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FOUNDATION INVESTIGATION AND DESIGN REPORT
HWY 11/17N MACKENZIE RIVER BRIDGE
G.W.P. 6120-03-00, SITE: 48C-344 1 (EBL) & 48C-344-2 (WBL)

GEOCRES Number:

52A-142

Report to

Ministry of Transportation Ontario

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the site of a proposed bridge structure over MacKenzie River, Highway 11/17 near Thunder Bay, Ontario. The new twin structures will carry the east (EBL) and west bound (WBL) lanes of the proposed realigned Highway 11/17 over the MacKenzie River.

The purpose of the 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 and cross-sections, 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 present investigation. Thurber carried out the investigation for the Ministry of Transportation Ontario under Agreement Number 6008-E-0017.

2 SITE DESCRIPTION

The site of the proposed crossing lies on the future new alignment of Highway 11/17 approximately 250 m north of the existing crossing. The site is approximately 30 km east of Thunder Bay.

Geologically, the site area is located within the physiographic region known as the Canadian Shield, characterized by Pre-Cambrian bedrock typically occurring as rounded knobs and ridges with muskeg deposits developed in poorly drained depressions.

Locally, the MacKenzie River runs in a valley approximately 25 m deep and 200 m wide and the river flows to the south. On the west side of the river the valley slope consists of bedrock with a shallow cover of sand and silt overburden. Exposed bedrock is visible on the Hydro right of way that forms the southern boundary of the highway right of way. On the east side, the bedrock is covered by a thicker deposit of sand and silty sand till that is visible as terraces in the cleared Hydro right-of-way. The eroded surfaces on the east side expose numerous cobbles and boulders. The river bed appears to be composed largely of boulders though there is exposed bedrock on the banks nearby.

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The site is heavily treed and there is no development immediately to the west of the site. MacKenzie Heights Road crosses the right-of-way 300 m east of the river and one private residence on the west side of the road lies within the proposed ROW. Several other residences lie north of the right-of-way. On the existing highway, there is one private residence on the north side 200 m east of the river and a commercial property (apparently not in operation) 200 m west of the river.

Photographs of the site and its surroundings are included in Appendix F.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project was carried out during the period of May 26 to June 1, 2009. Twelve boreholes numbered 09-1 to 09-12 were drilled near each foundation element of the multi-span twin structure to depths ranging from 0.5m to 8.4 m. The approximate locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix E.

Borehole locations were selected in the field during a site visit that was carried out jointly by staff from the Ministry, McCormick Rankin Corporation and Thurber. The locations selected represented the best judgement of the team as to where the foundation elements would be located. It should be noted that the GA for the bridges was developed after the borehole locations had been selected. The borehole locations were surveyed for Thurber by J.D. Barnes after completion of the drilling.

A combination of hollow-stem auger drilling and diamond coring techniques were used to advance the boreholes. Samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) in the overburden soils. All the auger boreholes were extended to refusal on bedrock on inferred bedrock. Based on preliminary discussions with MTO, it was decided that the borehole at the foundation elements would be cored at least 1.5 m into bedrock by diamond coring techniques, unless situated on a bedrock outcrop.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Five standpipe piezometers consisting of 19 mm PVC pipe with a slotted screen were installed and enclosed in filter sand to permit longer term groundwater level monitoring. The locations and completion details of the piezometers are shown in Table 3.1. The boreholes in which no piezometers were installed were backfilled with auger cuttings. The borehole completion details are shown in Table 3.1.

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Table 3.1 – Borehole Completion Details

Borehole Location	Piezometer Tip Depth/ Elevation (m)	Completion Details
09-01	None Installed	Backfilled with cuttings to ground surface.
09-02	None Installed	Backfilled with cuttings to ground surface.
09-03	5.4 / 220.4	Piezometer with 1.5 m slotted screen installed with sand filter from 5.4 m to 3.0 m and cuttings from 3.0 m to ground surface.
09-04	None Installed	Backfilled with cuttings to ground surface.
09-05	None Installed	Backfilled with cuttings to ground surface.
09-06	None Installed	Backfilled with cuttings to ground surface.
09-07	None Installed	Backfilled with cuttings to ground surface.
09-08	3.1 / 232.0	Piezometer with 1.5 m slotted screen installed with sand filter from 3.1 m to 1.2 m and cuttings from 1.2 m to ground surface.
09-09	6.7 / 218.6	Piezometer with 1.5 m slotted screen installed with sand filter from 6.7 m to 4.7 m and cuttings from 4.7 m to ground surface.
09-10	4.4 / 217.4	Piezometer with 1.5 m slotted screen installed with sand filter from 4.4 m to 1.8 m and cuttings from 1.8 m to ground surface.
09-11	2.9 / 226.9	Piezometer with 1.5 m slotted screen installed with sand filter from 2.9 m to 0.9 m and cuttings from 0.9 m to ground surface.
09-12	None Installed	Backfilled with cuttings to ground surface.

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 on site prior to transportation to laboratory.

4 LABORATORY TESTING

All the recovered soil samples were subjected to Visual Identification (VI) and natural moisture content determination. Selected samples were subjected to gradation analysis. The results of this testing are shown on the Record of Borehole Sheets in Appendix A. The results of point load tests on rock cores retrieved from the boreholes are shown in Table B1 in Appendix B.

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5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A. Details of the encountered soil and rock stratigraphy are presented in this appendix and on the "Borehole Locations and Soil Strata" drawing in Appendix E. 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.

In general, the site is underlain by 0.5 m to 7.8 m of overburden soils overlying Granite bedrock. The overburden soils generally consist of a thin layer or organic peat layer, overlying sand, overlying granite bedrock.

5.1 Peat

Organic black peat was encountered at the ground surface in 8 of 12 boreholes advanced. The thickness of the organic layers ranged from 75 to 150 mm.

5.2 Sand Fill

A layer of sand fill was recorded in Boreholes 09-03 and 09-10 from ground surface to 2.3 and 1.4 m depth respectively. The underside of the fill is at elevation 223.5 and 220.3 respectively. This sand fill was placed prior to advancing the boreholes to create a working platform for the drill rig to set up safely. The material was excavated from beside the borehole locations.

5.3 Sand

From ground surface or immediately underlying the peat, a deposit of sand was encountered in the boreholes. This layer generally consists of sand with trace to some silt and trace gravel. Cobbles and boulders are present in this layer. The sand deposit was found overlying bedrock, extending to depths ranging from 0.5 to 7.8 m. The underside of this cohesionless layer is at elevation ranging from 217.4 to 236.0.

Standard penetration tests (SPT) in this deposit gave 'N' values generally ranging from 4 to 43 indicating a loose to dense relative density. This layer is generally in a compact state. Occasional SPT refusal values of greater than 100 blows for 0.3 m penetration indicating a very dense relative density were encountered where the sand contained cobbles or boulders. Figures B1 to B3 present grain size distribution curves for samples from this layer. The gradation test results are summarised in Table 5.1.

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TABLE 5.1 – Sand Gradation

	%
Gravel	0 - 15
Sand	49 - 94
Silt	5 - 50
Clay	1 - 6
Silt & Clay	5 - 32

Moisture content of this layer ranged between 2% to 32%.

5.4 Silty Sand / Silt and Sand Till

A 1.7 m layer of silt and sand till was encountered at BH09-03 (Elev. 222.1 to 220.4) overlying bedrock. A 2.1 m layer of silty sand till was encountered at BH09-09 (Elev. 220.9 to 218.9) overlying bedrock.

Two selected samples from this layer were subjected to grain size distribution tests and the results are presented in Figures B4 and B5. The gradation results are summarised in Table 5.2.

TABLE 5.2 – Silt and Sand Till Gradation

	%
Gravel	2 - 9
Sand	38 - 59
Silt	34 - 48
Clay	3 - 13

SPT 'N' values of greater than 100 blows for 0.3 m penetration were achieved in this layer in both boreholes indicating a very dense state.

The moisture content of samples from these zones ranged from 11% to 14%.

5.5 Bedrock

The overburden soils described above are underlain by granite bedrock. Bedrock was proved by coring at each proposed abutment and pier, except for the west bound east abutment, where bedrock outcrop is visible close to ground surface. Table 5.3 summarizes the bedrock depth and the elevations to the top of bedrock at the foundation elements.

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TABLE 5.3 – Depth to Bedrock at Foundation Elements

Location	BH Number	Depth to Bedrock (m)	Top of Bedrock Elevation (m)
WBL East Abutment	09-01 / 09-02	1.0 / 0.5	234.9 / 234.5
WBL East Pier	09-3	5.4	220.4
WBL West Pier	09-4	0.7	222.6
WBL West Abutment	09-5 / 09-6	1.4 / 1.7	229.2 / 236.0
EBL East Abutment	09-8	3.1	232.0
EBL East Pier	09-9	6.9	218.9
EBL West Pier	09-10	4.4	217.4
EBL West Abutment	09-11 / 09-12	2.9 / 4.1	226.9 / 233.7

Bedrock is described as fresh. Its colour is red and black and it contains occasional quartz intrusions.

Core recovery in the bedrock ranged between 95% and 100%. The RQD values ranged from 80% to 100% indicating good to excellent rock quality.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, was generally low ranging from 0 to 2. Occasional sub-vertical joints were encountered and they were mostly tight with no visible infilling or secondary weathering material.

The estimated unconfined compressive strength of the rock ranged ^{from} between 150 and 257 MPa indicating a very strong rock. These estimated rock strength values are derived ^{or varied} from point load tests that were conducted on rock cores recovered from the boreholes. A summary of the Point Load Test Results is presented in Table B1 in Appendix B.

5.6 Water Levels

A standpipe piezometer was installed at each foundation element except for the west bound lane west abutment. Water levels were measured after completion of drilling prior to demobilization from the site and during a return site visit at a later date. The water level readings are presented in Table 5.2.

Table 5.4: Water Level Measurements

Date	BH 09-3		BH 09-8		BH 09-9		BH 09-10		BH 09-11	
	Depth (m)	Elev. (m)	Depth (m)	Elev. (m)	Depth (m)	Elev. (m)	Depth (m)	Elev. (m)	Depth (m)	Elev. (m)
May 30, 2009	3.9	221.9	2.6	232.5	4.5	221.2	2.2	219.6	2.7	227.2
June 1, 2009	4.1	221.7	2.4	232.7	4.8	220.9	2.3	219.5	2.6	227.3

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Based on these observations, local groundwater levels down the valley slope range from 232.7 to 219.5 m. Groundwater observations at this site are short term and levels are expected to fluctuate seasonally and after severe weather events.

6 MISCELLANEOUS

Thunder Bay Testing and Engineering Limited of Thunder Bay, Ontario supplied a track mounted CME 750 drill rig and conducted the drilling, sampling and in-situ testing operations.

The drilling and sampling operations in the field were supervised on a full time basis by Ms. Eckie Siu of Thurber.

Mr. Alastair E. Gorman, P.Eng., and Mr. Tony Harte, directed the field operations and prepared the report.

