-in pipe piles socketed into bedrock.

Driven steel H-piles were installed previously at this site to supplement the timber pile foundations, and could be considered for support of the replacement bridge. However, driven piles are not the recommended foundation option in consideration of the following:

- Driven H-piles could be deflected by or encounter refusal on boulders or rock fill in the existing embankment fill above the required tip depth at the bedrock surface;
- Driving of piles is more likely to result in disturbance of the existing bridge foundations, which is of concern during the first stage of bridge replacement when traffic will be carried on a single lane of the existing bridge;
- Vibration from pile driving has the potential to reactivate spreading movement of the embankment fill over the soft clay.

The use of caissons is not recommended in view of the potential issues outlined in Section 9.4.2.

9.7 Depth of Frost Protection

The design depth of frost penetration at this site is 2.3 m. The base of buried pile caps (if employed) must be provided with a minimum of 2.3 m of earth cover as protection against frost action.

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.

Driving of the sheet pile through the existing approach fill may encounter cobbles, boulders or zones of rock fill. The Contract Documents should contain a NSSP alerting the Bidders to the possibility of some sheet piles meeting refusal on the cobbles, boulders or rock fill, and the need to remove or penetrate these obstructions. Suggested text for the NSSP is included in Appendix F. Any visible obstructions such as boulders and rock protection along the sides of the embankment should be removed prior to driving the sheet piles.

Sheet piles should be provided with sheet pile tip protector to minimize any tip damage.

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 pile 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 1010 as amended by Special Provision 110S13. Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with OPSS 501.

Earth pressures acting on the sheet pile walls or a conventional abutment wall may be assumed to be triangularly distributed and governed by the characteristics of the backfill and existing fill. 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 10.1)

γ = unit weight of retained soil (see Table 10.1)

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

q = value of any surcharge (kPa)

Earth pressure coefficients for backfill to the sheet pile wall or abutment wall are dependent on the material used as backfill. Typical values are shown in Table 10.1.

Table 10.1 – Earth Pressure Coefficient (K)

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 Granular B Type III, or Existing Sand and Gravel Fill $\phi = 32^\circ$, $\gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.38*	0.31	0.46*
At rest (Restrained Wall)	0.43	-	0.47	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-

* 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 10.1 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 and Type III or 1.7 m for Granular A or Granular B Type II.

11 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 0.02g

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 11.1 may be used:

Table 11.1 – 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, Granular B Type III, or Existing sand and gravel fill $\phi = 32^\circ$, $\gamma = 21.2 \text{ kN/m}^3$
Active (K_{AE})*	0.28	0.32
Passive (K_{PE})	3.7	3.2
At Rest (K_{OE})**	0.45	0.50

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

** After Woods

The potential for liquefaction of the foundations soils was assessed using the Seed and Idriss (1971) method for cohesionless soils. Using the method, it is estimated that under the existing conditions the foundation soils at the abutments are not prone to liquefaction.

12 APPROACH EMBANKMENTS

The existing causeway and approach embankments are approximately 4.0 m high and were constructed by placement of granular fill over a deep deposit of soft silty clay on the lake bottom. The silty clay was incompletely displaced by the fill and as a consequence, a variable thickness of soft silty clay remains under the embankment fill and the fill thickness decreases rapidly towards the toe of the embankment slope. The fill thickness encountered below the roadway platform in boreholes drilled during the current investigation ranged from 7.1 to 15.0 m.

The existing Highway 11 grade will not be raised during bridge replacement. However, current plans call for a decreased bridge length and widening of the roadway platform, as follows:

- The new abutments will be located approximately 3.3 m in front of the existing abutments. A sheet pile enclosure will be installed behind the abutment piles, and additional fill with a maximum thickness of about 1.8 m will be placed within the wedge-shaped area enclosed by the sheet piles.
- The roadway platform will be widened by construction of gravity block retaining walls near the top of the slope extending for distances of about 11.6 m from the northwest, southwest and southeast ends of the sheet pile wall enclosures (the sheet pile walls themselves extend about 6.2 m behind the abutments). The maximum exposed wall height will be approximately 1.2 m and the platform widening behind the wall will be about 2.4 m.

Comments regarding the stability of the modified embankment slopes and settlement of the foundations soils under the loading of the new fill are provided in the following sections.

12.1 Slope Stability

Analyses were conducted to evaluate the stability of the forward slopes at the proposed east and west abutments constructed with backfilled sheet pile wall enclosures. As well, assessment of the side slopes of the proposed widened platform with gravity walls at the slope crest was carried out for the west approach where cross-sectional borehole information was available. 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 8 in Appendix G. The computed factors of safety are summarized in Table 12.1.

Table 12.1 – Results of Global Stability Analyses

Location	Condition	Factor of Safety	Figure (Appendix G)
East Abutment Forward Slope	Short term - undrained	1.61	1
	Long term - drained	1.53	2
West Abutment Forward Slope	Short term - undrained	1.56	3
	Long term - drained	1.55	4
West Approach, South Side	Short term - undrained	1.59	5
	Long term - drained	1.52	6
West Approach, North Side	Short term - undrained	1.41	7
	Long term - drained	1.41	8

The computed factors of safety generally exceed 1.5 and are considered acceptable. The failure arc representing the minimum factor of safety or 1.4 for the north side of the west approach (Figure 8) is located fully within the cohesionless granular fill and therefore this value is also considered adequate.

The forward slope analyses assume that the sheet piles will be extended down to at least Elev. 337.0. The length of sheet pile must also be designed to resist lateral earth pressures and maintain lateral stability, and it is recommended that the sheet piles be extended at least to the lake bed level.

12.2 Settlement

The placement of approximately 1.5 m to 1.8 m of new fill behind the sheet pile abutments and the new fill for widening of the roadway platform will induce immediate (elastic) settlement in the existing non-cohesive fill and time dependent (consolidation) settlement in the underlying very soft to firm silty clay.

The total immediate and consolidation settlements under the new fill loading were assessed using elastic methods and one-dimensional consolidation theory. Based on these analyses, the immediate and consolidation settlements at the abutments and platform widening are expected to be in the order of 10 mm and 30 mm, respectively. The elastic component will be immediate and essentially complete prior to approach slab construction. The consolidation component is expected to be completed within 3 to 6 months following fill placement.

Inspection of the roadway surface and padding of the asphalt to re-establish grades as necessary should be implemented after construction.

To reduce the post-construction settlement, use of lightweight material could be considered for new fill applications. However, this option is unlikely to be practical for the limited quantities and geometry of the proposed fill zones.

13 EROSION PROTECTION

Boulders and rock slabs currently exist on the slopes of the approach embankments. It is recommended that the rock protection be maintained and if necessary enhanced to provide protection against erosion and scour. Additional thickness of rock protection should be kept to a minimum to minimize future settlement due to consolidation of the underlying silty clay soils.

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

14 EXCAVATION AND GROUNDWATER CONTROL

If any earth excavation is required, it must be carried out in accordance with OPSS 902 and the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the native soils within the probable depth of excavation at this site may be classed as Type 3 soils above the water table and Type 4 soils below the water table.

Based on the preliminary GA for the bridge structure, new foundation construction is not expected to require excavation below the lake and groundwater level. It is recommended that excavation for removal of existing foundations be maintained above the lake water level. Any excavation below the lake level without prior dewatering is not recommended since the inflow of water will make it difficult to maintain a dry, sound base on which to work.

Bridge replacement will be carried out in stages to maintain one traffic lane operational at all times. Roadway protection will be required to facilitate staging. Roadway protection should be provided in accordance with OPSS 539 and designed for Performance Level 2.

The design of any dewatering system and any road protection that may be required is 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:

1. Alignment of the new pile foundations and sheet pile abutments should be carefully selected to avoid the existing pile foundations.
2. The embankment fill contains cobbles, boulders and rock fill which may affect installation of sheet piles, H-piles, drilled-in pipe piles or caissons. The Contractor shall be prepared to remove, drill through and/or penetrate these obstructions.
3. Caisson sockets or drilled-in pipe piles must be constructed using methodology capable of excavating the bedrock to the specified socket dimensions without disturbing or fracturing the bedrock forming the sidewalls and base of the socket. Further, it must be confirmed that the entire depth of socket is formed in bedrock and not partly in cobbles and boulders above the bedrock.

4. The existing causeway and approach embankments were constructed by end-dumping of granular fill over a deep deposit of soft silty clay on the lake bottom. Instability and failure of the clay was experienced during fill placement. The silty clay was incompletely displaced by the fill and as a consequence, a variable thickness of soft silty clay remains under the embankment fill and the fill thickness decreases rapidly towards the toe of the embankment slope.

Temporary construction works must not impact on the stability of the existing embankment slopes. Placement of additional fill on the embankment (i.e, as a pad for support of piling rigs) may lead to settlement, lateral spreading or failure of the embankment, and must be avoided. The capability of the existing embankment to support heavy construction equipment must be assessed by the Contractor, and this assessment may impact the type of equipment suitable for the project. Construction equipment must not travel near the toe of slope where the fill layer is thin.

16 CLOSURE

Engineering analysis and preparation of the report were carried out by Ms. R. Palomeque Reyna, 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.

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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


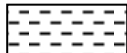



C_{pen} Shear Strength Determination by Pocket Penetrometer

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

UNIFIED SOILS CLASSIFICATION

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

EXPLANATION OF ROCK LOGGING TERMS

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

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

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

RECORD OF BOREHOLE No SMB-01

1 OF 2

METRIC

W.P. 6048-08-00 LOCATION Seven Mile Bridge N 5 391 874.2 E 284 020.2 ORIGINATED BY JM
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing COMPILED BY AN
 DATUM Geodetic DATE 2011.10.15 - 2011.10.15 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT			NATURAL MOISTURE CONTENT			LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					w P			w			w L				GR	SA	SI	CL																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
												○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE	WATER CONTENT (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															
341.0								20	40	60	80	100																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																			

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+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SMB-01

2 OF 2

METRIC

W.P. 6048-08-00 LOCATION Seven Mile Bridge N 5 391 874.2 E 284 020.2 ORIGINATED BY JM
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing COMPILED BY AN
 DATUM Geodetic DATE 2011.10.15 - 2011.10.15 CHECKED BY RPR

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

METRIC

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+³, ×³: Numbers refer to Sensitivity

ONTMT4S 1197.GPJ 2012TEMPLATE(MTO).GDT 8/30/13

RECORD OF BOREHOLE No SMB-02

2 OF 2

METRIC

W.P. 6048-08-00 LOCATION Seven Mile Bridge N 5 391 866.8 E 284 018.4 ORIGINATED BY JM
 HWY 11 BOREHOLE TYPE Hollow Stem Auger/NW Casing/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2011.10.15 - 2011.10.16 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					PLASTIC LIMIT W P NATURAL MOISTURE CONTENT W LIQUID LIMIT W L
	Continued From Previous Page							20 40 60 80 100					
330.0	SAND and GRAVEL , trace silt Very Dense Grey Wet (FILL) Cored through boulder (350mm) at 10.3m		9	SS	88/ 0.200		331					FI	RUN #1 TCR=100% SCR=86% RQD=100% UCS=197MPa (Average) RUN #2 TCR=100% SCR=100% RQD=100% UCS=197MPa (Average) RUN #3 TCR=87% SCR=87% RQD=87% UCS=166MPa (Average)
11.0	BEDROCK monzodiorite, grey, occasional vertical breaks		1	RUN			330					2	
	Sub-vertical fractures at 12.1m and 12.6m		2	RUN			329					1	
	Horizontal fracture at 12.5m and 12.9m						328					1	
			3	RUN								0	
												0	
												0	
326.9	END OF BOREHOLE AT 14.1m. BOREHOLE OPEN TO 13.9m AND WATER LEVEL AT 4.1m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.												
14.1	WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Oct. 16/11 4.2 336.8 Oct. 20/11 3.9 337.1 Oct. 27/11 4.3 336.7												

RECORD OF BOREHOLE No SMB-03

1 OF 2

METRIC

W.P. 6048-08-00 LOCATION Seven Mile Bridge N 5 391 866.3 E 284 024.9 ORIGINATED BY JM
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing COMPILED BY AN
 DATUM Geodetic DATE 2011.10.20 - 2011.10.20 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa			WATER CONTENT (%)					
								○ UNCONFINED + FIELD VANE	● QUICK TRIAXIAL × LAB VANE							
										20 40 60 80 100	20 40 60					
341.0																
0.0																
0.1	ASPHALT:(100mm)															
	SAND and GRAVEL, trace silt, occasional cobbles Dense to Very Dense Brown Moist (FILL) Cobbles from 1.1m to 1.4m		1	SS	39											
			2	SS	50/ 0.075										46 46 8 (SI+CL)	
			3	SS	37											
	Compact		4	SS	22											
			5	SS	16											
			6	SS	22										47 48 5 (SI+CL)	
	Loose Wet		7	SS	4											
			8	SS	13											
	Compact		9	SS	31											
	Dense														66 28 6 (SI+CL)	
331.6																
9.4	Silty CLAY, trace sand Very Stiff Grey															

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SMB-03

2 OF 2

METRIC

W.P. 6048-08-00 LOCATION Seven Mile Bridge N 5 391 866.3 E 284 024.9 ORIGINATED BY JM
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing COMPILED BY AN
 DATUM Geodetic DATE 2011.10.20 - 2011.10.20 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
	Continued From Previous Page													
	Silty CLAY , trace sand Very Stiff Grey		10	SS	24		331							0 3 47 50
329.8							330							
11.2	END OF BOREHOLE AT 11.2m UPON AUGER REFUSAL ON PROBABLE BEDROCK. BOREHOLE OPEN TO 9.4m AND WATER LEVEL AT 3.9m. BOREHOLE BACKFILLED WITH BENTONITE FROM 11.2m TO 1.5m, AUGER CUTTINGS FROM 1.5m TO 0.15m THEN ASPHALT TO SURFACE.													

RECORD OF BOREHOLE No SMB-04

1 OF 2

METRIC


W.P. 6048-08-00 LOCATION Seven Mile Bridge N 5 391 820.4 E 284 010.1 ORIGINATED BY JM
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing COMPILED BY AN
 DATUM Geodetic DATE 2011.10.16 - 2011.10.17 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
341.0																	
0.0	ASPHALT:(90mm)																
0.1	SAND and GRAVEL trace to some silt, occasional cobbles Very Dense to Compact Brown Moist (FILL)		1	SS	70												
			2	SS	19												
			3	SS	13												
			4	SS	12											43 44 13 (SI+CL)	
			5	SS	10												
			6	SS	3												
	Very Loose		7	SS	17											88 11 1 (SI+CL)	
	Compact Grey Wet		8	SS	11												
			9	SS	13												

Continued Next Page

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

METRIC

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

SOIL PROFILE				SAMPLES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER				TYPE	"N" VALUES	SHEAR STRENGTH kPa	WATER CONTENT (%)		
									○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE			
	Continued From Previous Page											
	SAND and GRAVEL , trace to some silt, occasional cobbles Compact Grey Wet (FILL)		10	SS	13							
			11	SS	25							
	Very Dense		12	SS	53							
326.0			13	SS	67							60 30 10 (SI+CL)
15.0	SAND and SILT , trace to some gravel, trace clay Very Dense Grey Moist to Wet (TILL)											
324.9			14	SS	100							
16.1	END OF BOREHOLE AT 16.1m UPON AUGER REFUSAL ON PROBABLE BEDROCK. BOREHOLE OPEN TO 16.1m AND WATER LEVEL AT 4.5m. BOREHOLE BACKFILLED WITH BENTONITE FROM 16.1m TO 1.5m, AUGER CUTTINGS FROM 1.5m TO 0.15m THEN ASPHALT TO SURFACE.				0.075							

ONTMT4S 1197.GPJ 2012TEMPLATE(MTO).GDT 8/30/13

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No SMB-05

1 OF 3

METRIC

W.P. 6048-08-00 LOCATION Seven Mile Bridge N 5 391 819.2 E 284 017.1 ORIGINATED BY JM
 HWY 11 BOREHOLE TYPE Hollow Stem Auger/NW Casing/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2011.10.19 - 2011.10.19 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				WATER CONTENT (%) W _p W W _L				
341.0																
0.0	ASPHALT:(100mm)															
0.1	SAND and GRAVEL, trace silt, occasional cobbles Very Dense to Compact Brown Moist (FILL)		1	SS	52											
			2	SS	15											
			3	SS	28											
	Loose to Compact		4	SS	9											
			5	SS	15										31 58 11 (SI+CL)	
			6	SS	12											
			7	SS	18											
	Grey Wet															
			8	SS	17										56 39 5 (SI+CL)	
			9	SS	12											

Continued Next Page

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

RECORD OF BOREHOLE No SMB-05

2 OF 3

METRIC

W.P. 6048-08-00 LOCATION Seven Mile Bridge N 5 391 819.2 E 284 017.1 ORIGINATED BY JM
 HWY 11 BOREHOLE TYPE Hollow Stem Auger/NW Casing/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2011.10.19 - 2011.10.19 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE										
								● QUICK TRIAXIAL × LAB VANE										
	Continued From Previous Page							20 40 60 80 100										
	SAND and GRAVEL , trace silt, occasional cobbles and boulders Compact Gray Wet (FILL)		10	SS	29		331											
							330											
	Boulder (250mm) at 12.5m		11	SS	19		329											
							328											
	Dense		12	SS	49		327											
327.0							326											
14.0	SAND and SILT , trace to some gravel, trace clay Compact Grey Wet (TILL)		13	SS	18		325											
							324											
325.5							323											
15.5	SAND , some gravel, trace silt Compact Grey Wet (TILL)		14	SS	28		322											
							321											
322.9			15	SS	26		320											
18.1	BEDROCK monzodiorite, grey, occasional vertical breaks		1	RUN			319											
	Horizontal fractures at 18.4m and 18.5m						318											
	Sub-vertical fracture at 20.2m		2	RUN			317											

Continued Next Page

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

RECORD OF BOREHOLE No SMB-05

3 OF 3

METRIC

W.P. 6048-08-00 LOCATION Seven Mile Bridge N 5 391 819.2 E 284 017.1 ORIGINATED BY JM
 HWY 11 BOREHOLE TYPE Hollow Stem Auger/NW Casing/NQ Coring COMPILED BY AN
 DATUM Geodetic DATE 2011.10.19 - 2011.10.19 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					WATER CONTENT (%)				
	Continued From Previous Page																
	BEDROCK monzodiorite, grey, occasional vertical breaks						321									0	RUN #3 TCR=100% SCR=100% RQD=100% UCS=151MPa (Average)
	Sub-vertical fracture at 20.6m, 20.8m and 21.6m															2	
			3	RUN												1	
																1	
																0	
319.1							320									0	
21.9	END OF BOREHOLE AT 21.9m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen.															0	
	WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) Oct. 19/11 4.5 336.5 Oct. 20/11 4.0 337.0 Oct. 27/11 4.0 337.0															0	

