

FOUNDATION INVESTIGATION AND DESIGN REPORT

OTTAWA AVENUE OVERPASS NBL

HIGHWAY 11 BURK'S FALLS TO SOUTH RIVER

ONTARIO

G.W.P. 759-93-00, W.P. 749-93-01, SITE: 44-414

Geocres Number: 31E-249

Report to

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted at the Ottawa Avenue Overpass NBL structure on the proposed four-lane of Highway 11 west of the Town of South 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 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 as a sub-consultant to Marshall Macklin Monaghan, under the Ministry of Transportation Ontario (MTO) Agreement Number 5005-A-000188.

2 SITE DESCRIPTION

The site is located at Ottawa Avenue, at approximately 1.4 km west of the existing Highway 11, west of the Village of South River in the Township of Machar. The topography is rolling with dry highland areas and swampy lowlands. Vegetation in the immediate vicinity of the site consists of grass and small shrubs at the south side of Ottawa Avenue and large trees at the north side of Ottawa Avenue. Vegetation further beyond the site consists of large trees. A number of beaver dams have created ponded water south of the site. The southwest quadrant of the site is occupied by a small cemetery and a house.

The general 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 where exposed. Locally, however, the site lies in a gently rolling area with glacio-fluvial overburden over bedrock. These glacio-fluvial soils are part of a minor physiographic region known as The Number 11 Strip, which follows Highway 11 from Gravenhurst to North Bay. Locally, the ground surface elevation changes abruptly, creating numerous steep slopes and the low-lying areas are occupied by beaver ponds.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation and field testing for this project were carried out between October 18 and November 08, 2005 and consisted of drilling and sampling four boreholes at the foundation elements to depths ranging from 32.0 m to 35.2 m and two boreholes at the approach embankments to depths of 12.0 m. The boreholes were numbered 414-107 to 414-112 and their approximate locations are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix F.

Access to the specific borehole locations at the north abutment foundation element was difficult due to a locally steep slope and a buried natural gas pipeline on the north side of Ottawa Avenue. The locations of the boreholes drilled for the north abutment were selected to be as close as feasible to the abutment location while allowing safe operation of the drill rig. Borehole 414-109 was drilled approximately 3.6 m north of the proposed north abutment location and Borehole 414-110 was drilled approximately 2.8 m north of the proposed north abutment location. Borehole 414-110 was also drilled approximately 4 m west of the east end of the proposed north abutment location.

A combination of hollow-stem auger drilling techniques and casing and washboring methods were used to advance the boreholes and samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) in the overburden soils. In Boreholes 414-107 and 414-109 diamond coring was required to penetrate large boulders encountered at a depths ranging from 25.1 m to 31.0 m. Boreholes 414-107 to 414-110 were also advanced 2.7 m to 3.0 m into bedrock by NQ size diamond coring techniques.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Standpipe piezometers consisting of 19 mm PVC pipe with slotted screens were installed and enclosed in sand in two boreholes (one at each foundation element) to permit longer term groundwater level monitoring. The locations and completion details of the piezometers are shown in Table 3.1.

Table 3.1 – Piezometer Installation Details

Piezometer Location	Piezometer Details	
	Tip Depth/ Elevation (m)	Completion Details
414-108	34.4/307.3	Piezometer with 3.0 m slotted screen installed with sand filter to 31.1 m, bentonite grout seal from 31.1 m to 0.9 m, bentonite seal from 0.9 m to ground surface.
414-110	32.0/311.6	Piezometer with 1.5 m slotted screen installed with sand filter to 29.0 m, bentonite seal from 29.0 m to 28.3 m, bentonite grout from 28.3 m to 0.6 m, bentonite seal from 0.6 m to ground surface.

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 Oakville 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.

4 LABORATORY TESTING

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

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 F. An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole Sheets governs any interpretation of the site conditions. Boreholes 414-109 and 414-110 were located approximately 3.6 m and 2.8 m away from the centre line of the north abutment respectively. However, the soil stratigraphy encountered is similar in all boreholes drilled for this structure and similar to that encountered in the investigation for the adjacent SBL structure. The stratigraphic model developed from this information is considered to be representative of the conditions at the abutments. Variation in the elevations of stratigraphic contacts should be expected, but not of a magnitude that is expected to have a significant impact on the project.

In general, the site is underlain by 29.3 m to 32.3 m of overburden soils overlying Pre-Cambrian bedrock. The overburden soils consist of topsoil, fill, sand, silt to silty sand, and sand with some gravel, cobbles and boulders.

5.1 Topsoil and Fill

Topsoil ranging from 50 mm to 150 mm was encountered across the site. Topsoil thickness may vary between and beyond the boreholes.

Fill was encountered at the south approach in Borehole 414-111, which was located to the east of the house. The fill consisted of fine to medium grained compact sand to a depth of 1.5 m or an elevation of 341.5 m.

5.2 Upper Sand

The topsoil and ground surface are underlain by a deposit of brown, dry to moist, silty sand to sandy silt that extends to depths ranging from 0.6 m to 3.1 m or to elevations ranging from 343.8 m to 338.8 m.