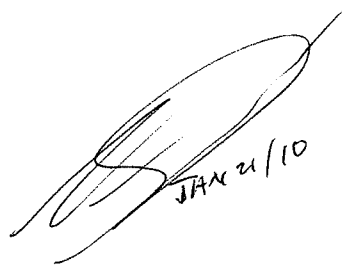
Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations projects, reviewed the report.

Thurber Engineering Ltd.



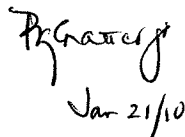
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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 GENERAL

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical design recommendations to assist the design team to select and design a suitable foundation system and approach embankments for the proposed structure.

Preliminary design documents indicate that Highway 11/17N will cross over the MacKenzie River via twin multi-span structures. The preliminary GA drawing indicates an approximate span of 180 m between the abutments. The finished grade of Highway 11/17N over the abutments is shown below, based on the GA drawing:

Structure	West Abutment	East Abutment
WBL	241.5	242.4
EBL	241.1	242.0

Each of the four approaches requires only a short fill before the vertical alignment goes into a cut.

The discussion and recommendations presented in this report are based on our understanding of the project and on the factual data obtained in the course of this investigation.

8 STRUCTURE FOUNDATIONS

The results of the investigation show that the site is underlain by Pre-Cambrian Granite at elevations ranging from Elevation 236.0 at the west approach, WBL (Borehole 09-6) to Elevation 217.4 at the west pier, EBL (Boreholes 09-10). The significant variations are due to the fact that the site spans a valley incised into the bedrock. The overburden on the west valley slope consists of comparatively thin sand and silt whereas on the east slope there are thicker deposits of sand and gravel with cobbles and boulders.

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In the preparation of geotechnical design recommendations, consideration was given to the following foundation types:

- Spread footings on native soil
- Spread footings bearing on bedrock.
- Steel H-piles driven to bedrock

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

8.1 Spread Footings on Soil

The possibility of designing spread footings bearing on the undisturbed native soil has been considered. The recommended geotechnical resistances at various foundation elements are shown in Table 8.1.

Table 8.1 - Geotechnical Resistance on Soil

Foundation Element	Elevation	ULS _r	SLS
WBL East Abutment	Not applicable – shallow bedrock		
WBL East Pier	223.0 <i>flow far from slope</i>	750	500
WBL West Pier	Not applicable – shallow bedrock		
WBL West Abutment	Not applicable – shallow bedrock		
EBL East Abutment	234.0	250	150
EBL East Pier	222.5	250	150
	221.0	750	500
EBL West Pier	218.5	750	500
EBL West Abutment	Support on bedrock recommended		

When considering spread footings bearing on undisturbed native soil, the following factors must be borne in mind:

- i. The stratigraphy is not well defined on the basis of the site investigation and field testing that was carried out. There is a risk that the soil stratigraphy at the foundation elements will not be consistent across the foundation element and that significant input and possible redesign will be required during the construction phase.
- ii. If the pier foundations bear on soil, the overall project design must incorporate measures to protect the foundations from local scour under the design storm conditions and must also contain measures to prevent widening or migration of the river channel.

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- iii. Some foundations may be on soil but others will be on bedrock, leading to increased differential settlement between adjacent foundations.

On account of the issues set out above, spread footing on soil are not the recommended foundation option for this site.

8.2 Spread Footings on Bedrock

The bedrock encountered at this site forms a suitable bearing stratum for spread footings. The top of bedrock elevations established in the course of the investigation are shown in Table 5.1. *5.3*

8.2.1 Geotechnical Vertical Resistance

Footings bearing directly on the bedrock may be designed on the basis of a factored geotechnical resistance at ULS of 5,000 kPa. The SLS condition will not govern for a footing bearing on bedrock. *where? Elevation? Found.*

All broken rock and other loose material must be removed from the bearing surface prior to placement of concrete. In the case of over-excavation, concrete fill may be used to reinstate the bearing surface to the design founding elevation.

Uneven rock surfaces or over-excavation may be made up using mass concrete fill. Where mass concrete fill is required, it is recommended that the footing be designed on the basis of a geotechnical resistance of 5,000 kPa and that the requirements for mass concrete be specified by the structural designer.

The SLS condition will not govern for a footing bearing on mass concrete as described herein.

A NSSP governing the placement of the mass concrete fill is included in Appendix D.

The stated bearing resistance is for vertical, concentric loads. In the case of eccentric or inclined loading, the geotechnical resistance must be calculated as illustrated in the CHBDC, 2000 Clause 6.7.3 and Clause 6.7.4.

8.2.2 Sliding Resistance

Initial calculations of the horizontal resistance may be carried out using a value of 0.7 for the ultimate friction factor of concrete poured on rock.

If the frictional component is insufficient, the horizontal resistance may be increased by dowelling into the rock mass. Dowels, in this application, are considered to provide only shear resistance. If vertical resistance in tension is required, rock anchors must be included in the design.

The dowel may be considered as acting as a fully embedded pile in the rock and hence will fail when the ultimate lateral resistance of the rock or grout is exceeded. Typically, dowels

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should be installed in holes drilled 1.2 m into the bedrock. With that depth of embedment, the performance of dowels in the range of 25M to 50M will be governed by structural considerations rather than geotechnical.

A Special Provision governing the installation and testing of dowels in rock is included in Appendix D.

8.2.3 Preparation of Bearing Surface on Bedrock

Where the surface of the bedrock within the footprint of the footing slopes at an inclination steeper than 5%, the rock must be excavated to a maximum slope of 5%. Following excavation, all rock shatter must be cleaned from the bearing surface. A NSSP to this effect is included in Appendix D.

8.3 Steel Piles Supported on Bedrock

Steel H-piles bearing on the bedrock can provide high bearing resistances as shown in Table 8.2, which shows the recommended structural resistance to use for various pile sections.

Table 8.2 – Resistance of Various Pile Sections on Bedrock

Pile Section	ULS _r (kN)
HP 310 X 110	2,000
HP 310 x 132	2,400
HP 310 X 152	2,750

The minimum length of pile driven into dense or very dense soil below the underside of the pile cap is 5 m. A review of the stratigraphy and the probable pile cap elevations indicates that, in general, this depth of pile embedment will not be available. While it is possible to install piles in sockets in the bedrock, this is not considered to be a cost effective solution for this site. Accordingly, the option of steel pile foundations has not been developed further.

8.4 Recommended Foundation

From geotechnical and cost effectiveness considerations, the recommended foundation system for this site is a spread footing bearing on bedrock.

Where the bedrock lies below the design underside of pile cap, mass concrete must be placed between the bedrock and the underside of the footing. The mass concrete must be detailed by the structural designer.



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8.5 Frost Cover

The depth of earth cover for frost protection at this site is 2.2 m. Frost penetration is not required for footings bearing on bedrock or mass concrete fill bearing on bedrock.

9 ABUTMENT CONSIDERATIONS

The geotechnical conditions encountered at this site are considered suitable for the design of conventional or semi-integral abutments. Due to the presence of bedrock at shallow depth, the site is not considered to be suitable for integral abutments.

10 BRIDGE APPROACHES

The GA drawing indicates that fill embankments will be required at the abutments and for a limited distance behind the abutments before the profiles enter cuts.

The overburden soils encountered at the site are shallow, cohesionless soils overlying bedrock that will provide stable foundations for approach embankments constructed with slopes at the following maximum inclinations:

- Rock fill – maximum inclination 1.25H:1V
- SSM or granular – maximum inclination 2H:1V

Where earth fill embankments are higher than 8 m, mid-height berms must be incorporated in the design. The berms must:

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

Where rock fill embankments are higher than 10 m, mid-height berms must be incorporated in the design. The berms must:

- extend for the length through which the embankment height exceeds 10 m
- be at least 2 m wide

Earth grading for the bridge approaches must be constructed in accordance with OPSD 200.020. Cut slopes may be formed at 2H:1V and embankments consisting of granular or SSM fill may be constructed with side slopes at 2H:1V from the point of view of stability.

Rock grading for the bridge approaches must be constructed in accordance with OPSD 201.020.

Earth fill embankment slopes and cut slopes must be provided with erosion protection in accordance with OPSS 572.

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11 EXCAVATION AND BACKFILL

11.1 General

All excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the native soils at this site are classified as Type 3 soils and temporary excavations must be sloped at 1H:1V.

Temporary excavation in sound bedrock may be carried out to near vertical slopes, though the resulting face may be prone to toppling or sliding wedge failures. Accordingly, a post-excavation inspection of the excavated slopes must be carried out and any potential unstable rock that is detected must be removed or stabilized.

If blasting is used to remove rock, it must be carried out in accordance with the Amendment to OPSS 120, August 1994.

11.2 Foundations

The excavation and backfilling for foundations must be carried out in accordance with SP 902S01.

12 GROUNDWATER CONTROL

Short term groundwater levels were recorded at depths of 0.0 to 4.8 m, corresponding to Elevation 237.1 to 219.5. These groundwater levels are generally within the cohesionless overburden soils and lie at 0.3 to 2.1 m above the bedrock surface, though greater distance above the bedrock may occur locally, in the spring or after heavy rainfall.

The presence of groundwater will not adversely impact the geotechnical resistance available in the bedrock but seepage may destabilize the sides of excavations and may carry sediment onto the prepared bedrock bearing surface.

Suitable systems are required to control groundwater seepage and surface runoff to prevent siltation of the bearing surface and to permit concrete to be placed in the dry. The design of these systems is the responsibility of the Contractor, but suitable measures might include perimeter ditches and pumping from sumps.

A NSSP on groundwater control is included in Appendix D.

Any accumulation of water from the base of the excavation must be removed prior to placing concrete or compacting granular fill. Placement of concrete or compacting of fill must be done in the dry.

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13 RETAINED SOIL SYSTEMS

Retained soil system (RSS) walls may be used subject to the requirements presented in this section.

RSS walls must be specified to be "High Performance" and "High Appearance". The contract drawings must include information on the longitudinal alignment of the wall in plan, the top and base elevations of the wall in profile, cross-sectional space constraints and an NSSP for the RSS wall.

13.1 Foundation

The performance of an RSS is dependent, among other factors, on the characteristics of its foundation. Failure to provide an adequate foundation may lead to settlement and distortion of the RSS and, in severe cases, to possible failure of the system. The foundation of the entire RSS mass must be considered, i.e. from the face of the wall to the furthest extent of the reinforcement.

At this site, provided the RSS mass is founded at the level of the dense native soil, or lower, geotechnical resistances of 250 kPa at SLS and 375 kPa at ULS_F may be used.

The RSS is a proprietary system and the supplier must design for internal, sliding and overturning stability and for any other failure modes identified by the supplier.

13.2 Global Stability

The global stability of a RSS wall constructed at this site, as described above, will not govern the design.

13.3 Review

If a RSS is selected for this site, the draft drawing should be sent to Thurber for review and commentary of the specific site conditions along the alignment of the wall.

14 BACKFILL TO ABUTMENTS

It is recommended that only granular backfill be used within the immediate approaches to the structure.

The backfill to the abutment walls should be in accordance with OPSS 902 as amended by Special Provision 902S01. Granular backfill should be placed to the extents shown in OPSD 3101.150.