RECORD OF BOREHOLE No SMB-06

1 OF 2

METRIC

W.P. 6048-08-00 LOCATION Seven Mile Bridge N 5 391 812.5 E 284 014.9 ORIGINATED BY JM
 HWY 11 BOREHOLE TYPE NW Casing COMPILED BY AN
 DATUM Geodetic DATE 2011.10.18 - 2011.10.18 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
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						
341.1								20	40	60	80	100						
0.0	ASPHALT:(100mm)							20	40	60	80	100						
0.1	SAND and GRAVEL, trace silt, occasional cobbles and boulders Very Dense to Compact Brown Moist (FILL)		1	SS	82	▽	341											
			2	SS	34		340											
			3	SS	13		339											
			4	SS	10		338											
			5	SS	17		337											
	Boulder (280mm) from 4.3m to 4.6m						336											
							335											
	Boulder (1300mm) at 5.0m						334											
							333											
			8	SS	17		332											
	Loose to Compact Grey Wet																	

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No SMB-06

2 OF 2

METRIC

W.P. 6048-08-00 LOCATION Seven Mile Bridge N 5 391 812.5 E 284 014.9 ORIGINATED BY JM
 HWY 11 BOREHOLE TYPE NW Casing COMPILED BY AN
 DATUM Geodetic DATE 2011.10.18 - 2011.10.18 CHECKED BY RPR

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
	Continued From Previous Page																
330.6	SAND and GRAVEL , occasional cobbles Compact Grey Wet (FILL) END OF BOREHOLE AT 10.5m. BOREHOLE OPEN TO 10.5m AND WATER LEVEL AT 3.8m. BOREHOLE BACKFILLED WITH BENTONITE FROM 10.5m TO 1.5m, AUGER CUTTINGS FROM 1.5m TO 0.15m THEN ASPHALT TO SURFACE.		9	SS	28		331										
10.5																	

Appendix B

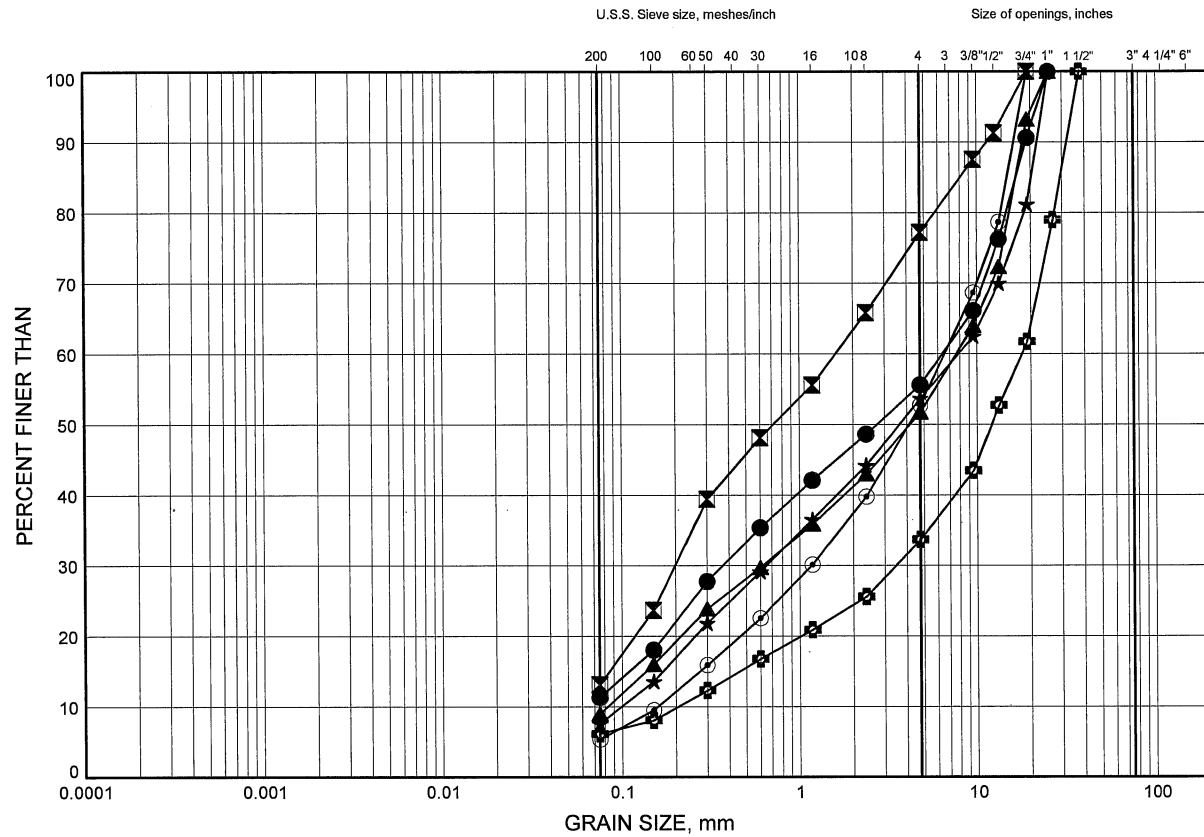
Laboratory Test Results

NWR 32 Rehabs

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)
●	SMB-01	1.83	339.17
⊠	SMB-01	6.40	334.60
▲	SMB-02	8.69	332.31
★	SMB-03	1.07	339.93
⊙	SMB-03	4.11	336.89
⊕	SMB-03	8.69	332.31

Date May 2013
W.P. 6048-08-00



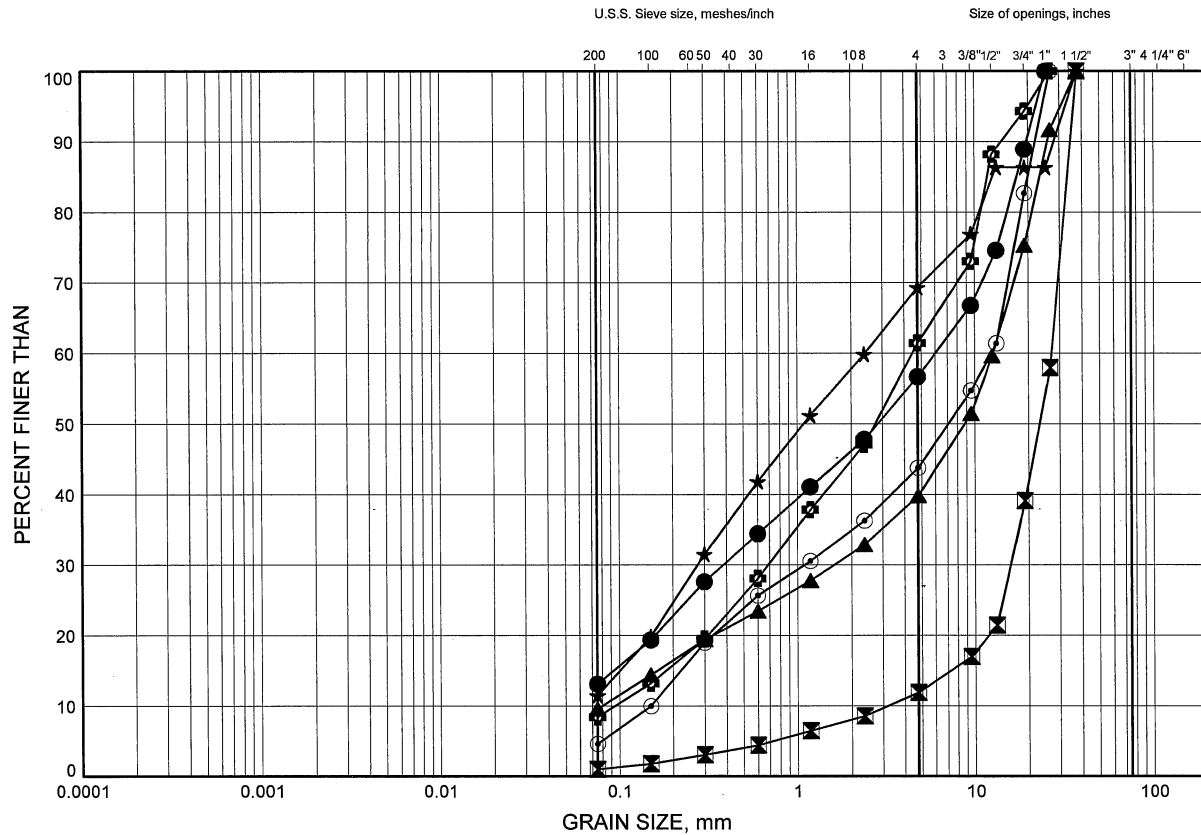
Prep'd AN
Chkd. RPR

NWR 32 Rehabs

GRAIN SIZE DISTRIBUTION

FIGURE B2

SAND & GRAVEL FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SMB-04	2.59	338.41
⊠	SMB-04	5.64	335.36
▲	SMB-04	14.71	326.29
★	SMB-05	3.35	337.65
⊙	SMB-05	7.16	333.84
⊕	SMB-05	13.26	327.74

Date May 2013
W.P. 6048-08-00



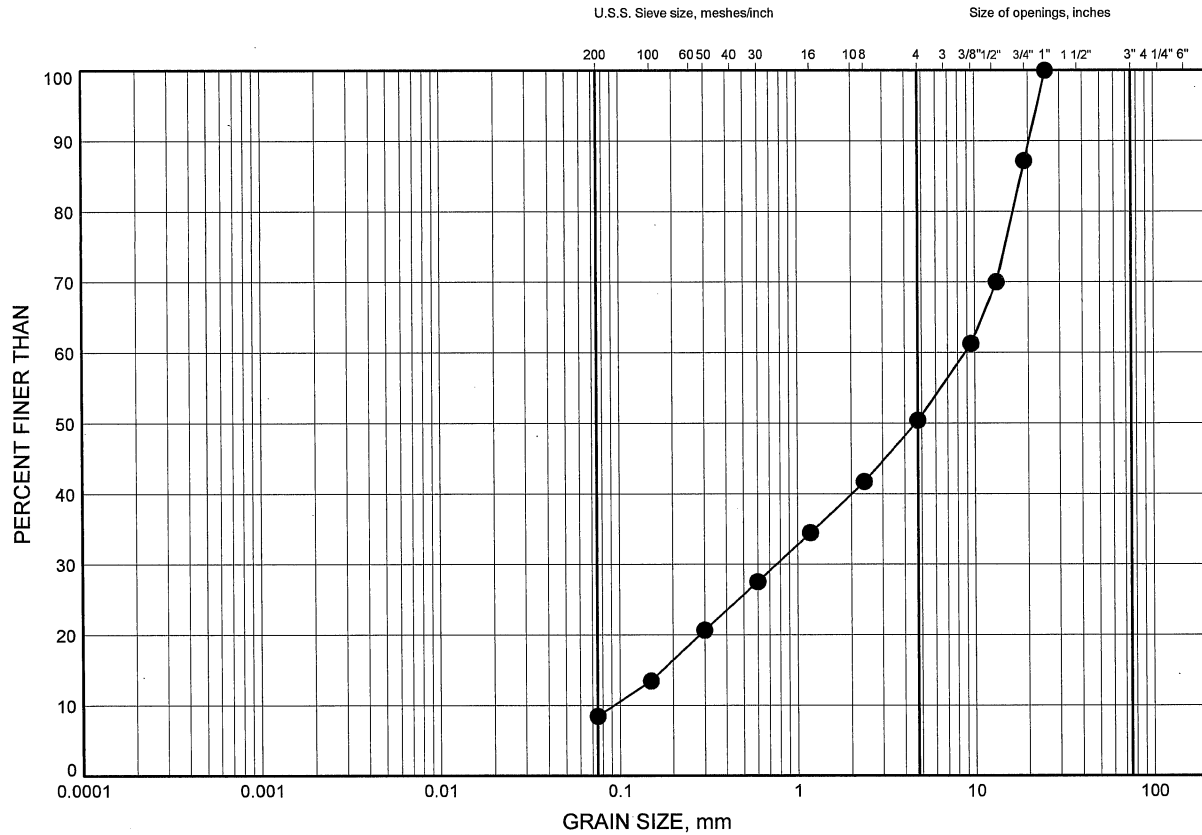
Prep'd AN
Chkd. RPR

NWR 32 Rehabs

GRAIN SIZE DISTRIBUTION

FIGURE B3

SAND & GRAVEL FILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SMB-06	1.83	339.27

GRAIN SIZE DISTRIBUTION - THURBER 1197.GPJ 5/2/13

Date May 2013
W.P. 6048-08-00

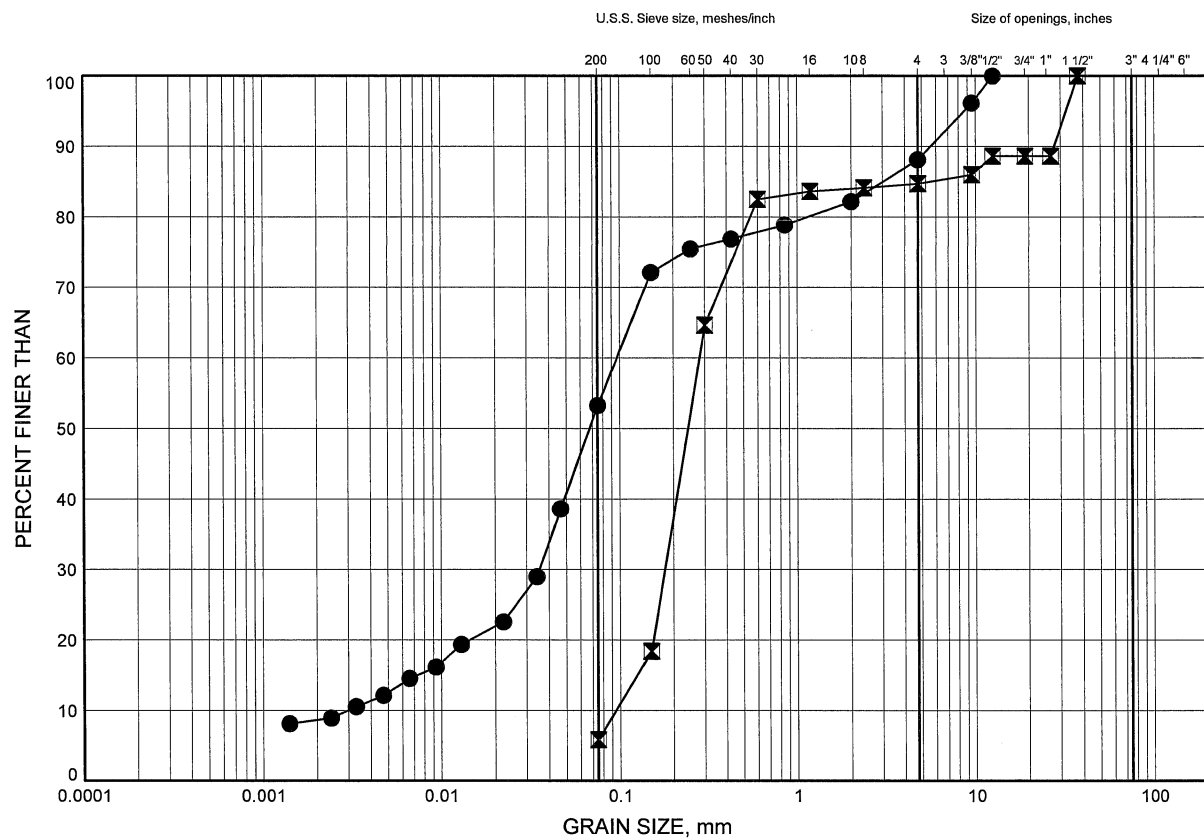


Prep'd AN
Chkd. RPR

NWR 32 Rehabs GRAIN SIZE DISTRIBUTION

FIGURE B5

SAND to SAND & SILT TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SMB-01	10.21	330.79
⊠	SMB-05	16.31	324.69

Date August 2013
W.P. 6048-08-00

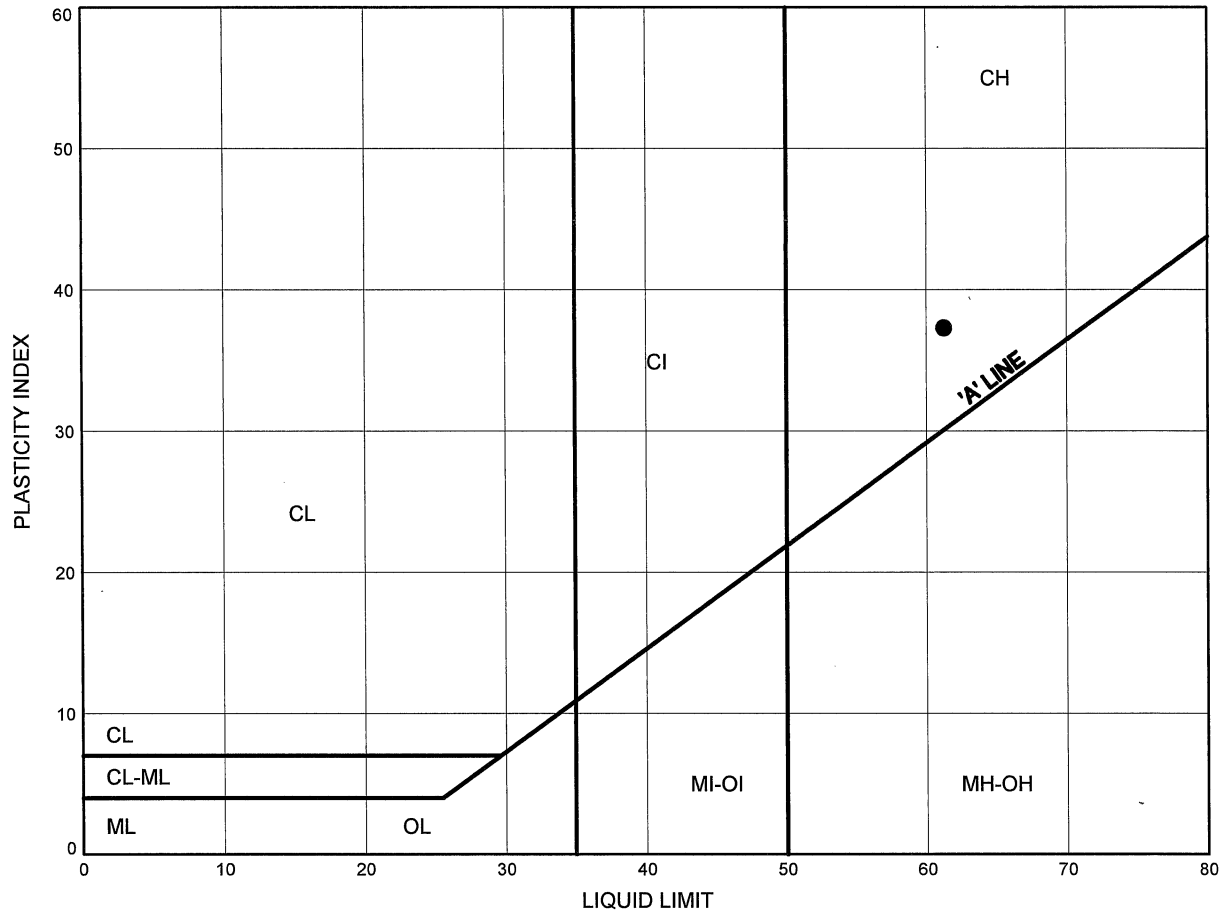


Prep'd AN
Chkd. RPR

NWR 32 Rehabs
ATTERBERG LIMITS TEST RESULTS

FIGURE B6

SILTY CLAY



LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	SMB-01	8.69	332.31

THURBALT 1197.GPJ 5/2/13

Date May 2013
 W.P. 6048-08-00



Prep'd AN
 Chkd. RPR

Appendix C

Record of Borehole Sheets and Soil Profile from Previous Investigation

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

APPENDIX I

CONTRACT 57034 BORING # 1 DATUM GEODETIC CASING BX
 BORING DATE FEB. 22, 1960 REPORT DATE MARCH 29, 1960 COMPILED BY M.W. & J.A. CHECKED BY J.F.
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