All granular material should meet the specifications of Special Provision 110F13 "Amendment to OPSS 1010, March 1993". Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with SP 105S10.

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Settlement within the mass of the backfill is expected to be approximately 0.5% of the fill depth and to be essentially complete at the end of construction.

The design of the abutment should incorporate a subdrain as shown in OPSD 3101.150.

15 EARTH PRESSURE

For cases where backfill to the abutment is placed in accordance with OPSD 3101.150, as recommended, the lateral earth pressure will be governed by the properties of the material within the backfill limits shown in the OPSD, i.e. a line projected up at 1.5H:1V for granular backfill.

If the support system allows yielding of the wall (unrestrained system), active horizontal earth pressure may be used in the geotechnical design of the structure. If the support system does not allow yielding (restrained system), at-rest horizontal earth pressures should be used.

Earth pressures acting on the structure should be computed in accordance with Clause 6.9 of the CHBDC but generally are given by the expression:

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

P_h = horizontal pressure on the wall (kPa)

K = earth pressure coefficient (see table below)

γ = unit weight of retained soil (see table below)

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

q = value of any surcharge (kPa)

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I or at a depth of 1.7 m for Granular A or Granular B Type II.

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are given in Table 15.1.

The use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) will result in lower earth pressures acting on the wall. The use of a material with a low friction angle and lower passive pressure coefficient (e.g. Granular B Type I) will result in lower passive pressures acting on the wall. The designer must make a selection based on the design requirements of the structure.

The factors in the Table 15.1 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 C6.9.1 (a) in the Commentary to the CHBDC, 2000.

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Table 15.1 – Earth Pressure Coefficients

Wall Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A or OPSS Granular B Type II <i>fine drag</i> $\phi = 35^\circ; \gamma = 22.8$ kN/m ³		OPSS Granular B Type I $\phi = 32^\circ; \gamma = 21.2$ kN/m ³		Rock Fill $\phi = 42^\circ; \gamma = 19.0$ kN/m ³	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	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	0.40*	0.31	0.48*	0.20	0.28*
At rest (Restrained Wall)	0.43	-	0.47	-	0.33	-
Passive (Movement Towards Soil)	3.70	-	3.30	-	5.0	-

* For wing walls.

16 SEISMIC CONSIDERATIONS

The site is treated as lying in Seismic Zone 0. Accordingly, seismic design is not required.

17 CONSTRUCTION CONCERNS

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

1. Variability of the Stratigraphy

Due to the fact that the site investigation was carried out before the general arrangement of the bridge was finalized, some foundation elements do not have a borehole located within their footprints. Accordingly, there is a risk associated with the variability of the stratigraphy and in particular the elevation of the bedrock surface.

The contract documents must make allowance for the possible variations in the quantities of such items as mass concrete fill and bedrock excavation.

The contract documents must contain a NSSP instructing the QVE to alert the Contract Administrator, and through him the design team, if the excavation for a footing base

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exposes conditions other than those indicated on the drawings. Suggested wording is provided in Appendix D.

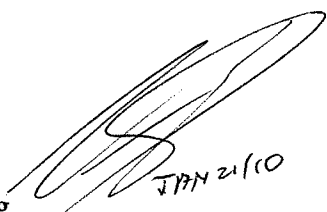
18 CLOSURE

Engineering analysis and preparation of the report were carried out by Mr. Alastair E. Gorman, P.Eng.

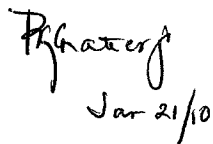
The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.

Alastair E. Gorman, P.Eng.,
Senior Foundations Engineer



JAN 21/10



JAN 21/10

P. K. Chatterji, P.Eng.,
Review Principal

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TOR

TOR	S	Rock
4 x 10	40	
4 x 16.5	60	6 m
4 x 16.5	60	6 m
	160 m	12 m

Appendix A

Record of Borehole Sheets

Summary

	S	B/R	Total
1	1m	—	1.0
2	0.5	—	0.5
3	5.4	1.6	7
4	0.7	1.5	2.2
5	1.4	1.6	3.0
6	1.7	—	1.7
7	7.8	—	7.8
8	3.1	1.5	4.6
9	6.9	1.5	8.4
10	4.4	1.5	5.9
11	2.9	1.5	4.4
12	4.1	—	4.1

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TOTAL

39.9 m



RECORD OF BOREHOLE No 09-01

1 OF 1

METRIC

G.W.P. 6120-03-00 LOCATION 5 378 440.9 N 383 213.9 E ORIGINATED BY ES
 HWY 17/11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SLL
 DATUM Geodetic DATE 2009-05-29 - 2009-05-29 CHECKED BY TH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
235.9								20	40	60	80	100					
0.0								20	40	60	80	100					
0.1	ORGANICS, black (75mm)																
235.6																	
0.3	SAND, some gravel, trace organics, trace roots and rootlets Dark Brown Moist		1	GS													
234.9	SAND, some silt, trace gravel Loose Brown Moist		1	SS	100/ .125		235										0 88 11 1
1.0																	
	END OF BOREHOLE AT 1.04m. ASSUMED BEDROCK. BOREHOLE OPEN AND DRY UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH AUGER CUTTINGS TO SURFACE.																

RECORD OF BOREHOLE No 09-02

1 OF 1

METRIC

G.W.P. 6120-03-00 LOCATION 5 378 438.2 N 383 206.9 E ORIGINATED BY ES
 HWY 17/11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SLL
 DATUM Geodetic DATE 2009-06-01 - 2009-06-01 CHECKED BY TH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
235.0														
0.0														
0.1	ORGANICS, black with roots and rootlets (75mm)		1	GS										GR SA SI CL
234.5														3 87 7 3
0.5	SAND, trace gravel Brown Moist													
	END OF BOREHOLE AT 0.5m. ASSUMED BEDROCK. BOREHOLE WAS CARRIED OUT BY SHOVEL EQUIPMENT DUE TO FLATNESS OF THE BEDROCK THROUGHOUT THE FOOTPRINT OF THE FOOTING LOCATION.													

RECORD OF BOREHOLE No 09-03

1 OF 1

METRIC

G.W.P. 6120-03-00 LOCATION 5 378 423.5 N 383 181.1 E ORIGINATED BY ES
 HWY 17/11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/BQ Rods COMPILED BY SLL
 DATUM Geodetic DATE 2009-05-27 - 2009-05-27 CHECKED BY TH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
								20 40 60 80 100								
225.8	SAND, some gravel, occasional cobbles and boulders Brown Moist (FILL) BACKFILL WAS PLACED FOR LEVELLING THE DRILL RIG ON THE BOREHOLE DUE TO SLOPE OF THE HILL.													GR SA SI CL		
0.0																
223.5	Gravelly SAND, occasional cobbles and boulders Very Dense Brown Moist		1	SS	100/ .100									2 38 48 12		
2.3																
222.1	SILT and SAND, some clay, trace gravel Very Dense Brown Moist (TILL) Cobbles		2	SS	100/ .100									2 38 48 12		
3.7																
220.4	GRANITE BEDROCK, fresh, reddish-black,		4	SS	100/ .100									RUN 1# TCR=100%, SCR=98%, RQD=98%, UCS=150.4MPa		
5.4																
218.9	END OF BOREHOLE AT 7.0m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) May 30, 09 3.9 221.9 Jun 01, 09 4.1 221.7		1	RUN												
7.0																

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

RECORD OF BOREHOLE No 09-04

1 OF 1

METRIC

G.W.P. 6120-03-00 LOCATION 5 378 388.6 N 383 102.6 E ORIGINATED BY ES
 HWY 17/11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/BQ Rods COMPILED BY SLL
 DATUM Geodetic DATE 2009-06-01 - 2009-06-01 CHECKED BY TH

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									
223.3								20	40	60	80	100					
0.0	ORGANICS, black (100mm)							○ UNCONFINED	+	FIELD VANE							
0.1	SAND, some gravel, trace silt, occasional cobbles							● QUICK TRIAXIAL	x	LAB VANE							
222.6	Brown																
0.7	Moist																
	GRANITE BEDROCK, fresh, reddish-black, horizontal joints at 0.9m and 1.3m. sub-horizontal joints at 1.5m, 1.6m and 2.1m. sub-vertical joints at 1.3 to 1.5m, and 1.8 to 1.9m.		1	RUN													
221.1																	
2.2	END OF BOREHOLE AT 2.2m. WATER LEVEL AT 2.2m UPON COMPLETION OF BOREHOLE. BOREHOLE BACKFILLED WITH CUTTINGS TO SURFACE.																

+ 3 . x 3 : Numbers refer to
Sensitivity



20
15 10 5
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 09-05

1 OF 1

METRIC

G.W.P. 6120-03-00 LOCATION 5 378 364.4 N 383 070.0 E ORIGINATED BY ES
HWY 17/11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/BQ Rods COMPILED BY SLL
DATUM Geodetic DATE 2009-06-01 - 2009-06-01 CHECKED BY TH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT Y kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
230.7							20 40 60 80 100							
0.0	Silty SAND, trace clay, trace gravel Loose Brown Moist		1	GS			○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE							
229.2			1	SS	4								4 68 26 2	
1.4	GRANITE BEDROCK, fresh, reddish-black, horizontal joints at 1.6m, 1.8m, 2.1m and 2.3m. rubble zone at 2.8m to 3.0m.		1	RUN									RUN 1# TCR=100%, SCR=80%, RQD=80%, UCS=154MPa	
227.7														
3.0	END OF BOREHOLE AT 3.0m. WATER LEVEL AT GROUND SURFACE UPON COMPLETION OF BOREHOLE. BOREHOLE BACKFILLED WITH AUGER CUTTINGS TO SURFACE.													

RECORD OF BOREHOLE No 09-06

1 OF 1

METRIC

G.W.P. 6120-03-00 LOCATION 5 378 358.2 N 383 051.0 E ORIGINATED BY ES
 HWY 17/11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SLL
 DATUM Geodetic DATE 2009-05-30 - 2009-05-30 CHECKED BY TH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa													
								○ UNCONFINED		+ FIELD VANE		● QUICK TRIAXIAL			x LAB VANE						
237.7							20	40	60	80	100										
0.0	ORGANICS, black, trace roots and rootlets: (150mm)		1	GS		237															
0.1	SAND, some gravel, trace silt, trace roots																				
237.0	Compact Dark Brown Moist		1	SS	14																
0.7	SAND, trace clay, trace silt, trace gravel																				
236.0	Compact Brown Moist		2	SS	100/																
1.7	END OF BOREHOLE AT 1.7m. ASSUMED BEDROCK. BOREHOLE OPEN AND WATER LEVEL AT 0.6m UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH AUGER CUTTINGS TO SURFACE.				.050																

RECORD OF BOREHOLE No 09-07

1 OF 1

METRIC

G.W.P. 6120-03-00 LOCATION 5 378 411.4 N 383 240.4 E ORIGINATED BY ES
 HWY 17/11 BOREHOLE TYPE Hollow Stem Augers/Trit-Cone/NW Casing COMPILED BY SLL
 DATUM Geodetic DATE 2009-05-27 - 2009-05-27 CHECKED BY TH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								20 40 60 80 100		PLASTIC LIMIT W _P				NATURAL MOISTURE CONTENT W		LIQUID LIMIT W _L	
								20 40 60 80 100		W _P				W		W _L	
242.1																	
0.0																	
0.1	ORGANICS: (100mm)																
	SAND, trace gravel, trace silt		1	GS													
	Compact																
	Brown		1	SS	29												
	Moist																
			2	SS	15												
			3	SS	22												
			4	SS	17												
			5	SS	27												
			6	SS	20												
			7	SS	16												
			8	SS	16												
			9	SS	17												
234.3			10	SS	100/												
7.8	END OF BOREHOLE AT 7.8m. ASSUMED BEDROCK. BOREHOLE OPEN AND DRY UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH AUGER CUTTINGS TO SURFACE.				.025												

RECORD OF BOREHOLE No 09-08

1 OF 1

METRIC

G.W.P. 6120-03-00 LOCATION 5 378 396.9 N 383 218.1 E ORIGINATED BY ES
 HWY 17/11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/BQ Casing COMPILED BY SLL
 DATUM Geodetic DATE 2009-05-26 - 2009-05-26 CHECKED BY TH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
235.1								20 40 60 80 100							
0.0	ORGANICS: (75mm)							○ UNCONFINED + FIELD VANE							
0.1	SAND, some gravel, occasional cobbles Dense Brown Moist		1	GS				● QUICK TRIAXIAL × LAB VANE							
			1	SS	37			20 40 60 80 100							
233.6															
1.4	SAND, some silt Compact Brown Moist		2	SS	17										
			3	SS	11										
232.0															
3.1	occasional cobbles and boulders		4	SS	100/										
	GRANITE BEDROCK, fresh, reddish-black, horizontal joint at 4.0m.		1	RUN	.075										
230.4															
4.6	END OF BOREHOLE AT 4.6m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) May 30, 09 2.6 232.5 Jun 01, 09 2.4 232.7														

RECORD OF BOREHOLE No 09-09

1 OF 1

METRIC

G.W.P. 6120-03-00 LOCATION 5 378 381.4 N 383 178.2 E ORIGINATED BY ES
HWY 17/11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/BQ Casing COMPILED BY SLL
DATUM Geodetic DATE 2009-05-26 - 2009-05-26 CHECKED BY TH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100						
225.7														
0.0 0.1	ORGANICS: (75mm)													
	SAND, with boulders and cobbles Very Dense Brown Moist		1	GS										
225.0														
0.7	SAND, some silt, occasional cobbles and boulders Very Dense to Compact Brown Moist Boulder (300mm Dia.)		1	SS	83/ 226		225							
			2	SS	14		224							
			3	SS	10		223							0 94 6 (SI+CL)
	occasional cobbles and boulders													
			4	SS	22		222							3 89 8 (SI+CL)
			5	SS	25		221							
220.9														
4.8	Silty SAND, trace gravel, trace clay, occasional bedrock fragments Very Dense Brown Moist (TILL)		6	SS	120/ 275		221							9 50 34 6
			7	SS	100/ .125		220							
			8	SS	100/ .125		219							3 59 35 3
218.9														
6.9	GRANITE BEDROCK, fresh, reddish-black,		1	RUN			218							RUN 1# TCR=100%, SCR=100%, RQD=100%, UCS=164MPa
217.3														
8.4	END OF BOREHOLE AT 8.4m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) May 30, 09 4.5 221.2 Jun 01, 09 4.8 220.9													

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

RECORD OF BOREHOLE No 09-10

1 OF 1

METRIC

G.W.P. 6120-03-00 LOCATION 5 378 348.2 N 383 102.0 E ORIGINATED BY ES
 HWY 17/11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/BQ Casing COMPILED BY SLL
 DATUM Geodetic DATE 2009-05-29 - 2009-05-30 CHECKED BY TH

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							
221.8								20 40 60 80 100							GR SA SI CL
0.0	SAND, some gravel, occasional cobbles, trace organics, roots and rootlets Brown Moist (FILL)														
220.3															
1.4	SAND, some gravel, occasional cobbles and boulders Compact Brown Moist		1	SS	30										
219.8															
2.0	Silty SAND, trace gravel Compact to Very Dense Grey Moist		2	SS	20										6 58 31 5
			3	SS	100/ .025										
217.9															
3.9	SAND, trace gravel, occasional cobbles Very Dense Grey Moist		4	SS	100/ .125										15 72 13 (SI+CL)
217.4															
4.4	GRANITE BEDROCK, fresh, reddish-black, sub-horizontal joints at 5.2 to 5.3m. horizontal joints at 5.8m.		1	RUN											RUN 1# TCR=100%, SCR=98%, RQD=100%, UCS=163MPa
215.8															
5.9	END OF BOREHOLE AT 5.9m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) May 30, 09 2.2 219.6 Jun 01, 09 2.3 219.5														

+ 3. X 3: Numbers refer to Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 09-11

1 OF 1

METRIC

G.W.P. 6120-03-00 LOCATION 5 378 327.5 N 383 075.6 E ORIGINATED BY ES
 HWY 17/11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/BQ Casing COMPILED BY SLL
 DATUM Geodetic DATE 2009-05-30 - 2009-05-30 CHECKED BY TH

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 (%)		
								20 40 60 80 100											
229.9																			
0.0	Silty SAND, trace to some gravel, trace clay, occasional cobbles		1	GS															
			1	SS	25														
	Dense to Compact Brown Moist		2	SS	43											10 58 28 4			
			3	SS	24											1 58 36 5			
226.9																			
2.9	GRANITE BEDROCK, fresh, very strong, reddish-black, sub-vertical joints at 3.0 to 3.1m. horizontal joints at 3.0m, 3.3m and 4.2m.		1	RUN												RUN 1# TCR=95%, SCR=92%, RQD=92%, UCS=165MPa			
225.4																			
4.4	END OF BOREHOLE AT 5.9m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) ELEV. (m) May 30, 09 2.7 227.2 Jun 01, 09 2.6 227.3																		

+ 3, X 3: Numbers refer to
Sensitivity


20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 09-12

1 OF 1

METRIC

G.W.P. 6120-03-00 LOCATION 5 378 317.1 N 383 048.6 E ORIGINATED BY ES
 HWY 17/11 BOREHOLE TYPE Hollow Stem Augers COMPILED BY SLL
 DATUM Geodetic DATE 2009-05-30 - 2009-05-30 CHECKED BY TH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										
								20 40 60 80 100										
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE										
							WATER CONTENT (%)											
							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p W W _L											
237.8	<div>ORGANICSblack, trace roots and rootlets: (150mm)</div> <div>SAND, trace gravel</div> <div>Compact</div> <div>Dark Brown</div> <div>Moist</div> <div>SAND and SILT, trace clay</div> <div>Compact</div> <div>Brown</div> <div>Moist</div>		1	GS		237												
0.0																		
0.1																		
237.1																		
0.7																		
					1		SS	11										
					2		SS	13										
					3		SS	14										
			4	SS	14													
233.7			5	SS	100/		234											
4.1	END OF BOREHOLE AT 4.1m. ASSUMED BEDROCK. BOREHOLE OPEN AND WATER LEVEL AT 3.8m UPON COMPLETION OF DRILLING. BOREHOLE BACKFILLED WITH AUGER CUTTINGS TO SURFACE.																	

+³ X³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

Appendix B
Laboratory Test Results

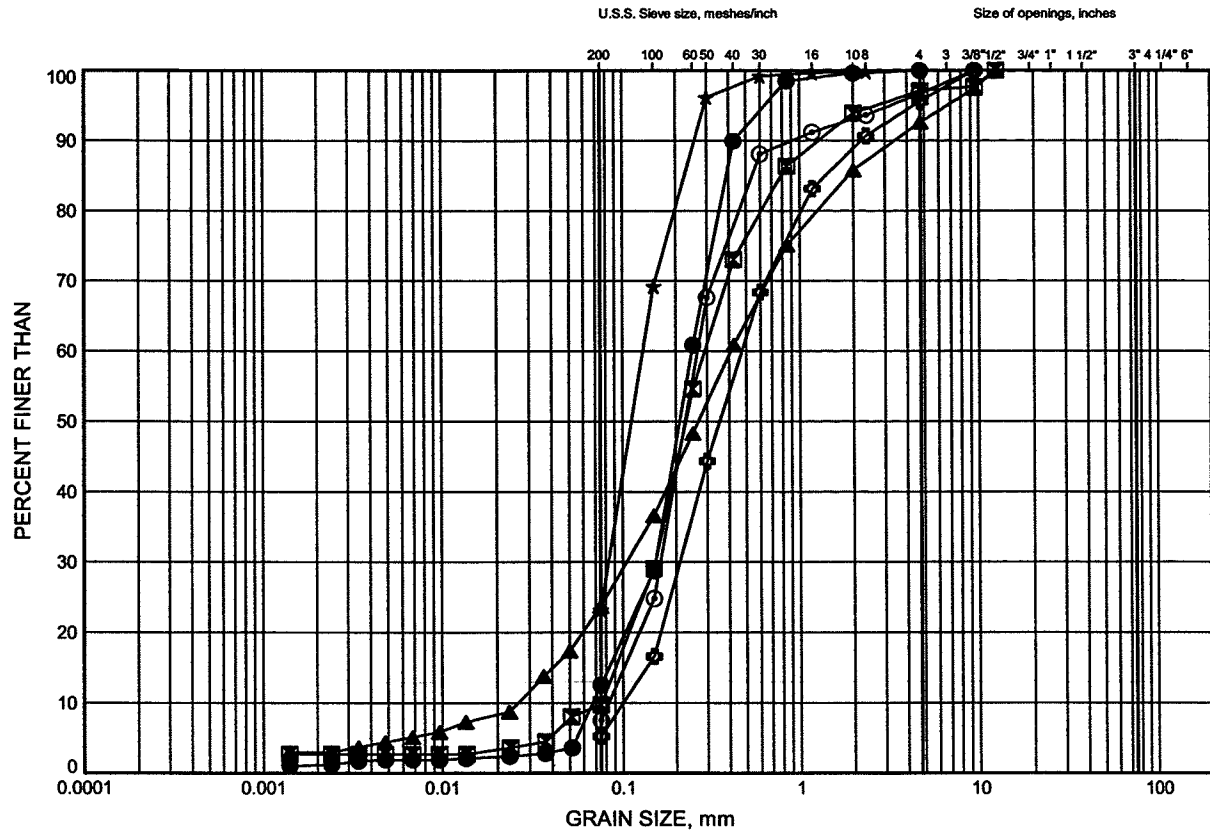
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Mackenzie River Bridge GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND



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

LEGEND

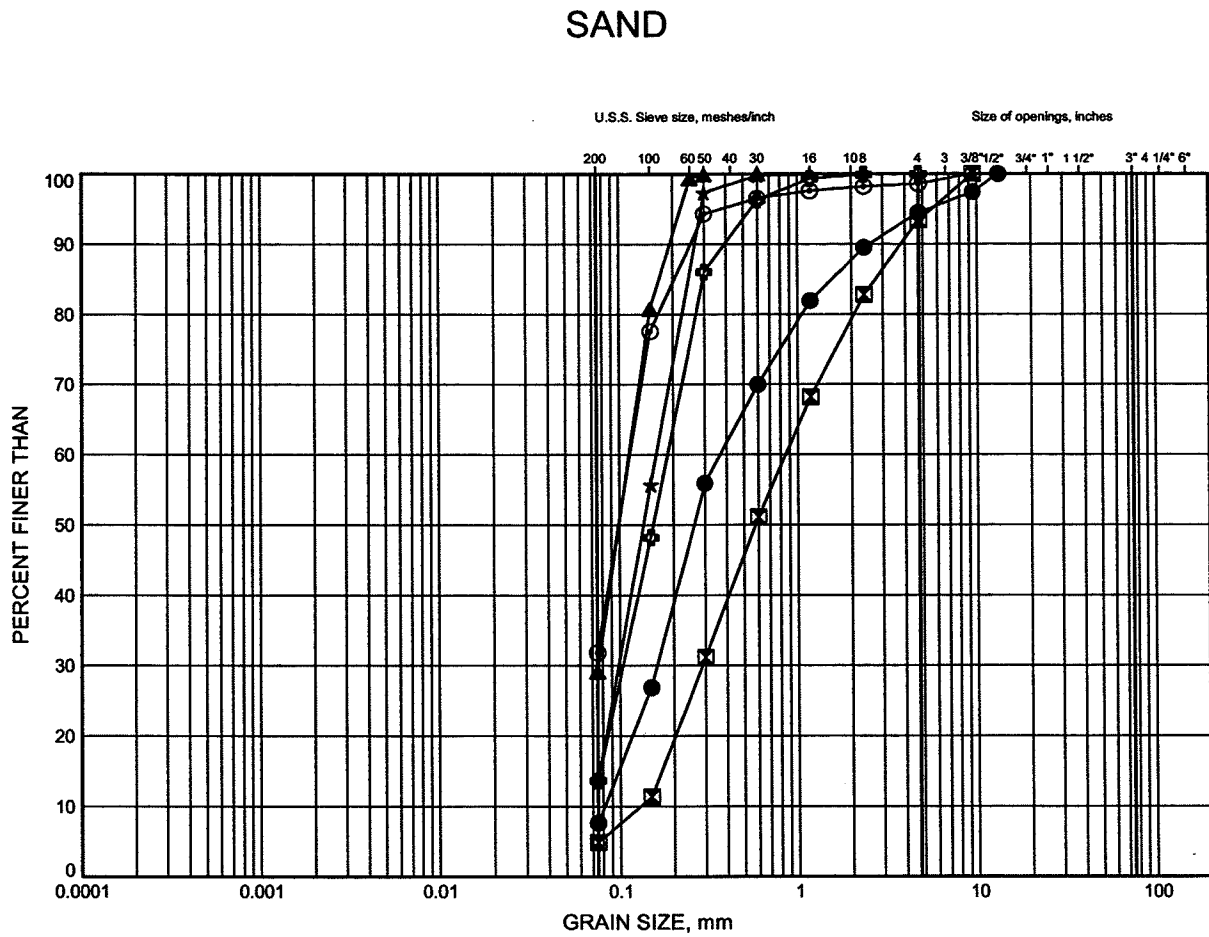
SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	09-01	0.91	235.02
⊠	09-02	0.30	234.65
▲	09-04	0.30	222.97
★	09-06	0.99	236.73
⊙	09-06	1.62	236.10
⊕	09-07	1.75	240.31



W.P.# 6120-03-00.....
Prepared By MFA.....
Checked By TJH.....

MacKenzie River Bridge GRAIN SIZE DISTRIBUTION

FIGURE B2



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	09-07	2.51	239.55
⊠	09-07	4.04	238.02
▲	09-07	6.32	235.74
★	09-07	7.09	234.97
⊙	09-08	1.75	233.33
⊗	09-08	2.51	232.57

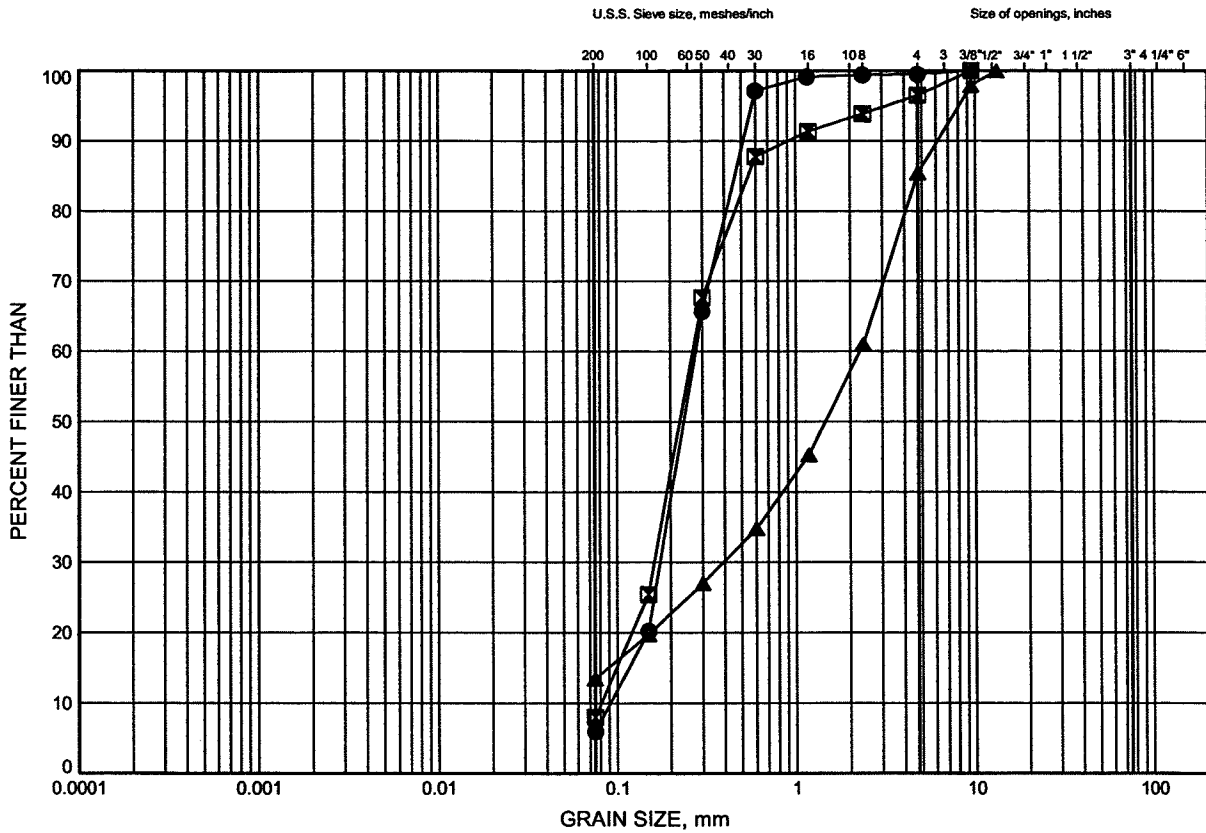


W.P.# .6120-03-00.....
Prepared By .MFA.....
Checked By .T.JH.....

MacKenzie River Bridge GRAIN SIZE DISTRIBUTION

FIGURE B3

SAND



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	09-09	2.51	223.22
■	09-09	3.28	222.45
▲	09-10	3.87	217.91

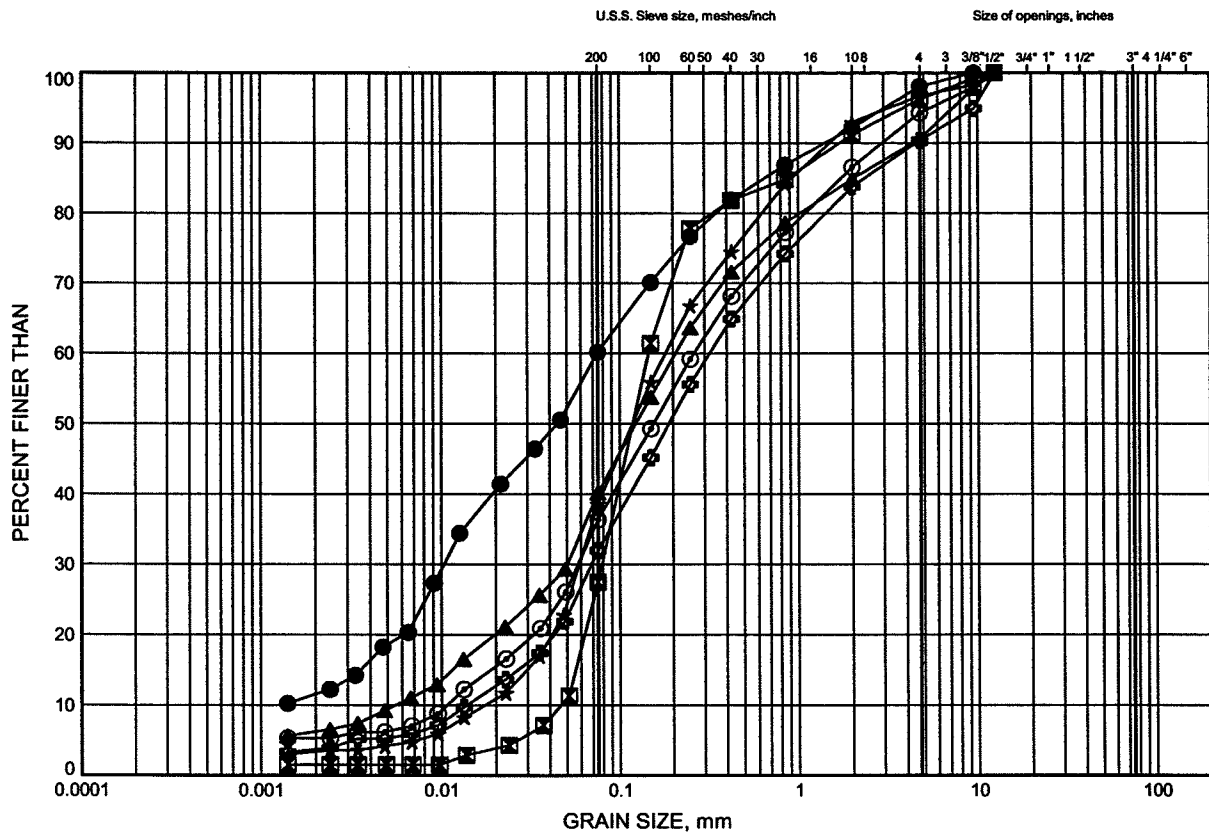


W.P.# 6120-03-00
Prepared By MFA
Checked By TJH

MacKenzie River Bridge GRAIN SIZE DISTRIBUTION

FIGURE B4

SILTY SAND / SILT AND SAND TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	09-03	3.86	221.96
⊠	09-05	1.07	229.59
▲	09-09	4.79	220.94
★	09-09	6.23	219.50
⊙	09-10	2.59	219.19
⊕	09-11	1.75	228.10