SAMPLE CONDITION

☐ DISTURBED
☐ FAIR
☐ GOOD
☒ LOST

SAMPLE TYPES

A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE
 F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 QC - TRIAXIAL CONSOLIDATED QUICK
 Q - TRIAXIAL QUICK
 S - TRIAXIAL SLOW
 1 - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL

SOIL PROFILE				SAMPLES			
EVN. EPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT ELEVATION SCALE	WATER CONTENT W _c	OTHER TESTS	CONDITION	PENETRATION RESISTANCE BLOWS/FT.
317.8		GROUND LEVEL	1120				
1.0		ROCK FILL			CORE RECOVERY 0%	BX CGKE	1
			1110				
		LOOSE TO COMPACT BROWN GRANULAR FILL					
			1100				
			1090				
1086.8		GREY SANDY TILL					
31.0		SOUND GREY SYENITE GNEISS BEDROCK					
53.0			1080				
079.0		END OF HOLE	1070		RC RECOVERY 96%	BX RC	7
58.8							

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

APPENDIX I

CONTRACT S 7034 BORING # 2 DATUM GEODETIC CASING BX
 BORING DATE FEB. 11, 1960 REPORT DATE MARCH 25, 1960 COMPILED BY M.W. EJA CHECKED BY JH
 SAMPLER HAMMER WT 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION

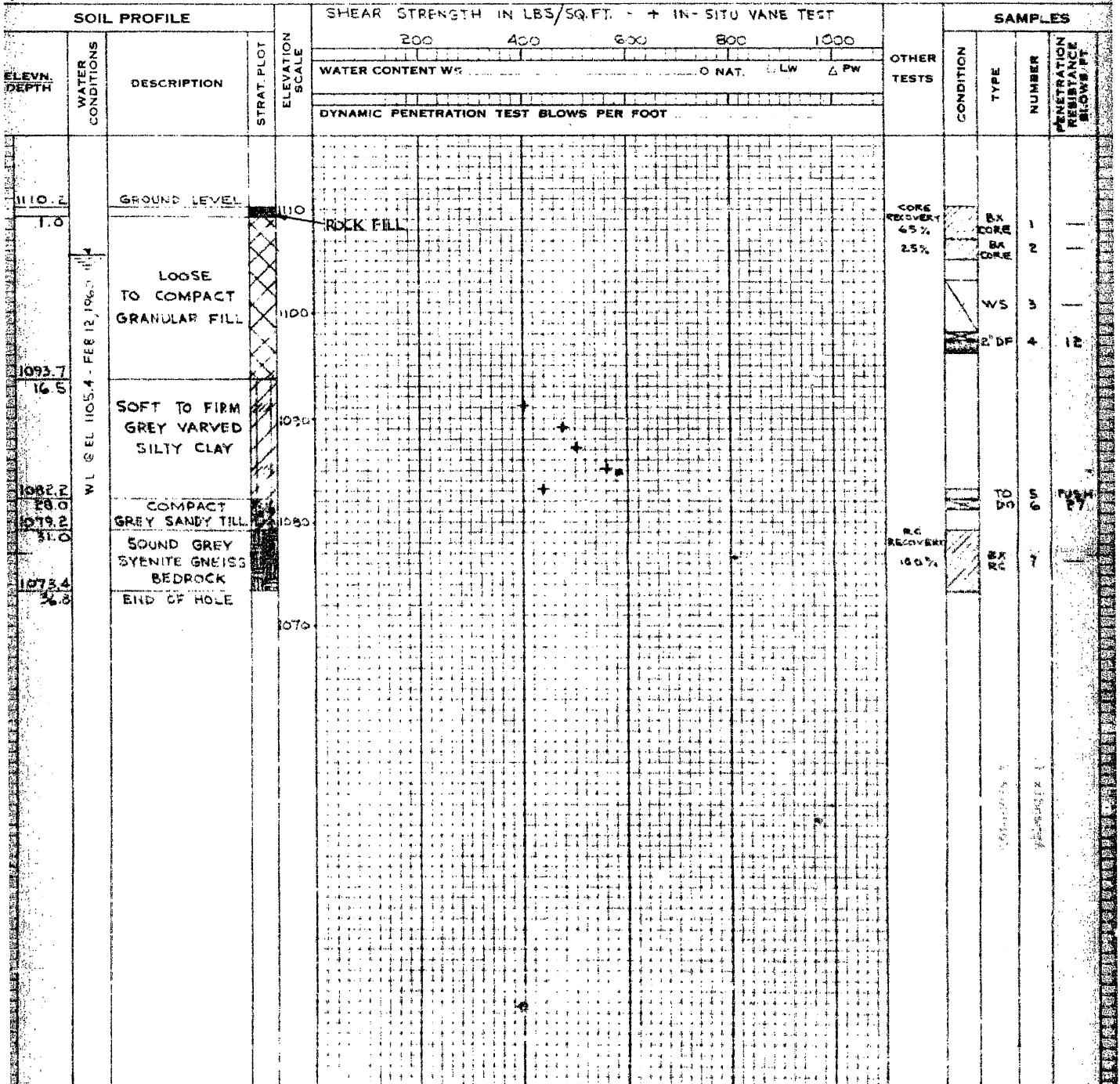
☐ DISTURBED
☐ FAIR
☐ GOOD
☐ LOST

SAMPLE TYPES

A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE
 F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 QC - TRIAXIAL CONSOLIDATED QUICK
 Q - TRIAXIAL QUICK
 S - TRIAXIAL SLOW
 1 - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

APPENDIX I

CONTRACT 57034 BORING # 3 DATUM GEODETIC CASING BX
 BORING DATE FEB. 15, 1960 REPORT DATE MARCH 25, 1960 COMPILED BY M.W. FJA CHECKED BY JLS
 SAMPLER HAMMER WT. 14.0 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION

☐ DISTURBED
☐ FAIR
☐ GOOD
☐ LOSE

SAMPLE TYPES

A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE
 F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 Qc - TRIAXIAL CONSOLIDATED QUICK
 Q - TRIAXIAL QUICK
 S - TRIAXIAL SLOW
 1 - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL

SOIL PROFILE				OTHER TESTS		SAMPLES			
ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE	OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
1116.9	0.0	ROAD SURFACE		1120					
1113.9	3.0	ROCK FILL		1110					
		LOOSE TO COMPACT BROWN GRANULAR FILL		1100					
				1090					
				1080					
		GREY SANDY TILL		1070					
		END OF HOLE PROBABLY BOULDER OR BEDROCK							

WATER CONTENT W _s		O NAT.		Δ LW		Δ Pw	

DYNAMIC PENETRATION TEST BLOWS PER FOOT	

CORE RECOVERY	
50%	

SAMPLES	
BA CORE	1
WS	2
WS	3
WS	4
WS	5
WS	6
WS	7
WS	8
WS	9
WS	10

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

APPENDIX I

CONTRACT 57034 BORING # 4 DATUM GEODETIC CASING BX
 BORING DATE FEB. 17, 1960 REPORT DATE MARCH 25, 1960 COMPILED BY M. W. J. A. CHECKED BY J. P.
 SAMPLER HAMMER WT. 140 LBS DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

SAMPLE CONDITION

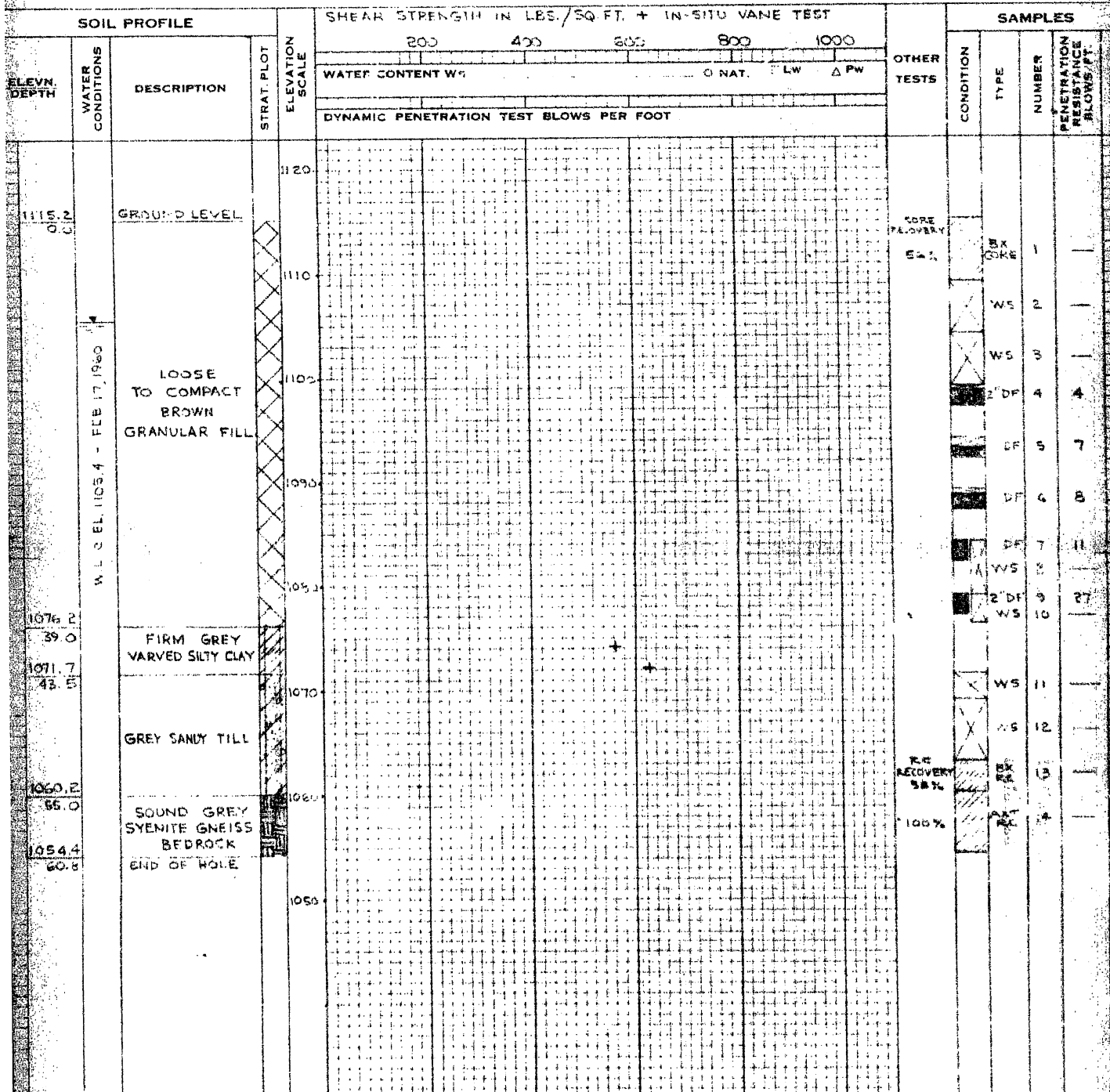
☐ DISTURBED
☐ FAIR
☐ GOOD
☐ LOST

SAMPLE TYPES

A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE
 F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

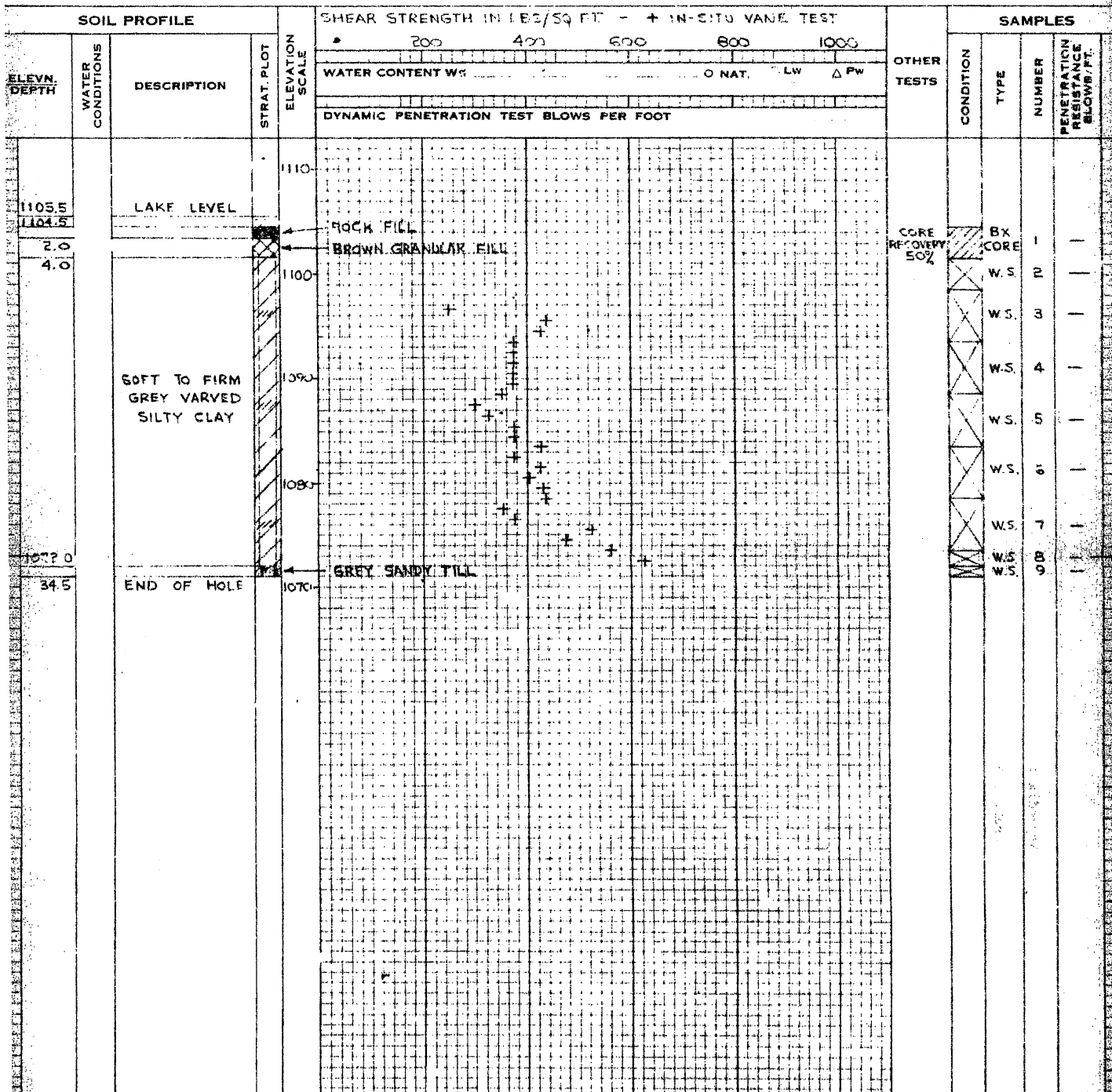
V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 QC - TRIAXIAL CONSOLIDATED QUICK
 Q - TRIAXIAL QUICK
 S - TRIAXIAL BLOW
 γ - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL



APPENDIX I

ABBREVIATIONS

γ - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL



OFFICE REPORT ON SOIL EXPLORATION

SAMPLE CONDITION

SAMPLE TYPES

ABBREVIATIONS

DISTURBED
FAIR
GOOD
LOST

A.S. - AUGER SAMPLE
S.T. - SLOTTED TUBE
W.S. - WASHED SAMPLE
D.O. - DRIVE-OPEN
D.F. - DRIVE-FOOT VALVE
C.S. - CHUNK SAMPLE

F.S. - FOIL SAMPLE
S.O. - SLEEVE-OPEN
S.F. - SLEEVE-FOOT VALVE
T.O. - THIN WALLED OPEN
R.C. - ROCK CORE

V - IN-SITU VANE TEST
M - MECHANICAL ANALYSIS
U - UNCONFINED COMPRESSION
QC - TRIAXIAL CONSOLIDATED QUICK
Q - TRIAXIAL QUICK
S - TRIAXIAL SLOW

7 - WET UNIT WEIGHT
K - PERMEABILITY
C - CONSOLIDATION

WL - WATER LEVEL IN CASING
WT - WATER TABLE IN SOIL

[illegible]

GEOCON

OFFICE REPORT ON SOIL EXPLORATION

APPEX-1

CONTRACT S7034 BORING # 7 DATUM GEODETIC CASING A.X.
 BORING DATE FEB 25, 1960 REPORT DATE MARCH 29, 1960 COMPILED BY J.A. CHECKED BY J.H.
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN - LBS. ENERGY)