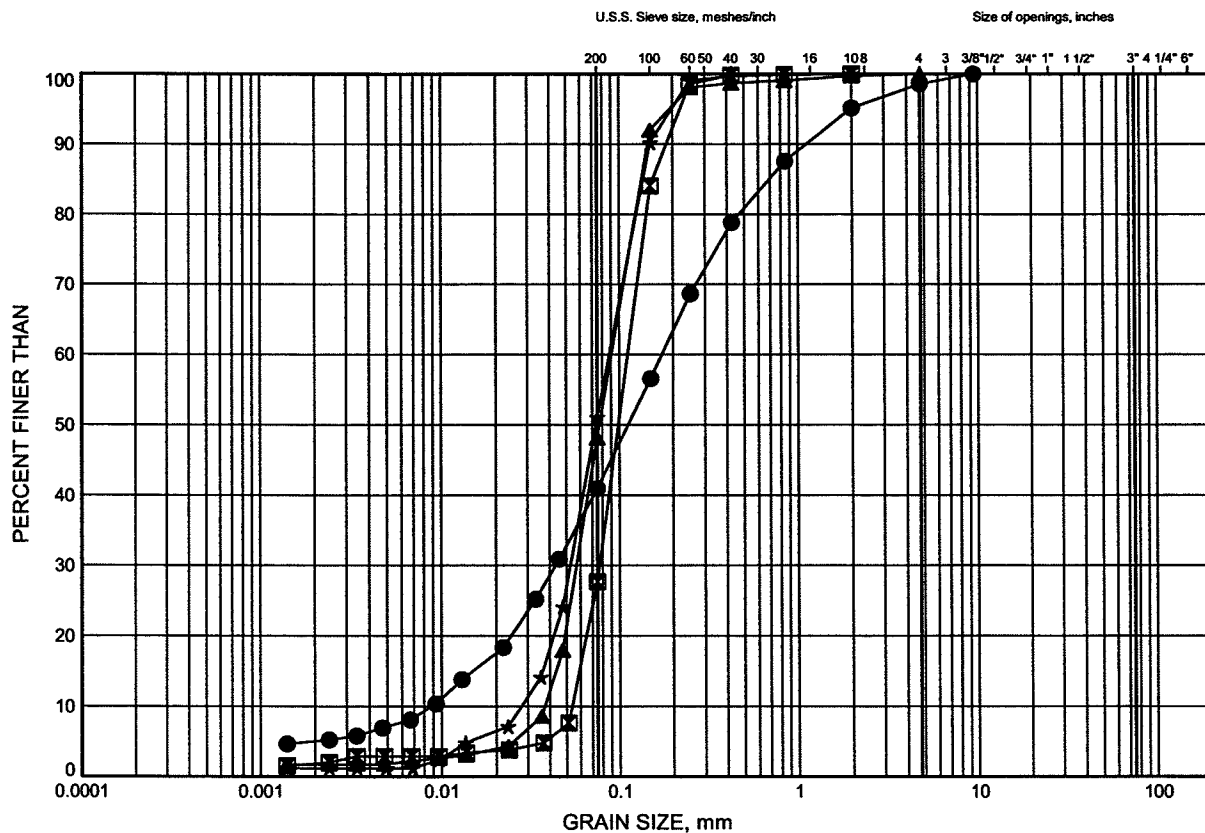


W.P.# 6120-03-00.....
Prepared By MFA.....
Checked By TJH.....

MacKenzie River Bridge GRAIN SIZE DISTRIBUTION

FIGURE B5

SILTY SAND / SILT AND SAND TILL



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

LEGEND

SYMBOL	BOREHOLE	DEPTH (m)	ELEV. (m)
●	09-11	2.51	227.34
⊠	09-12	1.75	236.06
▲	09-12	2.51	235.30
★	09-12	3.28	234.53



W.P.# 6120-03-00.....
Prepared By MFA.....
Checked By TJH.....

Appendix C

Foundation Comparison

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COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Foundation Element	Footings on Soil	Footings on Bedrock	Steel H-Piles
Abutments and Piers	<p>Advantages:</p> <ul style="list-style-type: none"> i. Conventional construction technique for most contractors. ii. Relatively little excavation. iii. Normally the most cost effective option. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Pier footings, in particular, may be at risk from scour when the river is at flood stage. ii. Pier footings, in particular, may be at risk from undermining if the river course changes over time. iii. Due to the stratigraphy, some footings will necessarily be on bedrock, leading to an increased potential for differential settlement. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance is available. ii. Resistant to scour and erosion. iii. Possible to keep all foundations on similar founding stratum, reducing the risk of differential settlement. iv. Lower unit cost compared to pile foundations. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Requires additional earth excavation and some rock excavation. ii. Requires mass concrete below some footings. iii. More costly than footings on soil. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance available by seating piles on bedrock. ii. Comparatively short abutment stem. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit cost compared to footings. ii. In general there is insufficient overburden to drive the piles, necessitating sockets in the bedrock to secure the pile tips.
	NOT RECOMMENDED	RECOMMENDED	NOT RECOMMENDED

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Appendix D

Special Provisions

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The following Special Provisions are referenced in this report:

110F13

105S10

Amendment to OPSS 206, December 1993

902S01

903S01

DOWELS INTO ROCK –

Special Provision

March 1, 2001

1.0 GENERAL

1.1 Scope

The work for the above noted tender item shall be in accordance with OPSS 904, including all special provision, except as extended herein. This document specifies additional requirements for the supply, installation and testing of Dowels into Rock for the pier footing.

1.2 Instructions to Contractor

1.2.1 These instructions are to be read in conjunction with the Contract Drawings.

1.2.2 A total of 2 test Dowels into Rock are required for the Dowels into Rock at the pier.

1.2.3 Dowels into rock at the pier shall be installed prior to unwatering the structure excavation. Dowels shall extend through tremie concrete and into sound bedrock to the specified embedment depth.

1.3 Qualifications

1.3.1 **Qualifications of Staff from Contractor or Sub-Contractor Completing Work for the Dowels into Rock:** All work shall be performed under the direction of personnel experienced with all aspects associated with the underwater installation of Dowels into Rock. Such experience shall have been obtained within the preceding five (5) years on projects of similar nature and scope to the work required for this project.

1.3.2 **Qualifications of the Quality Verification Engineer:** A resume of the work experience of the Quality Verification Engineer shall be submitted to the Contract Administrator for record purposes. The Quality Verification Engineer shall be a Professional Engineer licensed in the Province of Ontario having a minimum of five years of experience on projects of similar nature and scope to the work required for this project.

1.3.3 **Qualifications of the Design Engineer:** A resume of the work experience of the Design Engineer shall be submitted to the Contract Administrator for record purposes. The Design Engineer shall be a Professional Engineer licensed in the Province of Ontario having a minimum of five years of experience of projects of similar nature and scope to the work required for this project.

1.4 Responsibilities of the Contractor

- 1.4.1 The Contractor shall prove the allowable bond stress by tests of the Dowels into Rock on non-production Dowels into Rock.
- 1.4.2 The Contractor shall supply equipment, materials and skilled personnel to install production Dowels into Rock and conduct the specified acceptance tests. It shall be the responsibility of the Contractor to constantly monitor the acceptance tests, maintain specified test loads and record test measurements as specified by the Contract Administrator.
- 1.4.3 The Contractor is responsible for materials and workmanship. Any remedial measures, required because of defects in materials or workmanship, shall be completed by the Contractor at no cost to the Owner.
- 1.4.4 The Contractor shall submit 4 copies of all Working Drawings to the Contract Administrator as outlined in Section 1.6.

1.5 Definitions

- 1.5.1 Dowels into Rock: reinforcing steel bar and non-shrink grout.
- 1.5.2 Design Engineer: An Engineer who has a minimum of five (5) years experience in all aspects associated with the underwater installation of Dowels into Rock, including drilling, underwater grouting and doweling work. The Design Engineer shall be retained by the Contractor to design various components for the installation and testing for the Dowels into Rock.
- 1.5.3 Quality Verification Engineer: An Engineer who has a minimum of five (5) years experience in all aspects associated with the underwater installation of Dowels into Rock, including drilling, underwater grouting and doweling work. The Quality Verification Engineer shall be retained by the Contractor to ensure conformance with the contract documents and issue certificate(s) of conformance.

1.6 Submissions and Working Drawings

- 1.6.1 Working Drawings shall consist of drawings, testing and installation records, procedures and reports, and work plans.
- 1.6.2 The Contractor shall submit Working Drawings to the Contract Administrator as follows:
 - All Working Drawings that include drawing, testing and installation procedures and reports, and work plans shall be sealed and signed by the Design Engineer.
 - All Working Drawings that include testing and installation results and reports shall be signed and sealed by the Quality Verification Engineer.

- 1.6.3 Upon completion of testing or installation and testing for each component, the Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by a Quality Verification Engineer. The Certificate shall state that the work has been carried out in conformance with the Working Drawings and in general conformance with the contract documents.
- 1.6.4 Working Drawings consisting of testing an installation records and reports shall be submitted four days after completion of testing and installation. All other Working Drawings shall be submitted two weeks prior to construction.
- 1.6.5 Working Drawings to be submitted include the following with further details outlined in the remainder of this specification:
- Design calculations, specifications and shop drawings covering all aspects of fabrication, installation and acceptance testing of Dowels into Rock.
 - Test results verifying the 28 day strength of non-shrink grout.
 - The method for constructing of the holes, maintaining the holes, and placing reinforcing steel bars, grout and other materials in the holes, including casing sizes, bit sizes and tremie grouting methods.
 - The procedures to verify hole length. Records of measurements that verify the hole length.
 - Records of all drilling procedures, rock conditions encountered, and installation times.
 - Test procedures for Dowels into Rock.
 - Drawings and design calculations for a suitable reaction system for the applied test loads.
 - Records of vertical and horizontal movements of the reaction system, and elongation of the reinforcing steel bar.
 - Drawings and details for reference system arrangement.
 - Current calibration curves shall be provided for all gauges.
 - Complete test records for all tests including plots of dowel movement versus dowel load, dowel load versus time, and dowel movement versus time.
 - Remedial measures for unacceptable stressing results.

1.7 Subsurface Conditions

- 1.7.1 Rock and groundwater conditions are described in the Foundation Investigation Report for this Contract.

2.0 MATERIALS

The non-shrink grout shall be an approved DSM 9.10.35 non-shrink grout. The anti-washout agent shall be used with the non-shrink grout for the Dowels into Rock. The anti-washout agent shall be one of the following proprietary products:

Sikament 100 SC Anti-washout additive for underwater concrete/grouts
Sika Canada Inc.
970 Verbena Road
Mississauga, Ontario
L5T 1T6, Canada

Mr. Greg Dolenc
Phone: 416-795-3177
Mobile: 416-573-7223

Rheomac UW 450 Liquid anti-washout admixture
Masterbuilder Technologies
1800 Clark Boulevard
Brampton, Ontario
L6T 4M7, Canada

Mr. Eliseo Conciatori
Phone: 905-792-2012
Mobile: 416-567-7665

The Contractor shall provide the following information from the manufacturer for non-shrink grout and anti-washout agent:

- Data sheets for the non-shrink grout and anti-washout agent,
- Technical information that proves that the non-shrink grout and anti-washout agent are compatible, and
- installation procedures

3.0 EQUIPMENT

3.1 General

- 3.1.1 All equipment for the installation of the Dowels into Rock shall be suitable for the intended purposes and capable of working on the site under the prevailing access and clearance conditions.