SAMPLE CONDITION

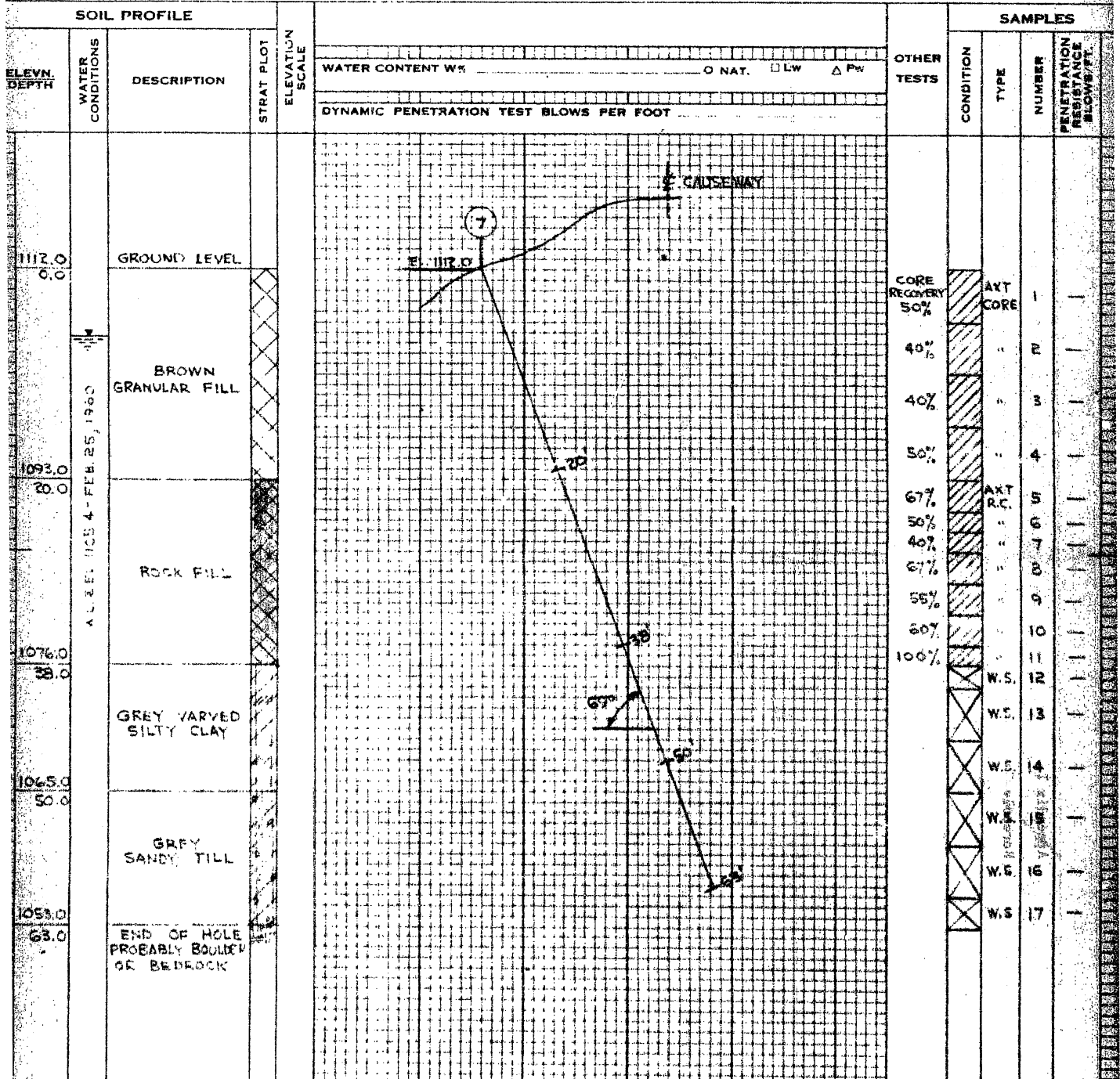
☒ DISTURBED
☐ FAIR
☐ GOOD
☐ LOST

SAMPLE TYPES

A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE
 F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 QC - TRIAXIAL CONSOLIDATED QUICK
 Q - TRIAXIAL QUICK
 S - TRIAXIAL SLOW
 T - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

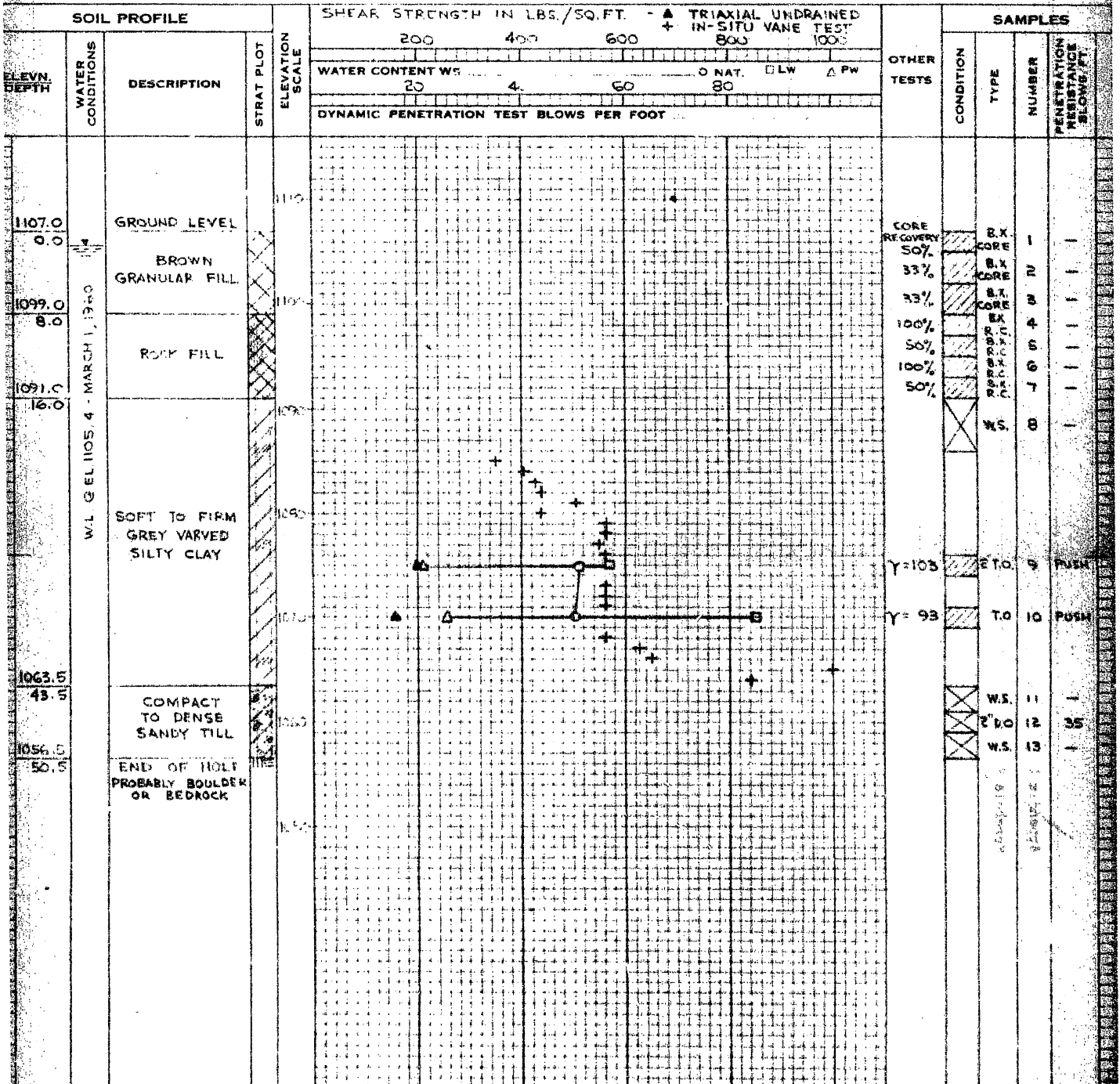
APPENDIX I

CONTRACT 57034 BORING # B DATUM GEODETIC CASING EX
 BORING DATE MARCH 11, 1962 REPORT DATE MARCH 20, 1962 COMPILED BY J.A. CHECKED BY J.H.
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

SAMPLE CONDITION
 DISTURBED
 FAIR
 GOOD
 LOST

SAMPLE TYPES
 A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE
 F.S. - FOIL SAMPLE
 S.O. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS
 V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 QC - TRIAXIAL CONSOLIDATED QUICK
 Q - TRIAXIAL QUICK
 S - TRIAXIAL SLOW
 γ - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL



GEOCON

OFFICE REPORT ON SOIL EXPLORATION

APPENDIX I

CONTRACT 57034 BORING # 9 DATUM GEODETIC CASING BX
 BORING DATE MARCH 17, 1960 REPORT DATE MARCH 29, 1960 COMPILED BY J.A. CHECKED BY JH
 SAMPLER HAMMER WT. 140 LBS. DROP 30 INCHES (PENETRATION RESISTANCES CONVERTED TO BLOWS OF 4200 IN. LBS. ENERGY)

SAMPLE CONDITION

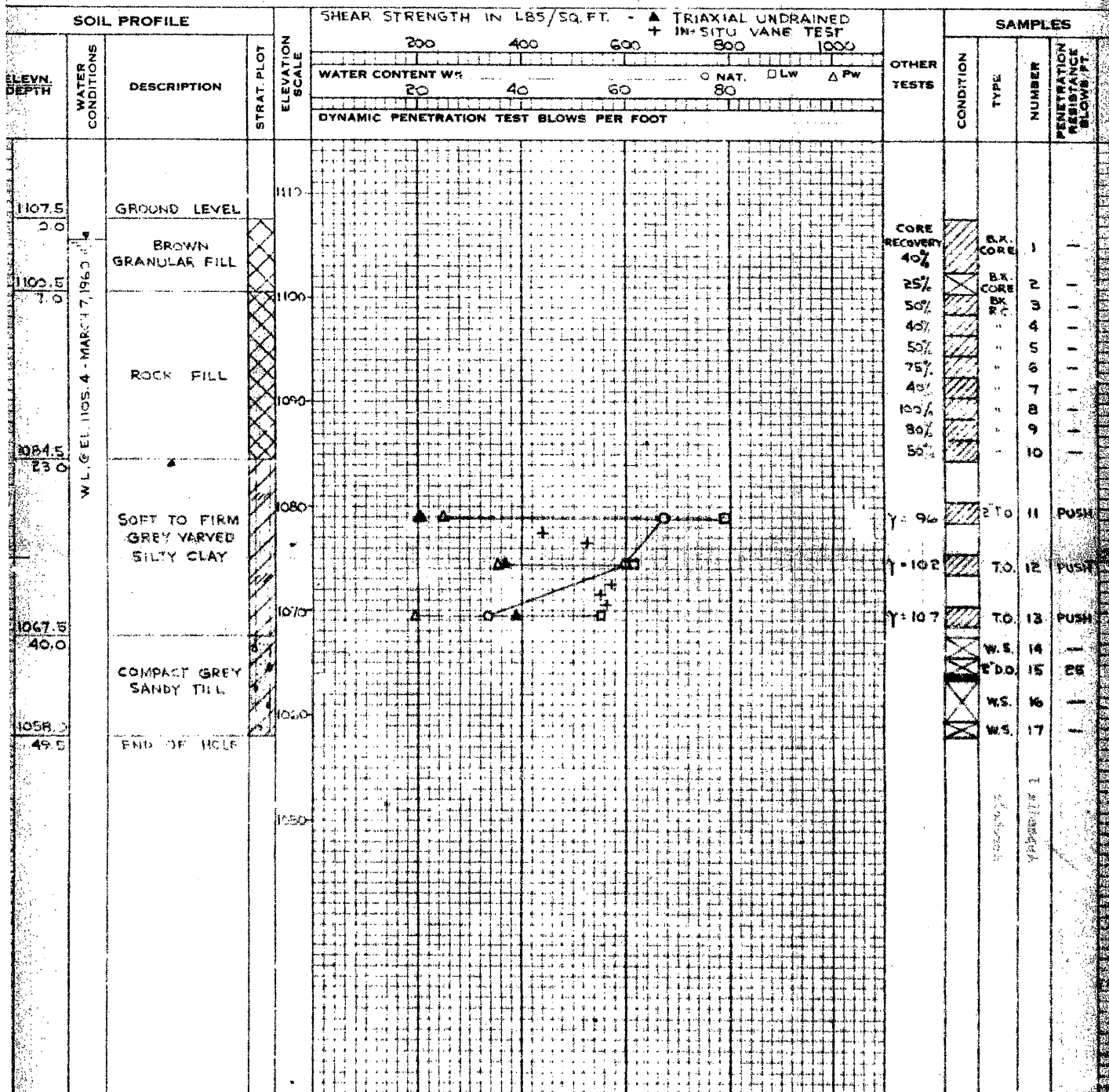
☐ DISTURBED
☐ FAIR
☐ GOOD
☐ LOST

SAMPLE TYPES

A.S. - AUGER SAMPLE
 S.T. - SLOTTED TUBE
 W.S. - WASHED SAMPLE
 D.O. - DRIVE-OPEN
 D.F. - DRIVE-FOOT VALVE
 C.S. - CHUNK SAMPLE
 F.S. - FOIL SAMPLE
 S.O. - SLEEVE-OPEN
 S.F. - SLEEVE-FOOT VALVE
 T.O. - THIN WALLED OPEN
 R.C. - ROCK CORE

ABBREVIATIONS

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 QC - TRIAXIAL CONSOLIDATED QUICK
 Q - TRIAXIAL QUICK
 S - TRIAXIAL SLOW
 1 - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION
 WL - WATER LEVEL IN CASING
 WT - WATER TABLE IN SOIL



OFFICE REPORT ON SOIL EXPLORATION

SAMPLE CONDITION

SAMPLE TYPES

ABBREVIATIONS

	DISTURBED
	FAIR
	GOOD
	LOST

A.S. - AUGER SAMPLE
S.T. - SLOTTED TUBE
W.S. - WASHED SAMPLE
D.O. - DRIVE-OPEN
D.F. - DRIVE-FOOT VALVE
C.S. - CHUNK SAMPLE

F.S. - FOIL SAMPLE
S.O. - SLEEVE-OPEN
S.F. - SLEEVE-FOOT VALVE
T.O. - THIN WALLED OPEN
R.C. - ROCK CORE

V - IN-SITU VANE TEST
M - MECHANICAL ANALYSIS
U - UNCONFINED COMPRESSION
QC - TRIAXIAL CONSOLIDATED QUICK
Q - TRIAXIAL QUICK
S - TRIAXIAL SLOW

γ - WET UNIT WEIGHT
K - PERMEABILITY
C - CONSOLIDATION

WL - WATER LEVEL IN CASINO
WT - WATER TABLE IN SOIL

SOIL PROFILE					OTHER TESTS					SAMPLES				
ELEVATION DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT	ELEVATION SCALE	WATER CONTENT W%					OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.
					DYNAMIC PENETRATION TEST BLOWS PER FOOT									
					10									
1105.4		ICE LEVEL		1110										
1.0		LAKE BOTTOM		1100										
		GREY SILTY CLAY		1090										
1086.4		END OF HOLE		1080										
19.0				1120										
		GROUND LEVEL		1110										
1113.0				1100										
0.0		GREY SILTY CLAY		1090										
				1080										
				1070										
1067.0		GREY SANDY TILL		1060										
46.0		END OF HOLE												
1063.0														
50.0														

OFFICE REPORT ON SOIL EXPLORATION

SAMPLE CONDITION

SAMPLE TYPES

ABBREVIATIONS

	DISTURBED
	FAIR
	GOOD
	LOST

A.S. - AUGER SAMPLE
S.T. - SLOTTED TUBE
W.S. - WASHED SAMPLE
D.O. - DRIVE-OPEN
D.F. - DRIVE-FOOT VALVE
C.S. - CHUNK SAMPLE

F.S. - FOIL SAMPLE
S.O. - SLEEVE OPEN
S.F. - SLEEVE-FOOT VALVE
T.O. - THIN WALLED OPEN
R.C. - ROCK CORE

V - IN-SITU VANE TEST
 M - MECHANICAL ANALYSIS
 U - UNCONFINED COMPRESSION
 QC - TRIAXIAL CONSOLIDATED QUICK
 Q - TRIAXIAL QUICK
 S - TRIAXIAL SLOW

7 - WET UNIT WEIGHT
 K - PERMEABILITY
 C - CONSOLIDATION

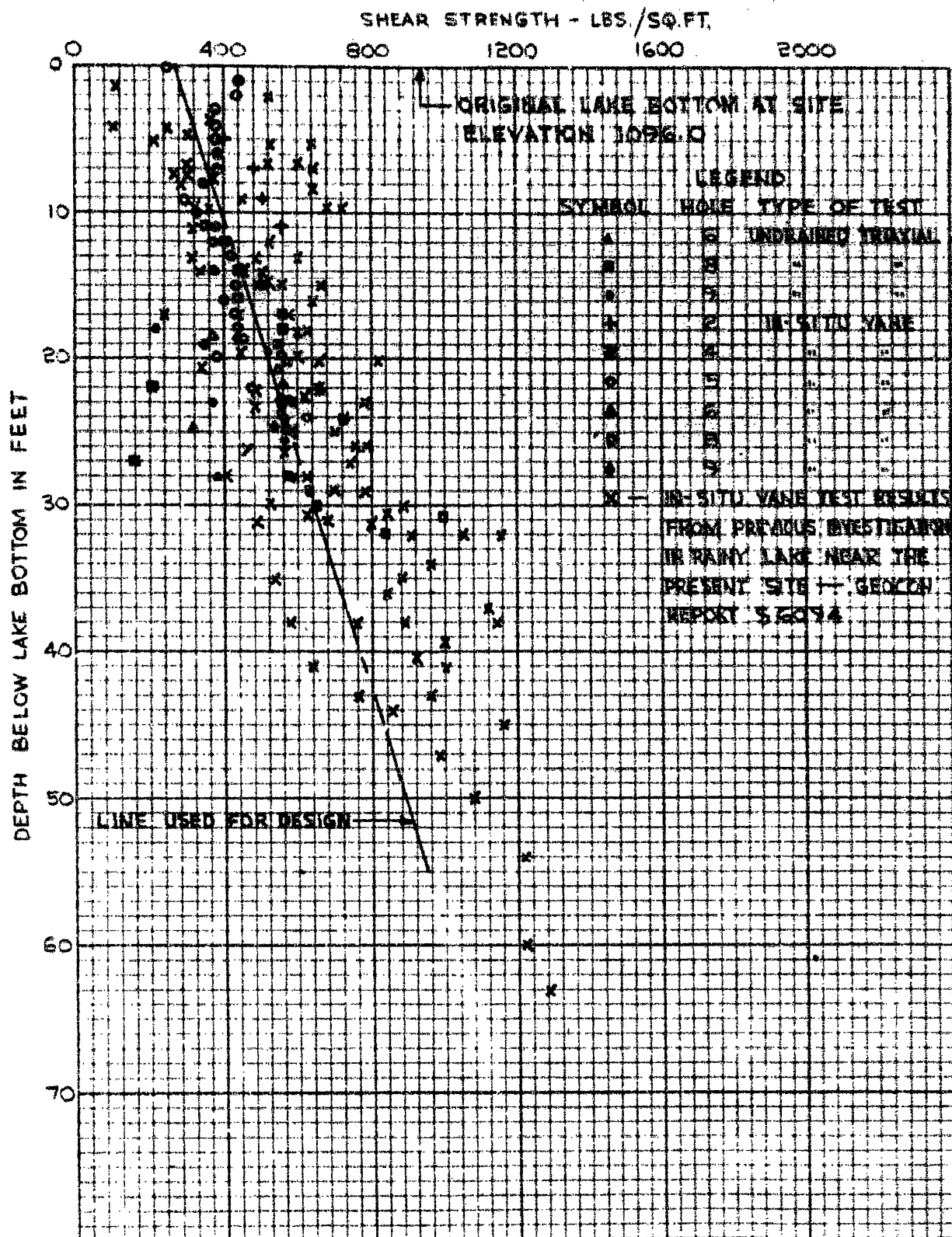
WL - WATER LEVEL IN CASING
WT - WATER TABLE IN SECTION