- 3.1.2 The equipment shall not cause damage to the reinforcing steel bars.

4.0 INSTALLATION

All work for the installation of Dowels into Rock shall be inspected by the Quality Verification Engineer.

4.1 Construction of Holes

- 4.1.1 The sides and end of the hole shall not be disturbed. The Contractor shall submit Working Drawings to the Contract Administrator that include the method for constructing of the holes, maintaining the holes, and placing reinforcing steel bar, grout and other materials in the holes. All excavated material shall be removed from the site.
- 4.1.2 The hole diameters and hole length for this project are as specified on the Contract Drawings. Prior to commencing drilling operations, the Contractor shall submit Working Drawings to the Contract Administrator outlining devised procedures to verify hole length. The Contractor shall submit Working Drawings that include drilling operations records to the Contract Administrator that include the above noted records.
- 4.1.3 At all times, the Contractor shall keep a record of all drilling procedures, rock conditions encountered, and installation times. The Contractor shall submit Working Drawings to the Contract Administrator that include the above noted records.

4.2 Installation of Reinforcing Steel Bar

- 4.2.1 Reinforcing steel bar shall be installed in strict accordance with the Contract Drawings and installation procedures.
- 4.2.2 Centering devices shall be provided to ensure that the reinforcing steel bar is located centrally in the hole.
- 4.2.3 Dowels into Rock at the pier shall be installed prior to unwatering the structure excavation. Dowels shall extend through the tremie concrete for the pier footing and into sound bedrock.
- 4.2.4 Reinforcing steel bar shall be installed after the dowel hole has been filled with non-shrink grout.

4.3 Grout and Anti-Washout Agent

- 4.3.1 The non-shrink grout shall entirely fill the annular space between the reinforcing steel bar and side for the dowel hole.
- 4.3.2 The placement of grout for the test Dowels into Rock shall be identical to the production Dowels into Rock.

- 4.3.3 Anti-washout agent shall be used in accordance with the specifications of the manufacturer.
- 4.3.4 Non-shrink grout shall be placed into the dowel hole using tremie placement methods.

5.0 TESTING REQUIREMENTS

All work for the testing of Dowels into Rock shall be inspected by the Quality Verification Engineer.

5.1 General Testing Requirements

- 5.1.1 Refer to the attached Instructions to Contractor and the Contract Drawings for specific test details.
- 5.1.2 The Contractor shall install the number of Dowels into Rock specified in the contract documents for testing purposes. The purpose of the testing the Dowels into Rock is to prove the adequacy of the proposed anchor configuration and installation procedures under the site conditions, and to provide design parameters.
- 5.1.3 The equipment, labour and materials for test dowels shall be identical to Dowels into Rock at the pier. The Dowels into Rock for testing shall be 55M dowels grouted into 140 mm diameter holes filled with an approved non-shrink grout with a minimum 4,000 mm embedment into sound bedrock.
- 5.1.4 The Contractor shall submit Working Drawings that include proposed procedures for testing of the dowels into Rock to the Contract Administrator. Such testing shall be executed in strict accordance with the proposed procedures of the Contractor.
- 5.1.5 The Quality Verification Engineer shall supervise the testing of the Dowels into Rock. The Contractor will notify the Contract Administrator of the testing schedule at least 10 days prior to commencement of the testing program. Testing for Dowels into Rock shall be conducted concurrently, as scheduled by the Contract Administrator. The tests shall normally be conducted between 8:00 hrs and 20:00 hrs from Monday to Friday, unless otherwise directed by the Contract Administrator.
- 5.1.6 The Contractor shall supply materials and skilled personnel to conduct the tests for the Dowels into Rock. The equipment and materials shall be capable of stressing the Dowels into Rock to the specified loads. It shall be the responsibility of the Contractor to constantly monitor the test, maintain specified test loads and to record test measurements as specified by the Quality Verification Engineer.
- 5.1.7 The test site shall be restored to its pre-test condition. Reinforcing steel bars used in tests shall be cut down 25 mm below the top of the sound bedrock.

5.2 Testing Location

- 5.2.1 The Contractor shall remove all loose rock down to sound bedrock at the test location.
- 5.2.2 The test Dowels into Rock shall be constructed at locations specified by the Contract Administrator. The water depth at the location of the test shall be at least 0.5 m deep.
- 5.2.3 If site conditions dictate, changes to the test locations will be considered. The Contractor shall provide the Contract Administrator at least 2 days notice in writing of this operation.

5.3 Testing Equipment

- 5.3.1 The dowels into rock will be carried out generally in accordance with the prevailing requirements of A.S.T.M. (Designation D1143-81) superseded where applicable by the procedures specified in this document.
- 5.3.2 The Contractor shall submit Working Drawings for a suitable reaction system for the applied test loads to the Contract Administrator. Jacks must be secured with chains to provide adequate protection for the personnel in the event of breakage of the reinforcing steel bar or stressing system.
- 5.3.3 The Contractor shall submit Working Drawings for the reference system arrangement to the Contract Administrator. All reference beams shall be as follows:
 - The beams shall be independently supported with the support firmly embedded in the ground.
 - The testing device shall not apply compression to the bedrock surrounding the test for the Dowels into Rock, within a circle concentric with the dowel hole and a diameter equal to 4.0 m.
 - Reference beams shall be sufficiently rigid to support instrumentation such that variations in readings do not occur.
- 5.3.4 The Contractor shall construct suitable enclosures to provide complete protection for equipment and instruments from variations in the weather conditions and disturbances during the test program. These provisions must meet the approval of the Quality Verification Engineer and will include that the test enclosures must be weather-proof and provide a consistent temperature in order to eliminate temperature variations that could affect instrumentation.

5.4 Testing for Dowels Into Rock, and Report

- 5.4.1 At all times, the Contractor shall keep records of vertical and horizontal movements of the reaction system, elongation of reinforcing steel bar, and

the record of test enclosure temperature. The movements shall be recorded with respect to an independent fixed reference point. The Contractor shall submit Working Drawings that include the above noted records to the Contract Administrator.

5.4.2 Dial gauges shall have at least a 76.2 mm (3.0 in.) travel. Longer gauge stems or sufficient gauge blocks shall be provided to allow for greater travel if required. Gauges shall have precision of at least 0.025 mm (0.0001 in.). The dial gauges shall be placed on smooth bearing surfaces mounted perpendicular to the direction of movement. All gauges, scales or reference points attached to the test anchor shall be mounted so as to prevent movement relative to the test anchor during the test. The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.

5.4.3 Jacks used for reinforcing steel bars shall have a minimum ram dimension of 152.6 mm (6.0 in.). The Contractor shall submit Working Drawings that include details for current calibration and curves for all gauges to the Contract Administrator.

5.4.4 Requirements for Clauses 5.4.1 to 5.4.4 shall be repeated as required at different testing locations.

5.5 Testing Loading

5.5.1 The testing procedures shall safely load test the Dowels into Rock in tension at a rate of approximately 100kN per minute to the test load of 1,150 kN. The load shall be increased by an additional 50 kN beyond this level as directed by the Quality Verification Engineer.

5.5.2 Each load shall be maintained for a minimum time of 15 minutes and until the rate of displacement is not greater than 0.25 mm (0.01 inches) per hour.

5.6 Acceptance Criteria

5.6.1 The following acceptance criteria apply:

The testing of dowels shall be carried out in advance of the installment of Dowels into Rock at the pier footing.

Tests for Dowels into Rock shall have a capacity of at least 1035 kN. The Quality Verification Engineer shall report on the acceptance of the tests for Dowels into Rock. The Quality Verification Engineer shall report on the testing of the Dowels into Rock including recommendations for increasing embedment depth, if necessary.

6.0 BASIS OF PAYMENT

Hwy 11/17N MacKenzie River Bridge

Payment at the contract unit price for the above tender item shall include full compensation for all labour, equipment, and materials to do the work. No additional payment will be made for tests for Dowels into Rock which are deemed as included as part of the work for the above noted item.



EXCAVATION FOR FOUNDATIONS, Item No.

Non-Standard Special Provision

SCOPE

The work for the above noted tender item shall be in accordance with SSP902S01, except as extended herein. This NSSP specifies additional requirements for the excavation of rock, cleaning and inspection of excavations for structure foundations.

The Contractor is advised that the surface of the bedrock is variable across the site. Specifically, it is possible that an excavation for a footing may encounter sloping bedrock. Where the slope of the bedrock is greater than 5%, the bedrock shall be excavated to produce a bearing surface sloping at no steeper than 5%.

Following rock excavation, the Contractor shall clean all rock shatter, spalls and broken pieces of rock from the bearing surface.

BASIS OF PAYMENT

Payment for this item shall be at the rock excavation rate.

Appendix E

Drawings

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METRIC

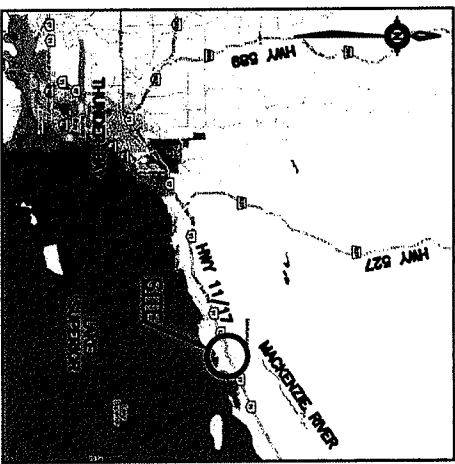
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
GWP No 6120-03-00

HIGHWAY 11/17
MTO NORTHWESTERN REGION
NEW MACKENZIE RIVER BRIDGES
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

TRIPPER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS



KEYPLAN

LEGEND

◆	Borehole
◆	Borehole and Cone
N	Blows /0.3m (Std Pen Test, 475u/blow)
CONE	Blows /0.3m (60° Cone, 475u/blow)
PH	Pressure, Hydraulic
±	Water Level
↑	Head Artesian Water
90%	Piezometer
A/R	Rock Quality Designation (RQD)
	Auger Refusal
NO	ELEVATION
09-01	235.9
09-02	235.0
09-03	225.8
09-04	223.3
09-05	230.7
09-06	237.7
09-07	242.1
09-08	235.1
09-09	225.7
09-10	221.8
09-11	229.9
09-12	237.8
	NORTHING
	EASTING