SOIL PROFILE								OTHER TESTS				SAMPLES							
ELEV. DEPTH	WATER CONDITIONS	DESCRIPTION	STRAT. PLOT ELEVATION SCALE	WATER CONTENT W _c				O NAT Lw Δ Pw				OTHER TESTS	CONDITION	TYPE	NUMBER	PENETRATION RESISTANCE BLOWS/FT.			
				DYNAMIC PENETRATION TEST BLOWS PER FOOT															
111.0 0.0		GROUND LEVEL	1120																
		GREY SILTY CLAY	1110																
			1100																
			1090																
			1080																
1069.0 42.0 1066.0 45.0		GREY SANDY TILL END OF HOLE	1070																
			1060																

SHEAR STRENGTH VS DEPTH BELOW LAKE BOTTOM

VARVED SILTY CLAY

APPENDIX II
FIGURE 1
PROJECT 57034



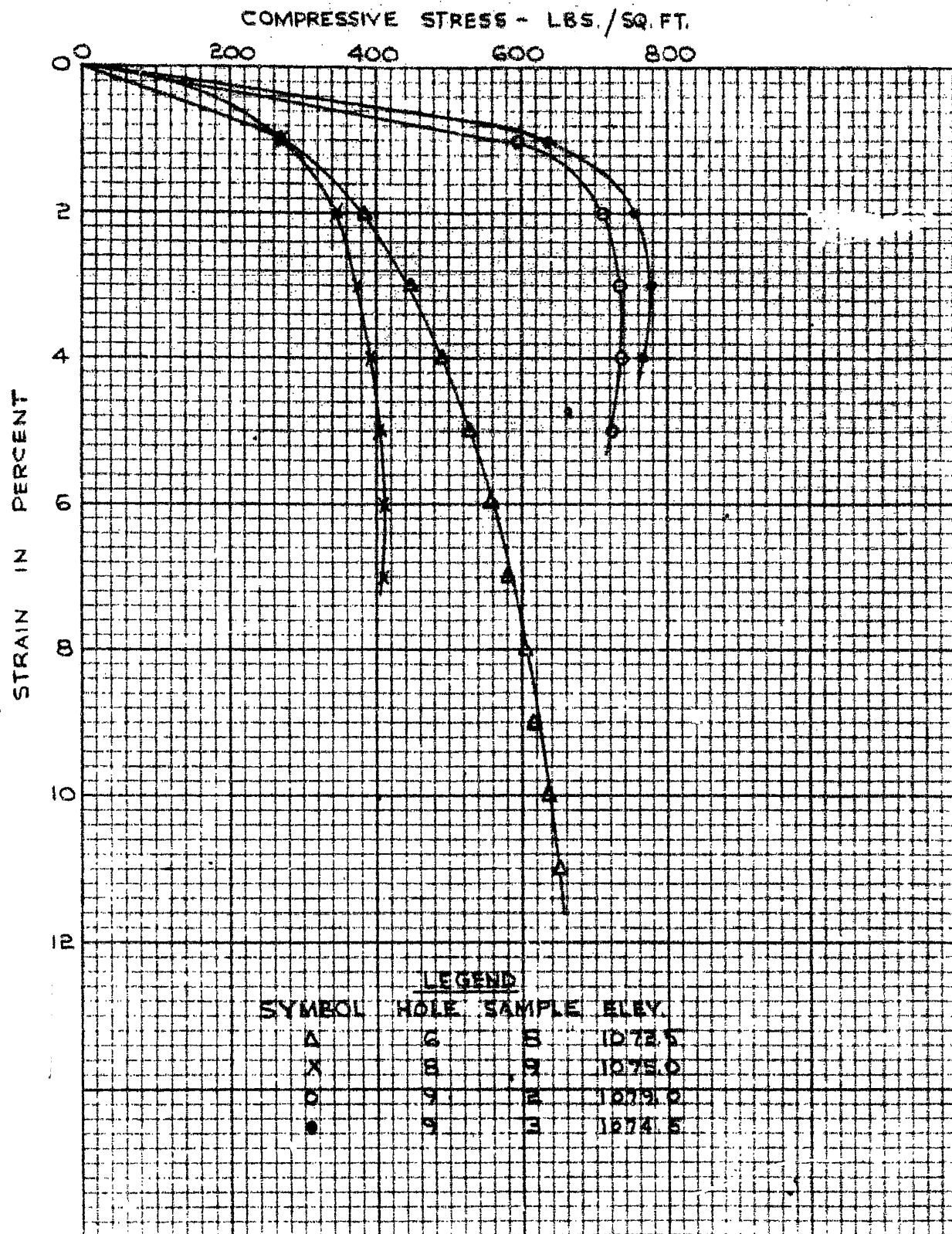
GEOCON

UNDRAINED TRIAXIAL COMPRESSION TESTS

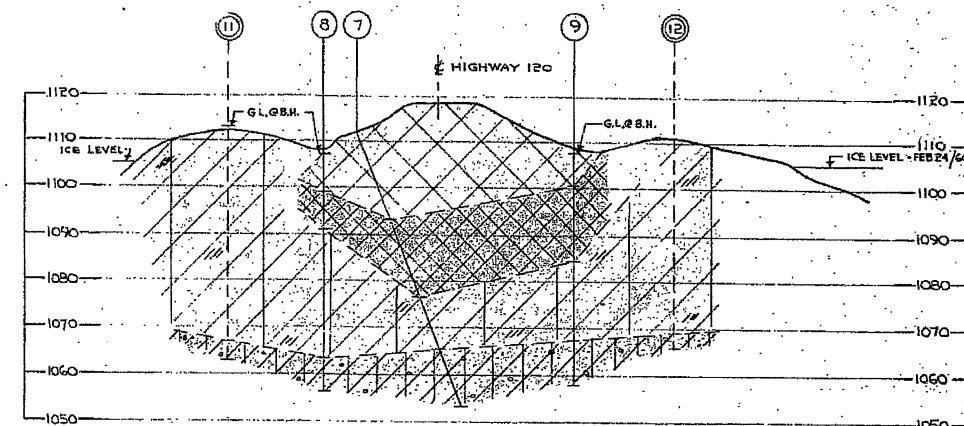
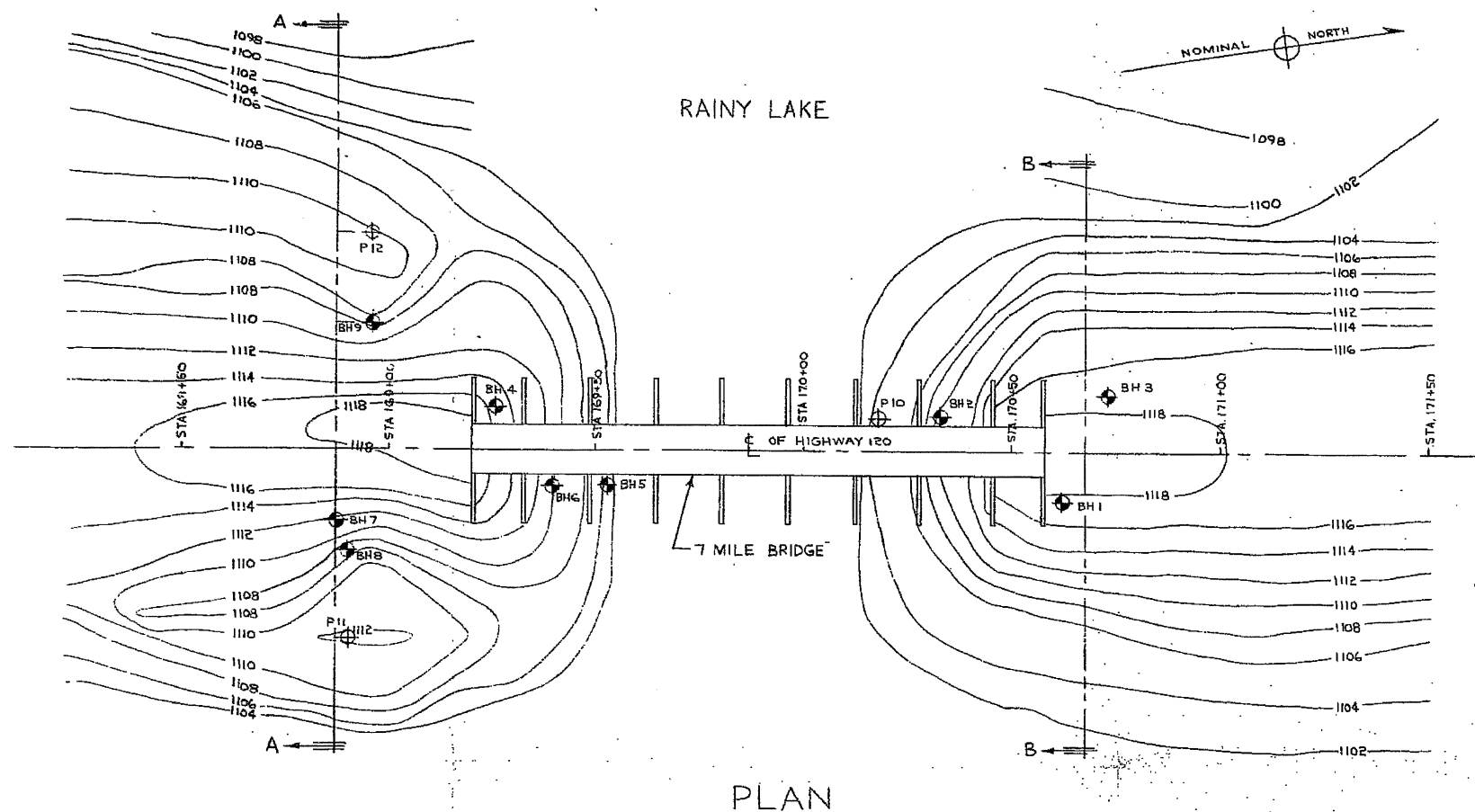
TYPICAL STRESS-STRAIN CURVES

GREY VARVED SILTY CLAY

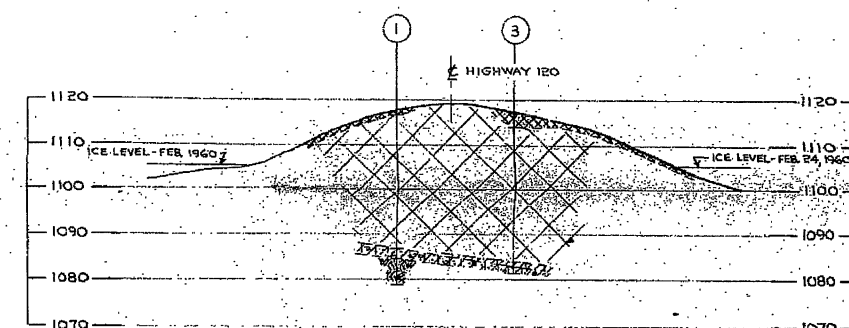
APPENDIX II
FIGURE 2
PROJECT S 7034



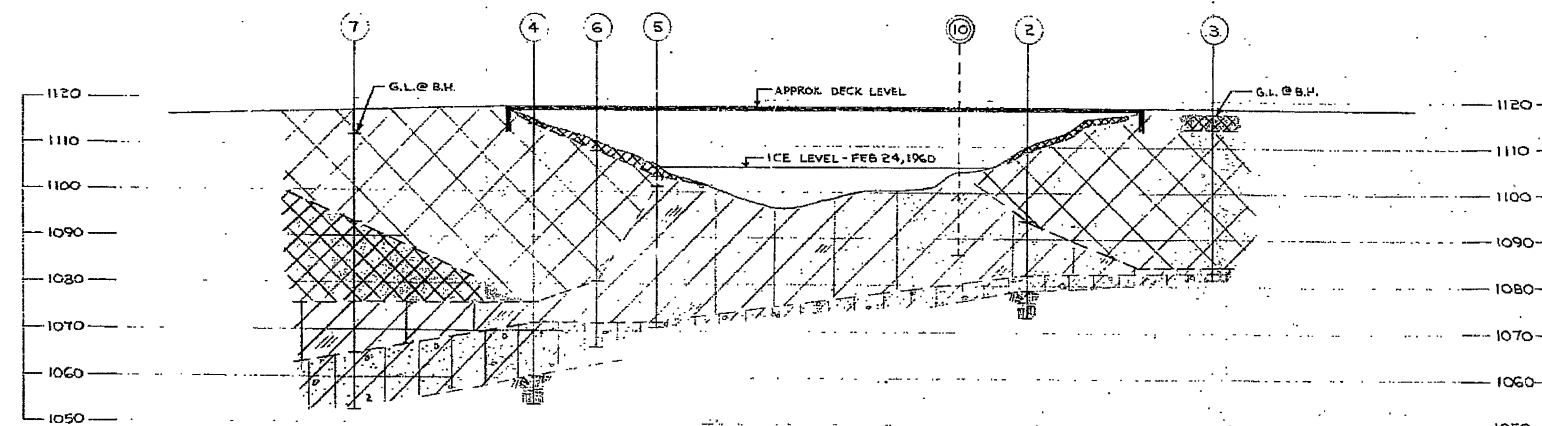
GEOCON



SECTION A-A



SECTION B-B



SECTION ALONG CENTRE LINE - HIGHWAY 120

- STRATIGRAPHY**
- ROCK FILL
 - LOOSE TO COMPACT BROWN GRANULAR FILL
 - SOFT TO FIRM GREY VARVED SILTY CLAY
 - COMPACT GREY SANDY TILL
 - SOUND GREY SYENITE GNEISS BEDROCK

- LEGEND**
- BOREHOLE IN PLAN
 - BOREHOLE IN ELEVATION
 - PROBING IN PLAN
 - PROBING IN ELEVATION

SPECIAL NOTE: DATA CONCERNING THE VARIOUS STRATA HAVE BEEN OBTAINED AT BOREHOLE LOCATIONS ONLY. THE SOIL STRATIGRAPHY BETWEEN BOREHOLES HAS BEEN INFERRED FROM GEOLOGICAL EVIDENCE AND SO MAY VARY FROM THAT SHOWN.

REVISIONS			REFERENCE			REFERENCE		
MARK	DATE	DESCRIPTION	DWG. NO.	DESCRIPTION	DWG. NO.	DESCRIPTION	DWG. NO.	DESCRIPTION

DEPARTMENT OF HIGHWAYS, ONTARIO
TORONTO

SEVEN MILE BRIDGE STRUCTURE
RAINY LAKE, ONTARIO

BORING PLAN AND SOIL STRATIGRAPHY

GEOCON LTD

DATE APRIL 6, 1960 SCALE 1" = 20'-0"

MADE BY J.A. CHKD. BY J.S. APPD. BY W.Y.

No. S 7034-2

Appendix D

Site Photographs



Photograph 1 – North Side of Seven Mile Bridge, looking east



Photograph 2 – North Side of Seven Mile Bridge, looking west



Photograph 3 – South Side of Seven Mile Bridge, looking east



Photograph 4 – South Side of Seven Mile Bridge, looking west



Photograph 5 – Seven Mile Bridge, looking east



Photograph 6 – Seven Mile Bridge, looking west

Appendix E

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES

Spread Footings	H-Piles Driven to Bedrock	Drilled-in Pipe Piles	Caissons/Drilled Piers
<p>Advantages:</p> <ul style="list-style-type: none"> i. Generally less costly construction than deep foundation elements. ii. High values of geotechnical resistance are available on the bedrock. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance available for piles driven to bedrock. ii. Excavation and dewatering requirements are minimized. iii. Pile base inspection not required. iv. Installation of piles could continue in freezing weather. v. Allows integral abutment design. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance available for pipe piles socketed into bedrock. ii. Excavation and dewatering requirements are minimized. iii. Liner is not required to support excavation sidewalls. iv. Less vibration and disturbance than driven piles. v. Cleaning and inspection of the socket base is not required. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance available for caissons socketed into bedrock ii. Construction of caissons could continue in freezing weather. iii. Excavation and dewatering requirements are minimized.
<p>Disadvantages:</p> <ul style="list-style-type: none"> i. Excavation depth to construct footings on native soils or bedrock is not practical. ii. Deep excavation in cohesionless soils extending below the groundwater level would require cofferdam construction. iii. Environmental impact on the lake during excavation. 	<p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit costs than footings. ii. Potential for pile deflection or refusal on cobbles, boulders and rock fill within the existing embankment fill. iii. No special installation methods required compared to caissons or drilled-in pipe piles. iv. Potential for embankment instability or spreading due to vibrations from pile driving. 	<p>Disadvantages:</p> <ul style="list-style-type: none"> i. Possibly higher unit cost compared to other foundation options such as footings. ii. Specialized installation. iii. Piles must be socketed into very strong bedrock. iv. Concreting or grouting of the annular space within the pile socket is required. 	<p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit cost compared to other foundation options such as footings or driven piles. ii. Specialized installation measures such as temporary liners and drilling mud will be required to install caissons under the water table. iii. Caissons must be socketed into very strong bedrock. iv. Potential difficulty in cleaning and inspecting rock sockets. v. Installation through cobbles and boulders will be difficult.
NOT RECOMMENDED	FEASIBLE	RECOMMENDED	NOT RECOMMENDED

Appendix F

List of SPs and OPSS, and Suggested Text for Selected NSSPs

1. List of Special Provisions and OPSS Documents Referenced in this Report

- OPSS 501
- OPSS 539
- OPSS 804
- OPSS 902
- OPSS 903
- OPSS 1010
- OPSD 208.010
- Special Provision 110S13

2. Suggested Text for NSSP on “Construction of Driven H-piles”

Installation of H-piles shall be in accordance with OPSS 903 and the following.

Cobbles, boulders and rock fill are present within the existing embankment fill on site. Boulders and rock slabs are present on the embankment slopes. The cobbles, boulders and rock fill may interfere with pile installation and some piles may meet refusal on boulders or rock fill above the bedrock surface. The Contractor must be prepared to remove, dislodge or otherwise penetrate these obstructions to advance the piles to bedrock while meeting the specified deflection tolerances.

The H-piles must be provided with pile tip protectors to minimize tip damage.

If the piles meet refusal at a depth less than the anticipated depth, the QVE must terminate driving before the pile is damaged due to over-driving. The QVE must immediately bring it to the attention of the CA. If the CA cannot resolve the issue, it must be referred to the design team for resolution.

3. Suggested Text for NSSP on “Construction of Drilled-in Pipe Piles”

Installation of drilled-in pipe piles shall be in accordance with OPSS 903 and the following.

Drilled-in pipe pile installation at this site will require excavation through granular embankment fill with cobbles, boulders and rock fill, as well as cohesionless soils below the groundwater table. The piles must also be advanced into the underlying bedrock. The Contractor is advised of the following:

- The installation methods and equipment must be capable of dislodging, removing or otherwise penetrating cobbles or boulders in the embankment fill.
- The bedrock consists of very strong monzodiorite (granite-like) rock. The strength and hardness of the bedrock must be taken into account when selecting equipment to advance the pile into rock. Equipment supplied to advance the pile into rock must be capable of penetrating the bedrock to create a clean socket without disturbing or fracturing the bedrock adjacent to the pile. Blasting to facilitate the removal of bedrock is not permitted.
- The rock embedment length must be formed entirely within the bedrock below the level of any cobbles overlying the bedrock. Any length of pile above the bedrock surface will not be considered part of the specified length of rock embedment.
- The annular space between the rock socket wall and pile shall be filled with 30MPa concrete or grout to the top of the bedrock surface. The plumbness and alignment of the pile shall be maintained during concreting.
- During and subsequent to installation, the pipe pile may be partially filled with water and it may not be practical to dewater the pipe prior to concreting. Tremie concreting will be required for concreting these pipe piles.

4. Suggested Text for NSSP on “Construction of Caissons”

Caisson installation shall be in accordance with OPSS 903. The Contractor is further advised of the following:

- The installation methods and equipment must be capable of dislodging, removing or otherwise penetrating cobbles or boulders in the embankment fill.
- Caissons will extend through cohesionless soils below the groundwater level. Measures must be employed to maintain sidewall stability in the caisson excavation and prevent collapse/washing of cohesionless soils into the rock socket. Selection of the methods and equipment employed to achieve this is the responsibility of the Contractor.
- The bedrock consists of very strong monzodiorite (granite-like) rock. The strength and hardness of the bedrock must be taken into account when selecting equipment to advance the pile into rock. Equipment supplied to advance the pile into rock must be capable of penetrating the bedrock without disturbing or fracturing the bedrock adjacent to the pile. Blasting to facilitate the removal of bedrock is not permitted.
- The rock socket must be formed entirely within the bedrock below the level of any cobbles, boulders and highly fractured material. Any length of caisson above the bedrock surface will not be considered part of the specified length of rock socket.

5. Suggested Text for NSSP on Installation of Steel Sheet Piles

Cobbles, boulders and rock fill are present within the existing embankment fill on site. Boulders and rock slabs are present on the embankment slopes. These cobbles and boulders may impede the driving of sheet piles and at some locations the sheet piles may not be able to penetrate the cobbles and boulders and reach the design depth of installation.

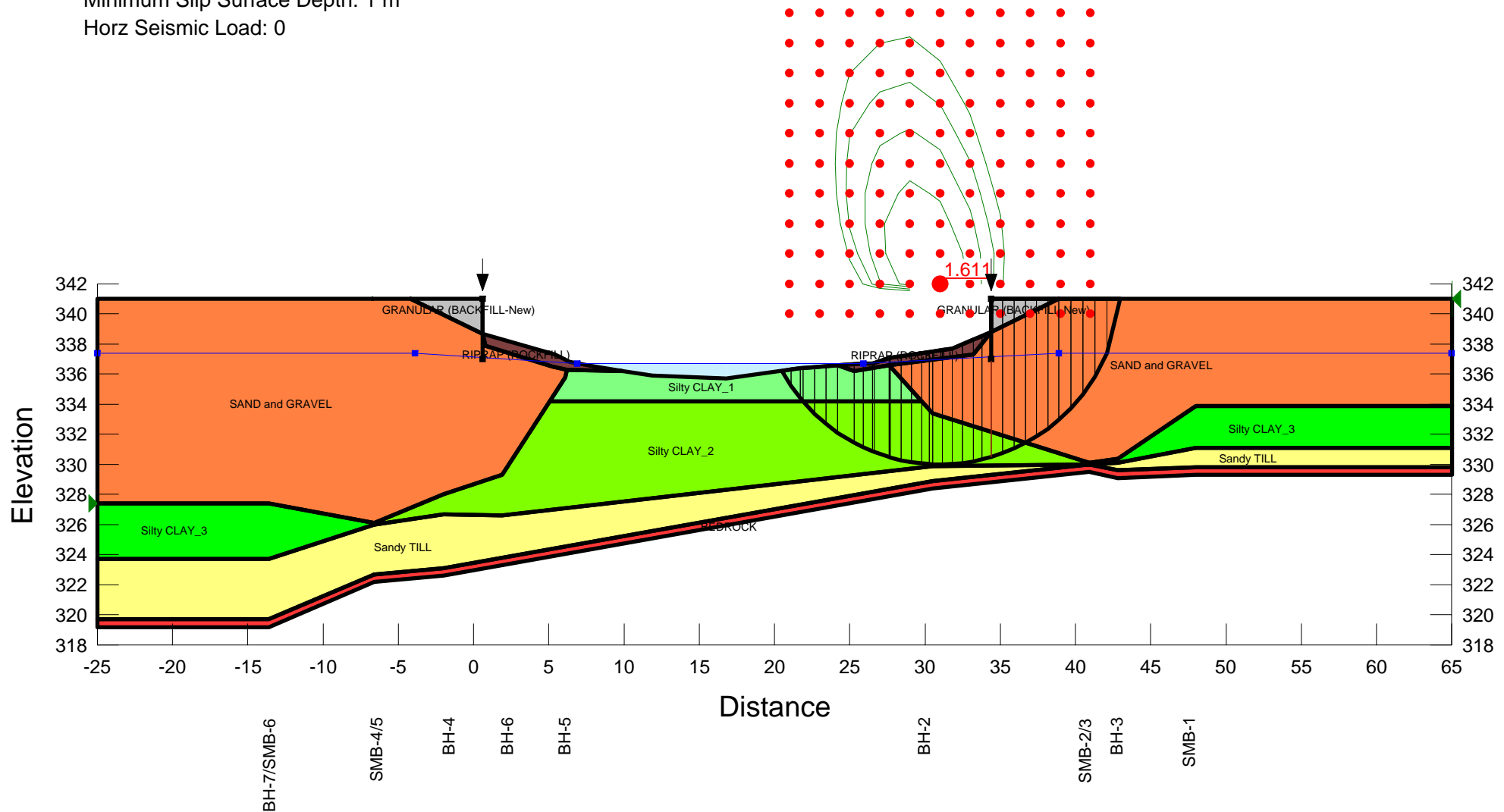
The Contractor shall be prepared to remove, drill through and/or penetrate these obstructions and extend the piles to the design depth.

Appendix G

Slope Stability Output

Title: Seven Mile Bridge
 Name: Analysis E.0.1
 Description: East Abutment
 Comments: Stability Analysis
 Last Edited By: Stephen Peters
 Last Solved Date: 9/3/2013, 11:25:19 AM
 Method: Morgenstern-Price, Half-Sine
 Minimum Slip Surface Depth: 1 m
 Horz Seismic Load: 0

SAND and GRAVEL	21 kN/m ³	0 kPa	34 °	1
RIPRAP (ROCKFILL)	19 kN/m ³	0 kPa	42 °	1
Sandy TILL	21 kN/m ³	0 kPa	32 °	1
BEDROCK	1			
GRANULAR (BACKFILL-New)	21 kN/m ³	0 kPa	32 °	1
Silty CLAY_1	18 kN/m ³	14 kPa	0 °	1
Silty CLAY_2	18 kN/m ³	14 kPa	2.5 kPa/m	0 kPa 334.2 m 1
Silty CLAY_3	18 kN/m ³	40 kPa	0 °	1



Title: Seven Mile Bridge

Name: Analysis E.0.2

Description: East Abutment

Comments: Stability Analysis

Last Edited By: Stephen Peters

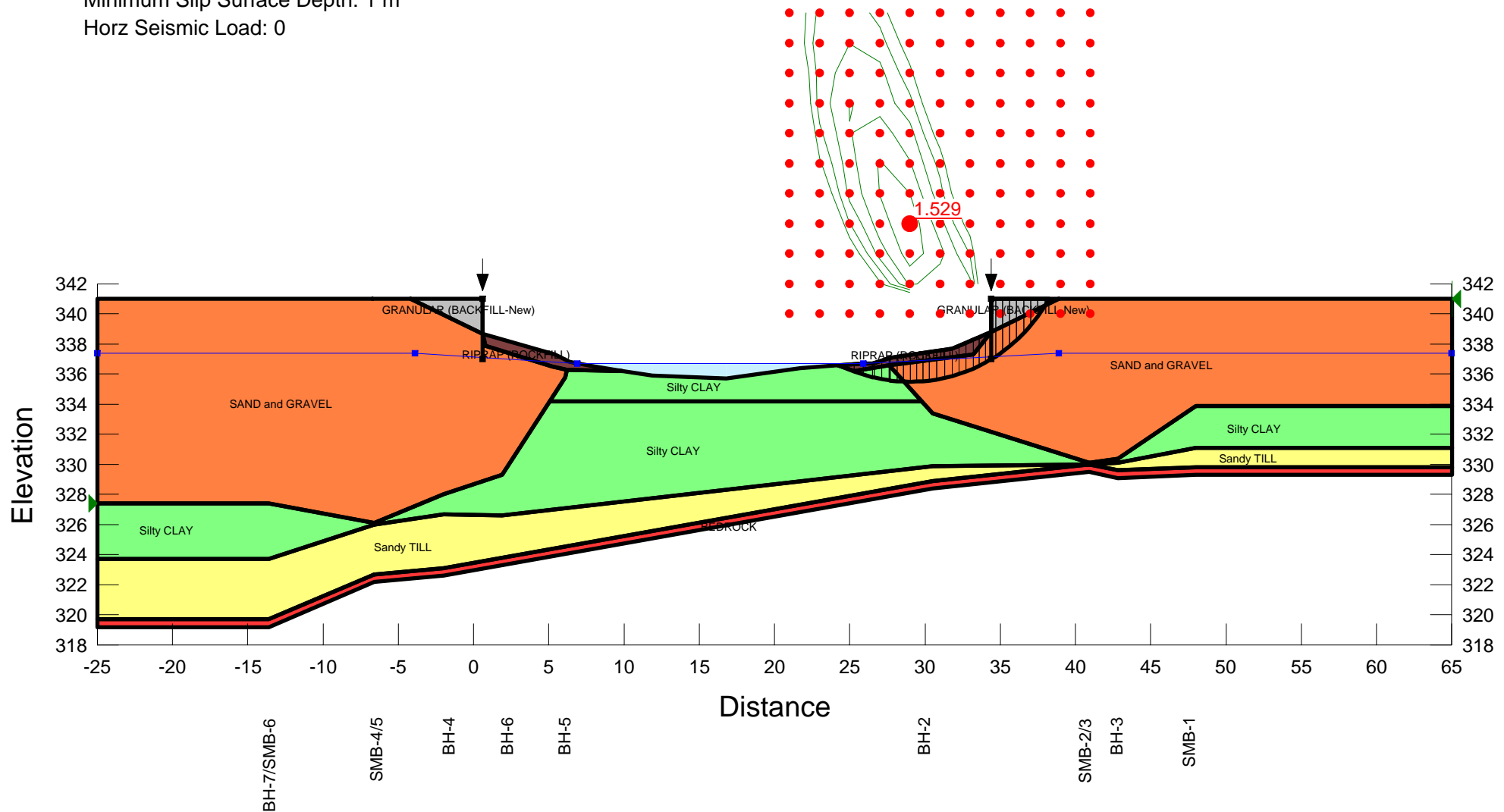
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Method: Morgenstern-Price, Half-Sine

Minimum Slip Surface Depth: 1 m

Horz Seismic Load: 0

SAND and GRAVEL	21 kN/m ³	0 kPa	34 °	1
RIPRAP (ROCKFILL)	19 kN/m ³	0 kPa	42 °	1
Silty CLAY	18 kN/m ³	0 kPa	27 °	1
Sandy TILL	21 kN/m ³	0 kPa	32 °	1
BEDROCK	1			
GRANULAR (BACKFILL-New)	21 kN/m ³	0 kPa	32 °	1



Title: Seven Mile Bridge

Name: Analysis W.0.1

Description: West Abutment

Comments: Stability Analysis

Last Edited By: Stephen Peters

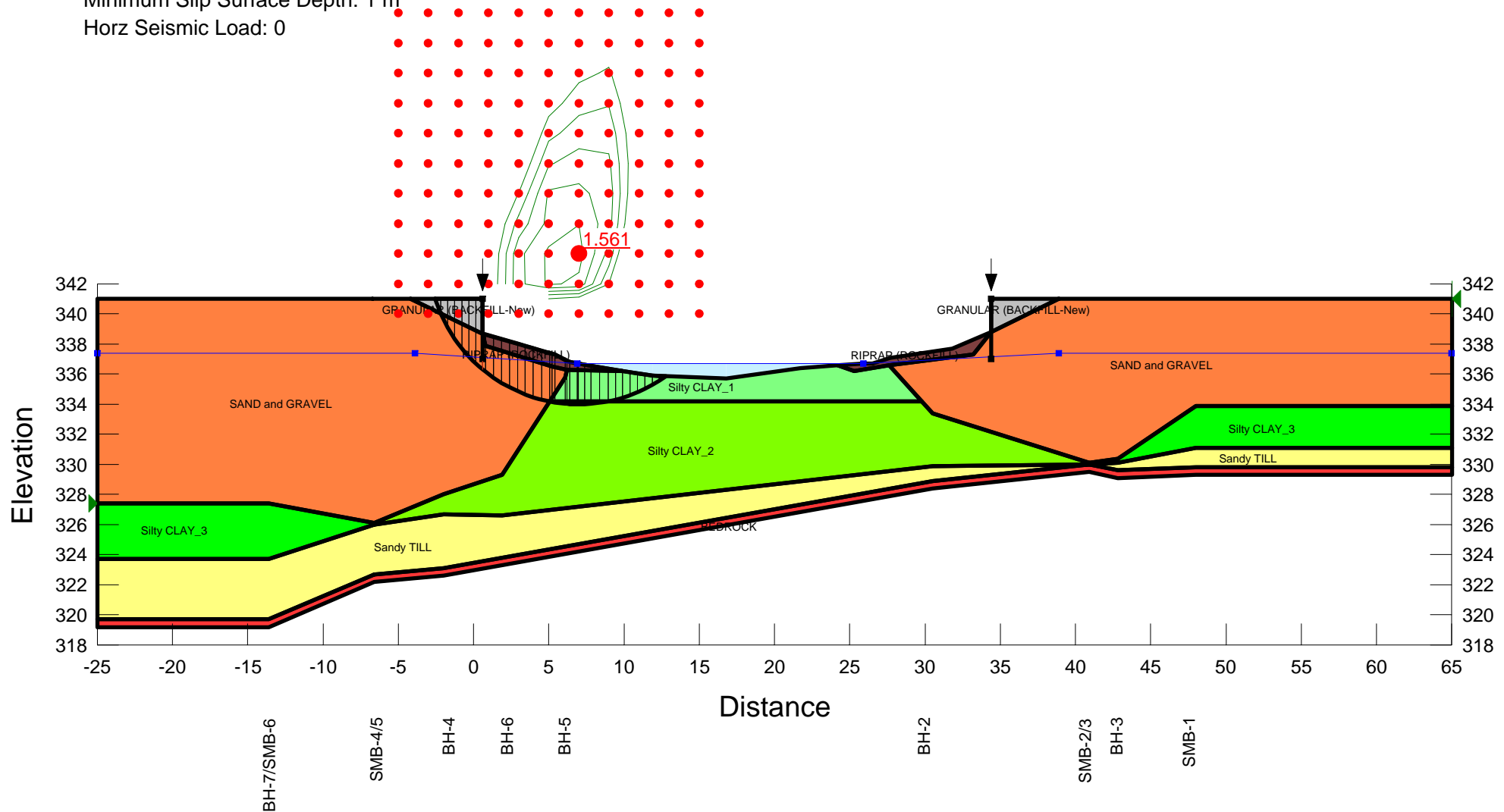
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Method: Morgenstern-Price, Half-Sine

Minimum Slip Surface Depth: 1 m

Horz Seismic Load: 0

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RIPRAP (ROCKFILL)	19 kN/m ³	0 kPa	42 °	1
Sandy TILL	21 kN/m ³	0 kPa	32 °	1
BEDROCK	1			
GRANULAR (BACKFILL-New)	21 kN/m ³	0 kPa	32 °	1
Silty CLAY_1	18 kN/m ³	14 kPa	0 °	1
Silty CLAY_2	18 kN/m ³	14 kPa	2.5 kPa/m	0 kPa 334.2 m 1
Silty CLAY_3	18 kN/m ³	40 kPa	0 °	1