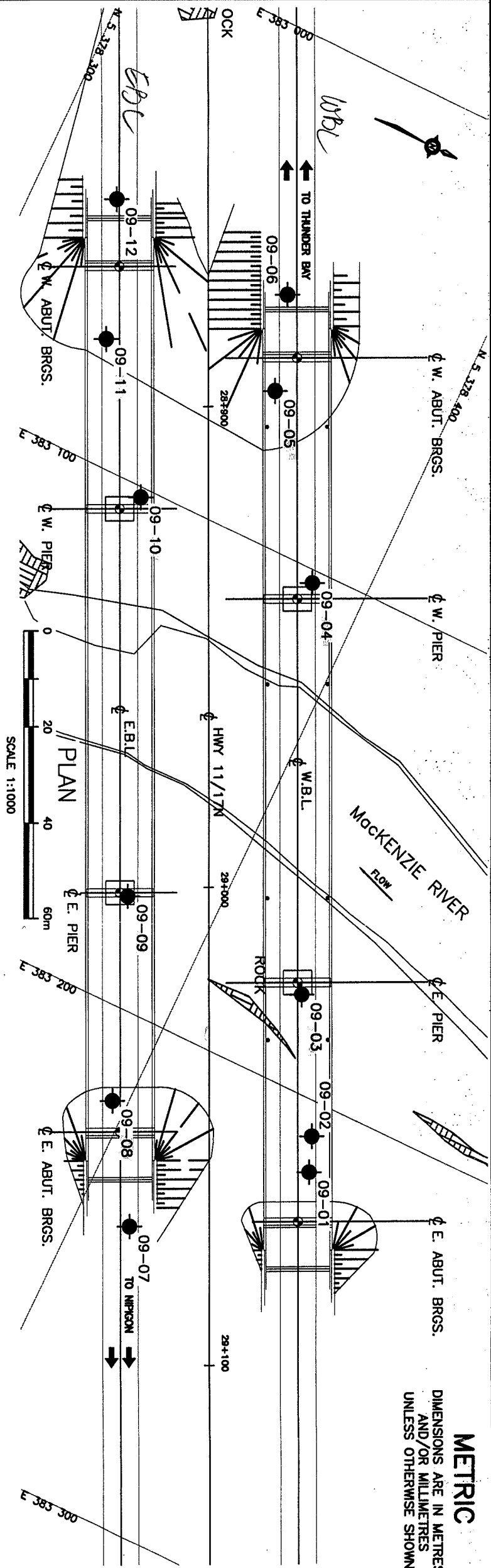
09-01	235.9	5 378 440.9	383 213.9
09-02	235.0	5 378 438.2	383 206.9
09-03	225.8	5 378 423.5	383 181.1
09-04	223.3	5 378 388.6	383 102.6
09-05	230.7	5 378 364.4	383 070.0
09-06	237.7	5 378 358.2	383 051.0
09-07	242.1	5 378 411.4	383 240.4
09-08	235.1	5 378 396.9	383 218.1
09-09	225.7	5 378 381.4	383 178.2
09-10	221.8	5 378 348.2	383 102.0
09-11	229.9	5 378 327.5	383 075.6
09-12	237.8	5 378 317.1	383 048.6

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.

GEORES No.

REVISIONS	DATE	BY	DESCRIPTION	DATE
DESIGN T/H	CHK PKG CODE	LOAD		JAN. 2010
DRAWN MFA	CHK AEG SITE	STRUCT		DWG 1



METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

CONT No
GWP No 6120-03-00



HIGHWAY 11/17
MTO NORTHWESTERN REGION
NEW MACKENZIE RIVER BRIDGES
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



LEGEND

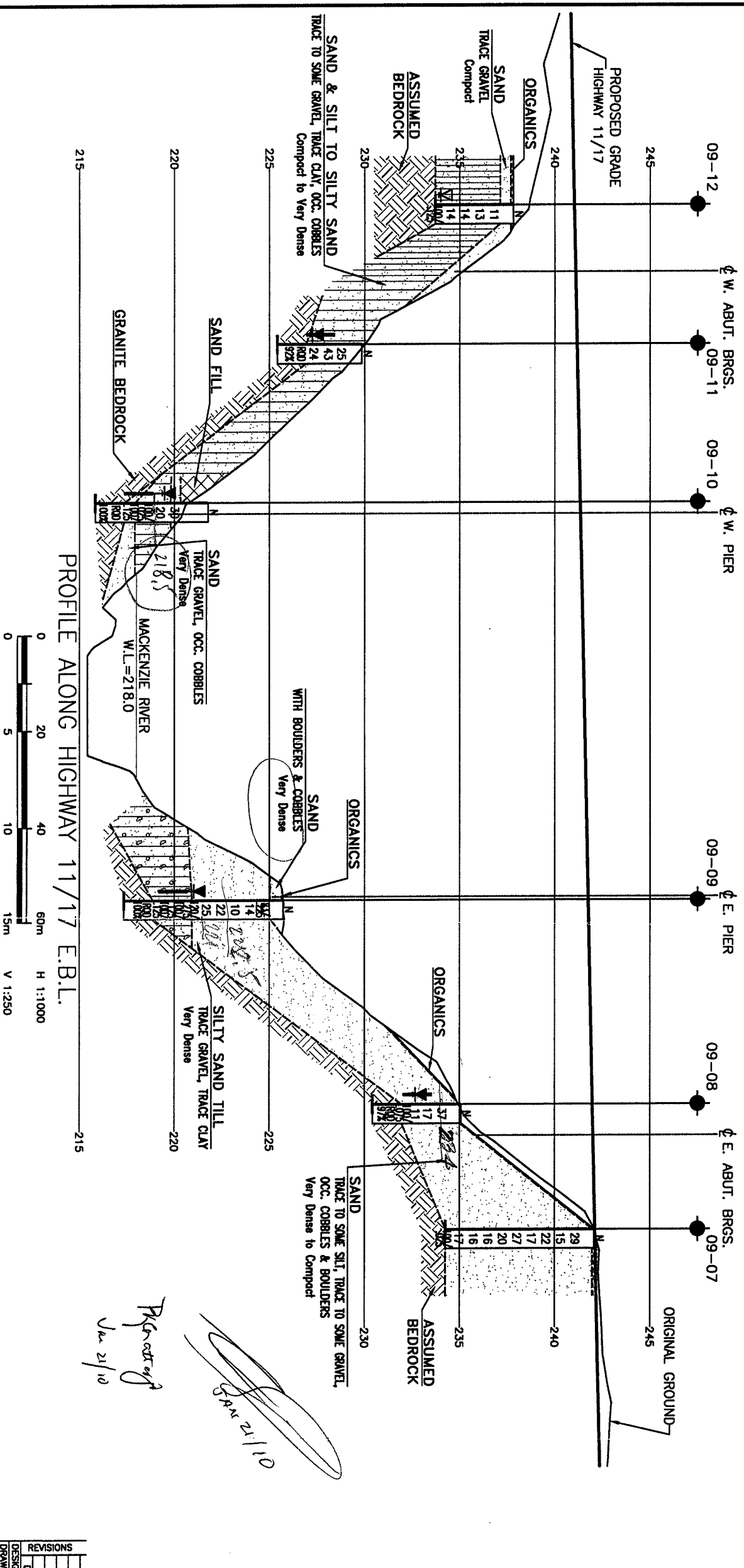
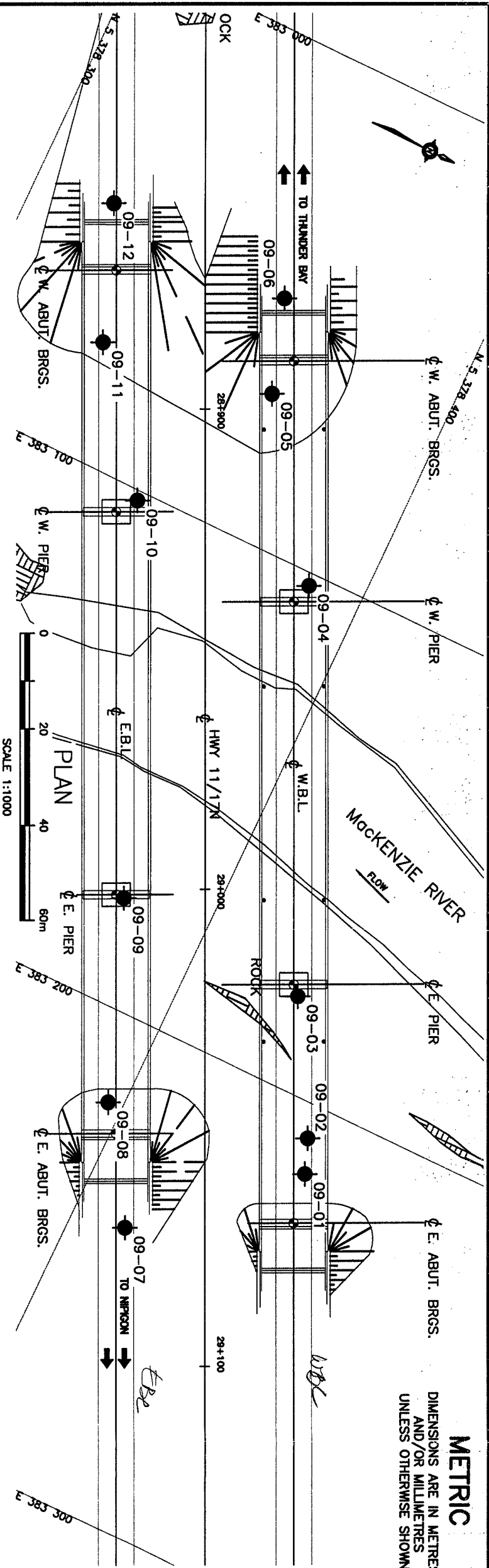
L E G E N D			
◆	Borehole		
◆	Borehole and Cone		
N	Blows /0.3m (Std Pen Test, 475N/blow)		
CONE	Blows /0.3m (60° Cone, 475N/blow)		
PH	Pressure, Hydraulic		
Σ	Water Level		
↑	Head Artesian Water		
90%	Piezometer		
A/R	Rock Quality Designation (RQD)		
	Auger Refusal		
NO	ELEVATION	NORTHING	EASTING
09-01	235.9	5 378 440.9	383 213.9
09-02	235.0	5 378 438.2	383 206.9
09-03	225.8	5 378 423.5	383 181.8
09-04	223.3	5 378 398.6	383 102.6
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09-08	235.1	5 378 396.9	383 216.1
09-09	225.7	5 378 381.4	383 178.2
09-10	221.8	5 378 348.2	383 102.0
09-11	229.9	5 378 327.5	383 075.6
09-12	237.8	5 378 317.1	383 048.6

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.

GEOCRES No.

REVISIONS	DATE	BY	DESCRIPTION	DATE	JAN. 2010
DESIGN T&H	CHK PRC	CODE	LOAD	DATE	JAN. 2010
DRAWN	MFL	CHK AEG	SITE	STRUCT	DWG 2



Appendix F

Site Photographs

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Looking west across the MacKenzie River at the ROW



Looking southwest from the site towards the Hydro ROW



Looking east across the MacKenzie River from the West Abutment WBL



Looking west along EBL on east side of the MacKenzie River