Title: Seven Mile Bridge

Name: Analysis W.0.2

Description: West Abutment

Comments: Stability Analysis

Last Edited By: Stephen Peters

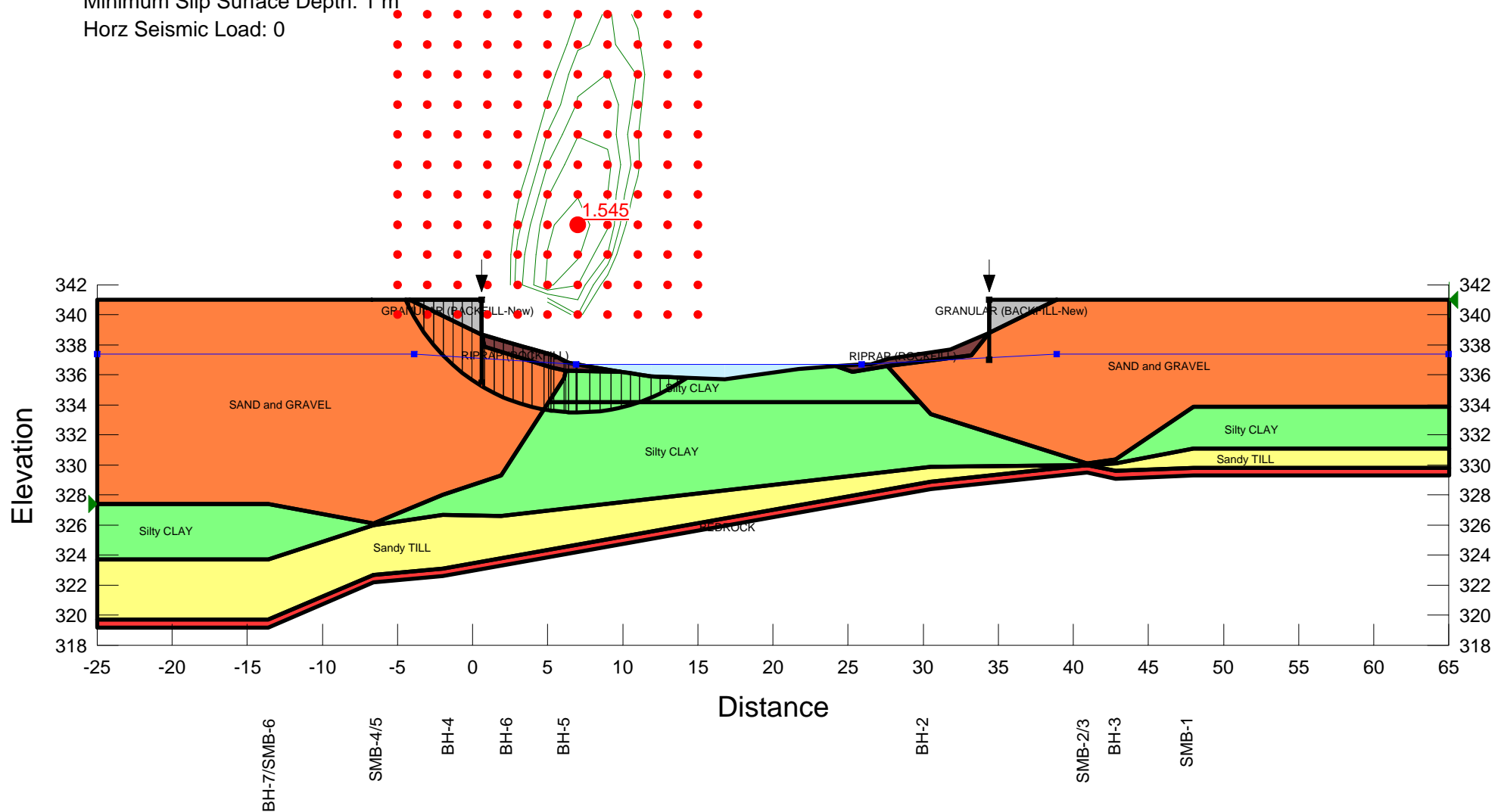
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Method: Morgenstern-Price, Half-Sine

Minimum Slip Surface Depth: 1 m

Horz Seismic Load: 0

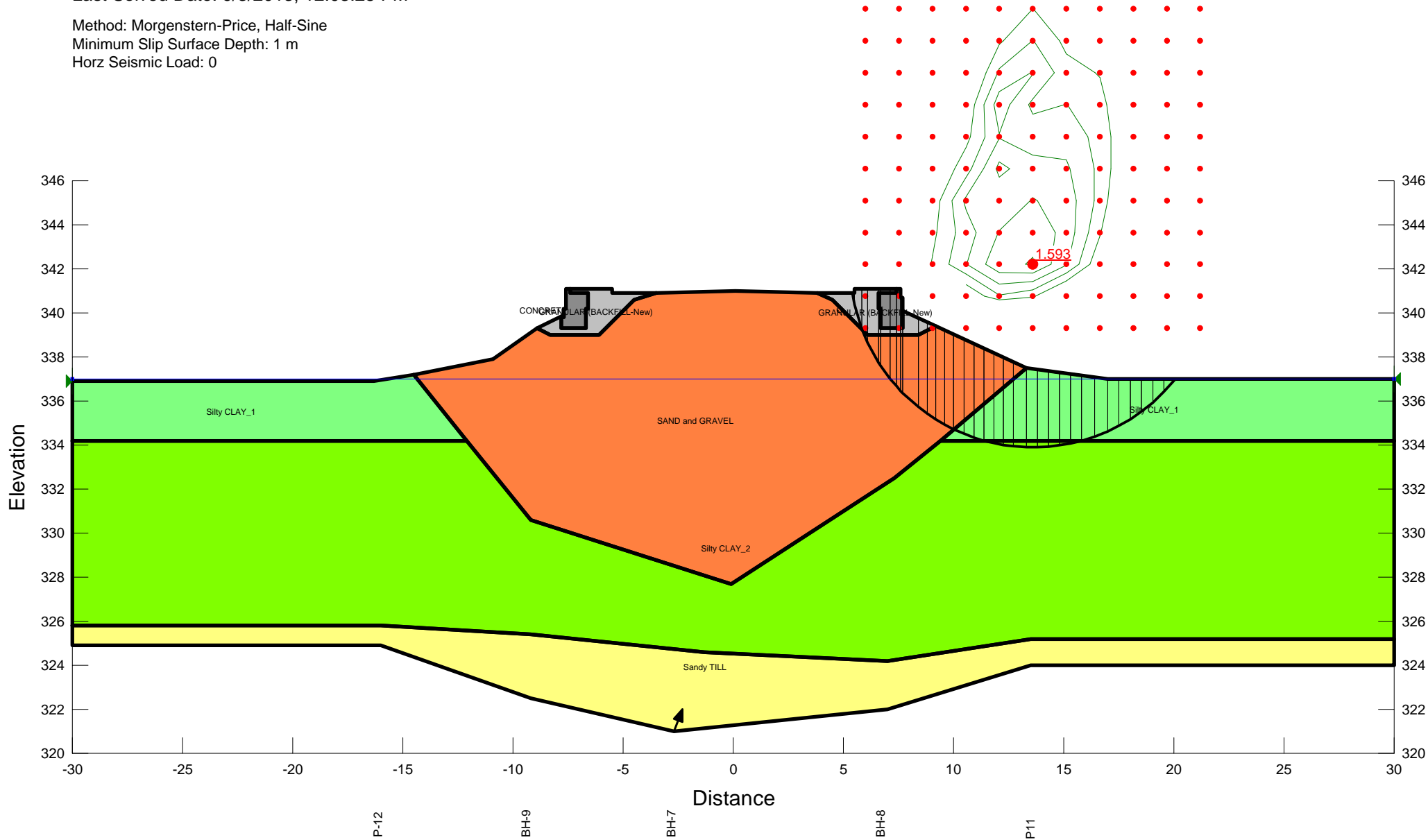
SAND and GRAVEL	21 kN/m ³	0 kPa	34 °	1
RIPRAP (ROCKFILL)	19 kN/m ³	0 kPa	42 °	1
Silty CLAY	18 kN/m ³	0 kPa	27 °	1
Sandy TILL	21 kN/m ³	0 kPa	32 °	1
BEDROCK	1			
GRANULAR (BACKFILL-New)	21 kN/m ³	0 kPa	32 °	1



Title: Seven Mile Bridge
 Name: Analysis 0.0.1
 Description: West Abutment
 Comments: Stability Analysis
 Last Edited By: Stephen Peters
 Last Solved Date: 9/3/2013, 12:03:25 PM

Method: Morgenstern-Price, Half-Sine
 Minimum Slip Surface Depth: 1 m
 Horz Seismic Load: 0

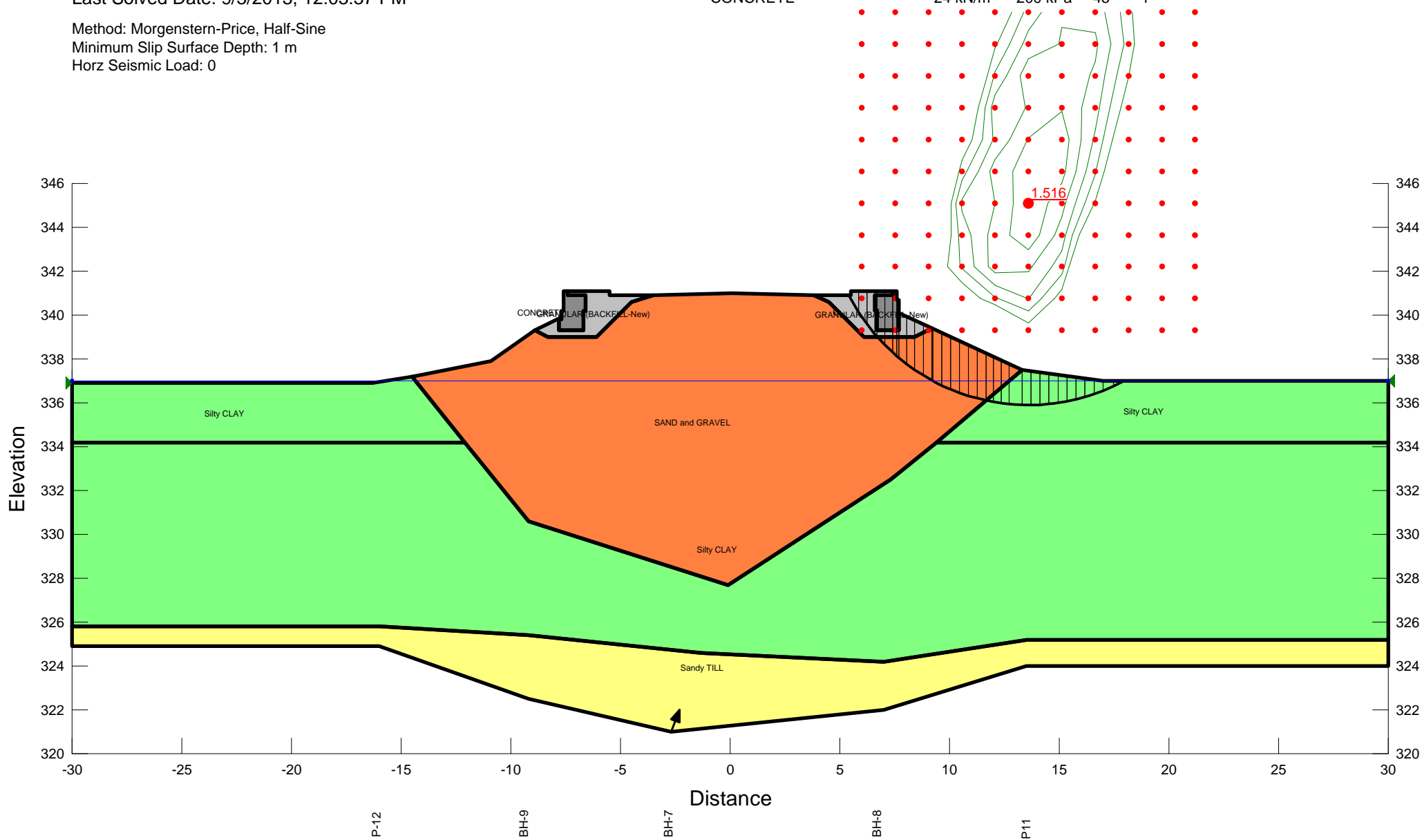
SAND and GRAVEL	21 kN/m ³	0 kPa	34 °	1
Sandy TILL	21 kN/m ³	0 kPa	32 °	1
GRANULAR (BACKFILL-New)	21 kN/m ³	0 kPa	34 °	1
Silty CLAY_1	18 kN/m ³	14 kPa	0 °	1
Silty CLAY_2	18 kN/m ³	14 kPa	2.5 kPa/m	0 kPa 334.2 m 1
CONCRETE	24 kN/m ³	200 kPa	45 °	1



Title: Seven Mile Bridge
 Name: Analysis 0.0.2
 Description: West Abutment
 Comments: Stability Analysis
 Last Edited By: Stephen Peters
 Last Solved Date: 9/3/2013, 12:03:37 PM

Method: Morgenstern-Price, Half-Sine
 Minimum Slip Surface Depth: 1 m
 Horz Seismic Load: 0

SAND and GRAVEL	21 kN/m ³	0 kPa	34 °	1
Silty CLAY	18 kN/m ³	0 kPa	27 °	1
Sandy TILL	21 kN/m ³	0 kPa	32 °	1
GRANULAR (BACKFILL-New)	21 kN/m ³	0 kPa	34 °	1
CONCRETE	24 kN/m ³	200 kPa	45 °	1



Horz Seismic Load: 0

CONCRETE

24 kN/m³ 200 kPa 45 ° 1

0 kPa 334.2 m 1

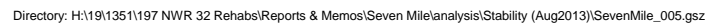


FIGURE 7

Title: Seven Mile Bridge

Name: Analysis 0.1.2

Description: West Abutment

Comments: Stability Analysis

Last Edited By: Stephen Peters

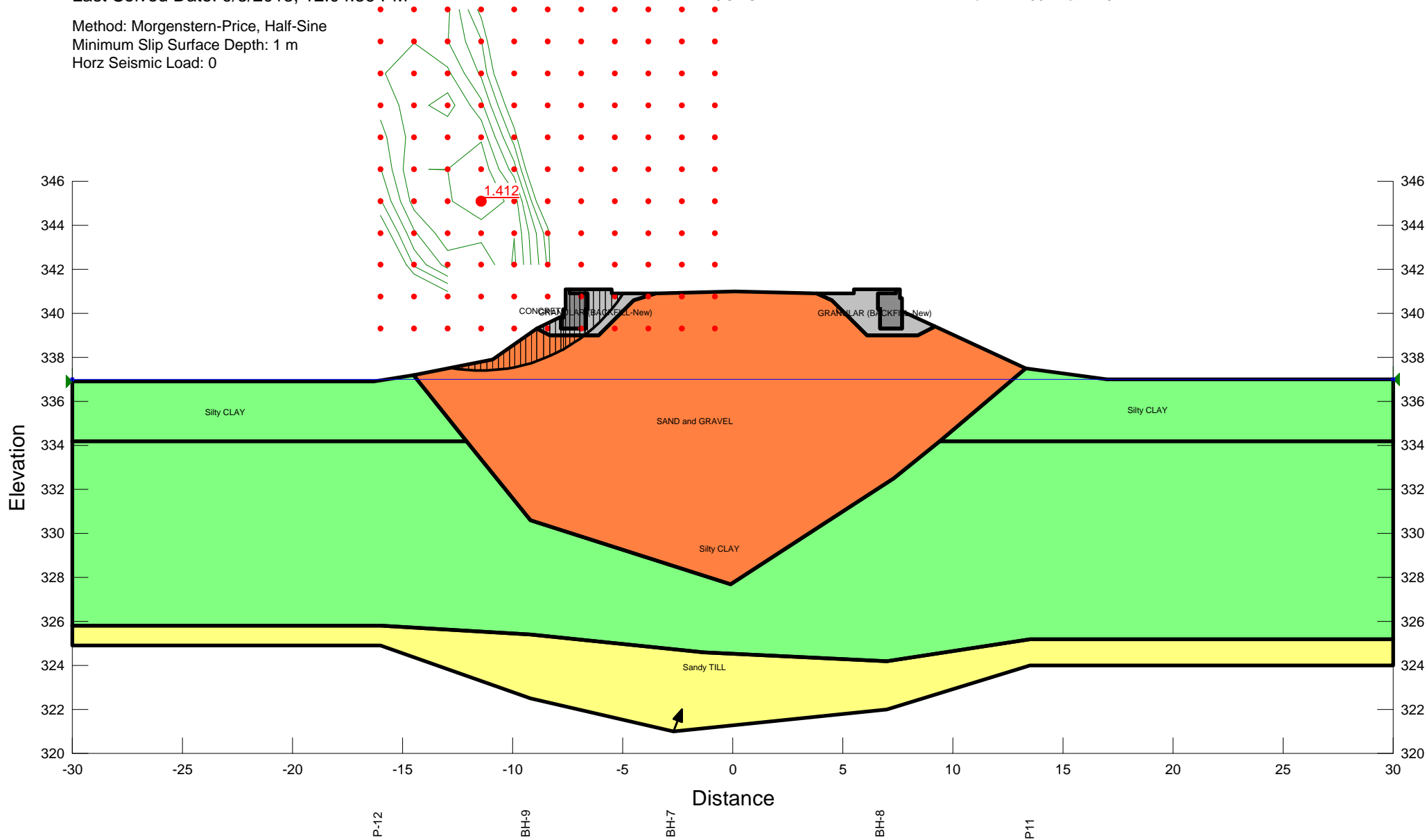
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Method: Morgenstern-Price, Half-Sine

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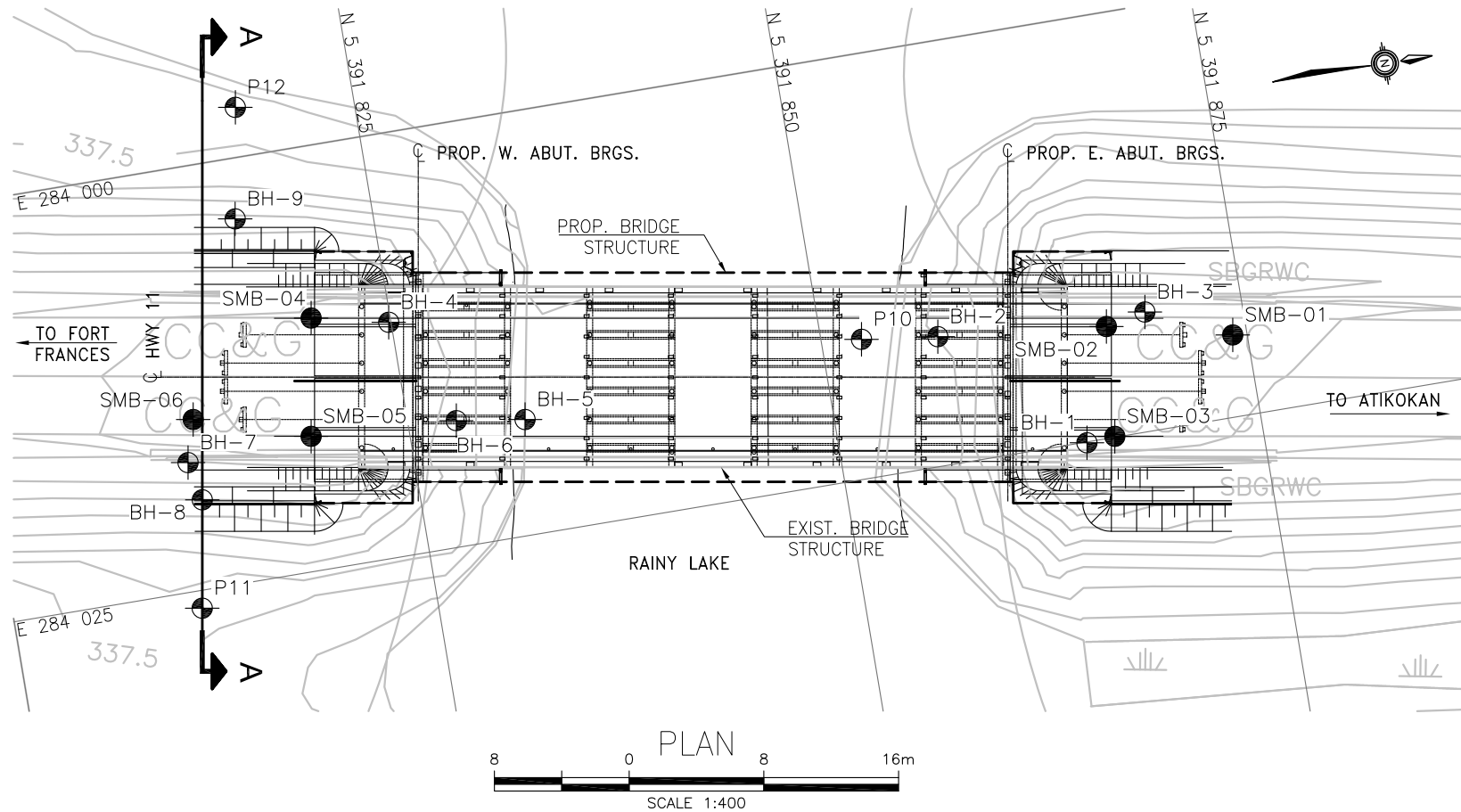
Horz Seismic Load: 0

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Silty CLAY	18 kN/m ³	0 kPa	27 °	1
Sandy TILL	21 kN/m ³	0 kPa	32 °	1
GRANULAR (BACKFILL-New)	21 kN/m ³	0 kPa	34 °	1
CONCRETE	24 kN/m ³	200 kPa	45 °	1

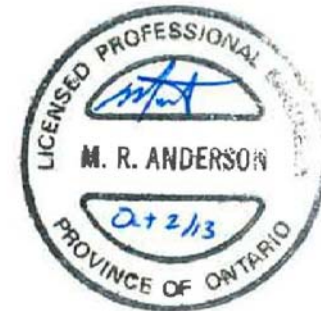


Appendix H

Drawing titled “Borehole Locations and Soil Strata”



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

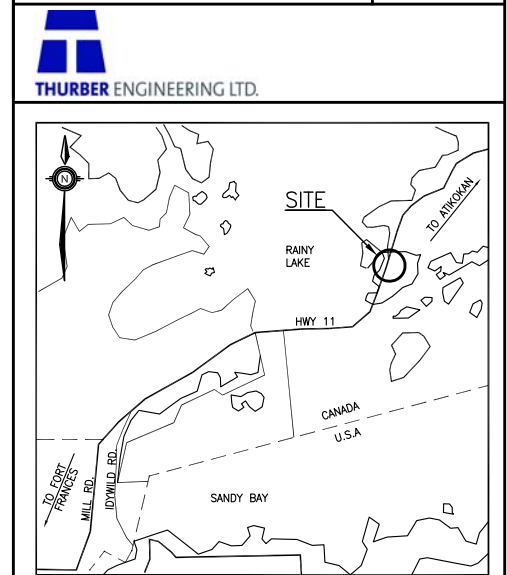


HWY 11
CONT No. 2013-6021
WP No. 6049-08-02

HIGHWAY 11
SEVEN MILE BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

MRC
McCORMICK RANKIN
A member of MMM GROUP

SHEET
12



KEYPLAN

LEGEND

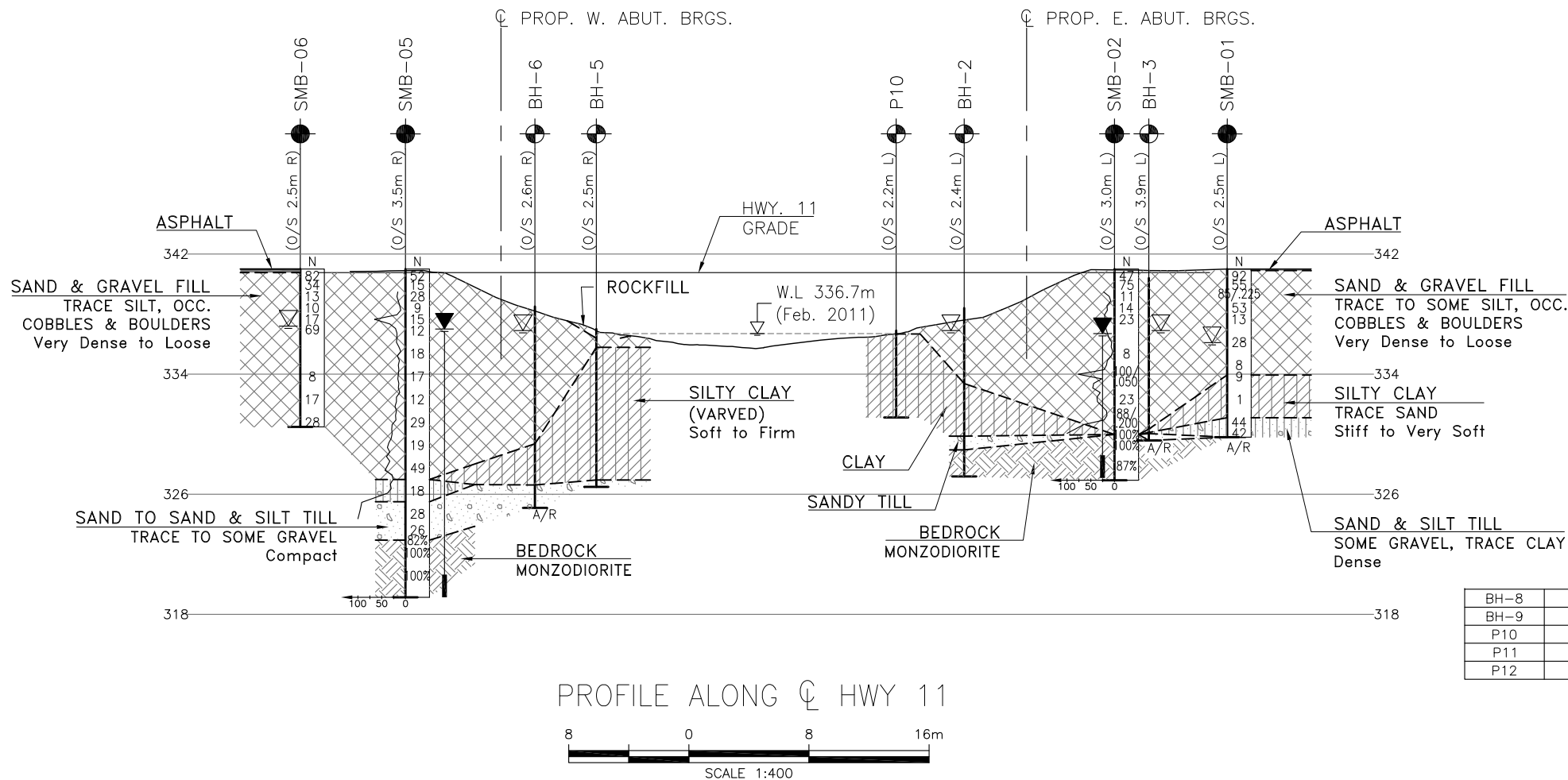
●	Borehole (Current Investigation)
○	Borehole (Previous Investigation)
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
▽	Water Level During Drilling
↑	Water Level In Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
SMB-01	341.0	5 391 874.2	284 020.2
SMB-02	341.0	5 391 866.8	284 018.4
SMB-03	341.0	5 391 866.3	284 024.9
SMB-04	341.0	5 391 820.4	284 010.1
SMB-05	341.0	5 391 819.2	284 017.1
SMB-06	341.1	5 391 812.5	284 014.9
BH-1	340.7	5 391 864.6	284 025.0
BH-2	338.4	5 391 856.9	284 017.4
BH-3	340.4	5 391 869.2	284 018.0
BH-4	339.9	5 391 824.9	284 011.1
BH-5	337.0	5 391 831.9	284 018.2
BH-6	338.5	5 391 827.9	284 017.6
BH-7	338.9	5 391 811.7	284 017.4

-NOTES-

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

GEOCRIS No. 52C-28



BH-8	337.4	5 391 812.2	284 019.7
BH-9	337.6	5 391 816.9	284 003.6
P10	336.9	5 391 852.4	284 016.8
P11	339.2	5 391 811.1	284 026.1
P12	338.6	5 391 818.0	283 997.1

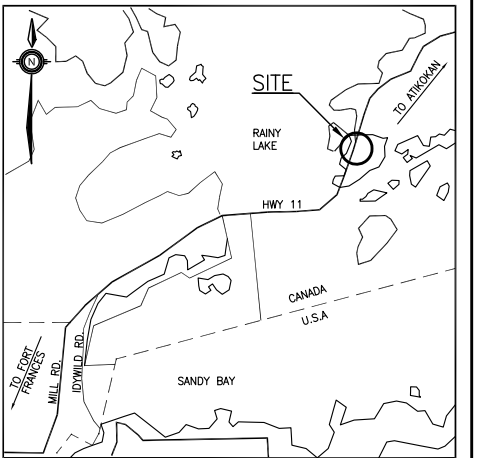
REVISIONS	DATE	BY	DESCRIPTION
DESIGN	RPR	CHK	RPR
DRAWN	AN	CHK	
CODE	LOAD		
SITE	45-76	STRUCT	DWG 2
DATE	OCT. 2013		

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

HWY 11
CONT No. 2013-6021
WP No. 6049-08-02





HIGHWAY 11
SEVEN MILE BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

HEET
13



KEYPLAN

LEGEND

	Borehole (Current Investigation)
	Borehole (Previous Investigation)
N	Blows /0.3m (Std Pen Test, 475J/blow)
CONE	Blows /0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
	Water Level During Drilling
	Water Level In Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

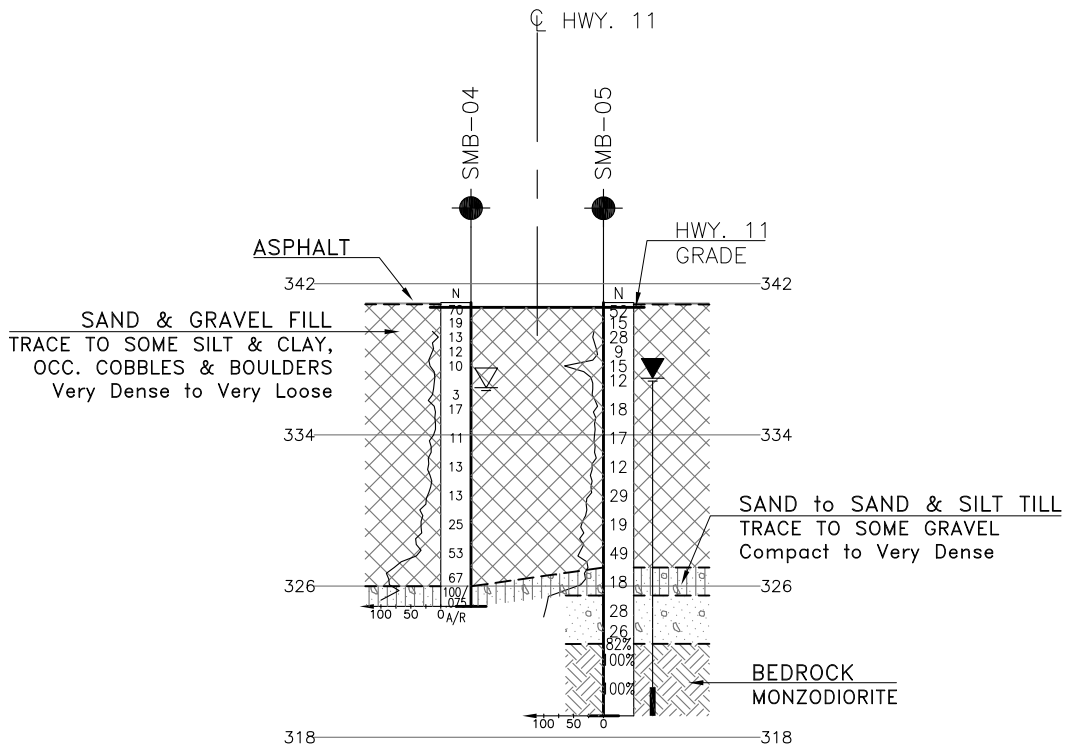
NO	ELEVATION	NORTHING	EASTING
SMB-01	341.0	5 391 874.2	284 020.2
SMB-02	341.0	5 391 866.8	284 018.4
SMB-03	341.0	5 391 866.3	284 024.9
SMB-04	341.0	5 391 820.4	284 010.1
SMB-05	341.0	5 391 819.2	284 017.1
SMB-06	341.1	5 391 812.5	284 014.9
BH-1	340.7	5 391 864.6	284 025.0
BH-2	338.4	5 391 856.9	284 017.4
BH-3	340.4	5 391 869.2	284 018.0
BH-4	339.9	5 391 824.9	284 011.1
BH-5	337.0	5 391 831.9	284 018.2
BH-6	338.5	5 391 827.9	284 017.6
BH-7	338.9	5 391 811.7	284 017.4

-NOTES-

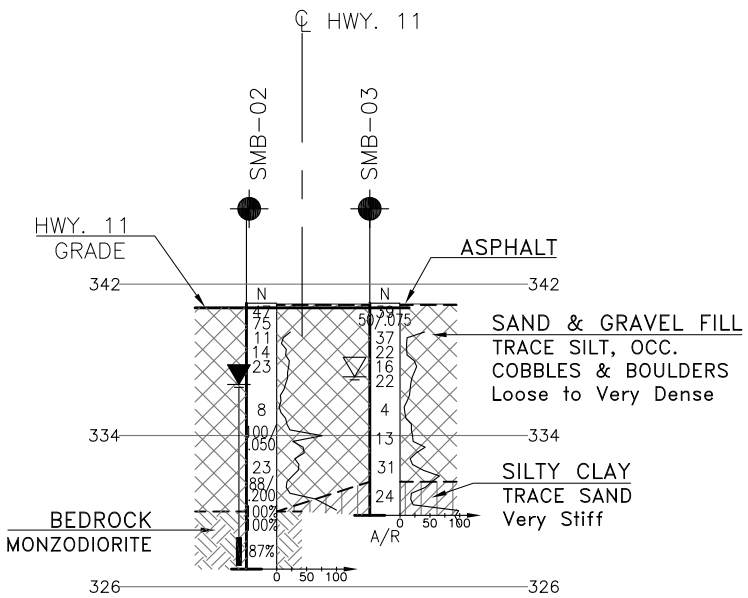
- 1) The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.
- 2) This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

GEOCRES No. 52C-28

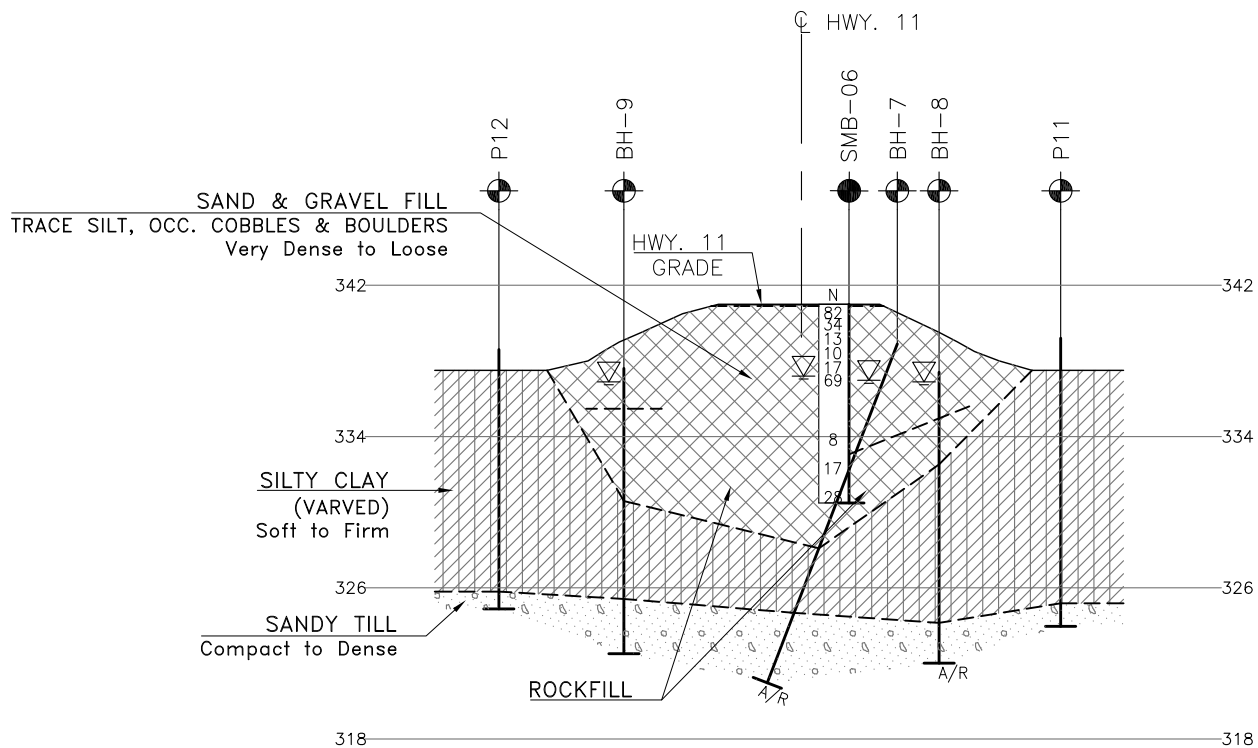
BH-8	337.4	5 391 812.2	284 019.7
BH-9	337.6	5 391 816.9	284 003.6
P10	336.9	5 391 852.4	284 016.8
P11	339.2	5 391 811.1	284 026.1
P12	338.6	5 391 818.0	283 997.1

[illegible]

SECTION ALONG WEST ABUTMENT



SECTION ALONG EAST ABUTMENT



SECTION ALONG A-A