A sample from this deposit was subjected to a grain size distribution test and the result is reported on the Record of Borehole Sheet and is plotted in Figure B1 in Appendix B.

SPT 'N' values ranged from 3 to 22 blows for 0.3 m penetration in this stratum indicating a very loose to compact relative density. The moisture content of samples from this deposit ranged from 10% to 22%.

5.3 Silt to Silty Sand

Across the site the topsoil and upper sand deposits are further underlain by a major deposit of wet silt to silty sand below the water table. The deposit consists of zones of silt with trace to some sand, silt and sand, and fine to medium grained silty sand. At the north abutment in Boreholes 414-109 and 414-110, large zones of fine grained sand with some silt were also encountered within this deposit. The deposit was penetrated in all of the boreholes where it was found to extend to depths ranging from 25.1 m to 28.3 m or to elevations ranging between 319.5 m and 313.3 m.

Samples from this deposit were subjected to grain size distribution tests and the results are reported on the Record of Borehole Sheets and are plotted in Figures B2 to B5 in Appendix B.

Standard Penetration tests in this deposit gave 'N' values ranging from 6 to 85 blows per 0.3 m penetration but generally 'N' values ranging from 13 to 76 blows per 0.3 m penetration were recorded indicating a compact to very dense relative density. Higher 'N' values (more than 50 blows for under 0.3 m penetration) are attributed to the probable presence of cobbles and boulders within the deposit.

The moisture content of samples from this deposit varies between 16% and 30%.

5.4 Lower Sand

A deposit of wet sand (lower sand) was encountered below the silt to silty sand. The deposit is a medium to fine grained sand with some gravel, some cobbles and trace silt and exists below the water table. Boulders were also encountered in this deposit in Boreholes 414-107 and 414-109. The deposit extends to depths ranging from 29.3 m to 32.3 m or to elevations ranging between 314.3 m and 309.5 m.

Standard Penetration tests in this deposit yielded 'N' values generally ranging from 55 blows per 0.3 m penetration to more than 50 blows for under 0.3 m penetration, which is

attributed to the presence of cobbles and boulders within the deposit. Based on these results the deposit is considered to have a very dense relative density.

The moisture content of samples from this deposit ranged from 24% to 26%.

Upon encountering boulders, Boreholes 414-107 and 414-109 were advanced using diamond coring techniques. Coring and washboring continued through the soils between the boulders, although recovery was limited to a few pieces of gravel and cobbles. The boulders were measured to range from 0.2 m to 0.5 m in thickness along the axes of the boreholes.

5.5 Bedrock

The overburden soils described above are underlain by gneiss bedrock. Bedrock was proved by coring at both abutments. Table 5.1 summarizes the bedrock depth and the elevations to the top of bedrock.

Table 5.1 – Depth to Bedrock

Location	BH Number	Depth to Bedrock	Top of Bedrock Elevation
South Abutment	414-107	32.3	309.5
South Abutment	414-108	31.4	310.3
North Abutment	414-109	31.7	312.9
North Abutment	414-110	29.3	314.3

The gneiss bedrock is described as fresh to slightly weathered. Its colour is pink with visible black sub-vertical banding in most cores.

Core recovery in the bedrock was generally between 80% and 100%. The RQD values ranged from 78% 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 3.

The unconfined compressive strength of the rock cores is estimated to range between 124 and 208 MPa indicating very strong rock. These estimated rock strength values are based on 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 Depths to Refusal

Effective refusal, defined as an SPT value exceeding 100 blows for 0.3 m of penetration (or 50 blows for less than 150 mm penetration), was encountered in the gravelly sand, cobbles and boulders above the bedrock surface in the three deep boreholes. The depths at

which effective refusal was encountered, and the depth to the bedrock surface, are shown in Table 5.2.

Table 5.2 – Refusal and Bedrock Depths

Location	Borehole	Refusal		Bedrock	
		Depth (m)	Elevation	Depth (m)	Elevation
South Abutment	414-107	28.2	313.6	32.3	309.5
	414-108	28.3*	313.3	31.4	310.3
North Abutment	414-109	25.1	319.5	31.7	312.9
	414-110	25.1	318.5	29.3	314.3

* Interpreted.

5.7 Water Levels

A standpipe piezometer was installed in one borehole at each abutment and water levels were measured on several visits made after the completion of drilling. The water level readings are presented in Table 5.2.

Table 5.2 – Water Level Measurements

Date	BH 414-108		BH 414-110	
	Depth	Elev.	Depth	Elev.
October 24, 2005	0.3	341.4		
October 26, 2005	0.3	341.4	2.9	340.7
October 27, 2005	0.3	341.4	2.8	340.8
November 5, 2005	0.2	341.5	2.5	341.1
November 7, 2005	0.2	341.5	2.4	341.2
November 15, 2005	0.4	341.3	2.7	340.9
November 17, 2005	0.4	341.3	2.6	341.0
November 21, 2005	0.4	341.3	2.7	340.9
November 23, 2005	0.4	341.3	2.7	340.9

Based on these observations, local groundwater levels exist at Elevations 340.9 m to 341.5 m. All groundwater observations at this site are short term and the levels are expected to fluctuate seasonally and after severe weather events. It is anticipated that standing water might be encountered across part or all of this site after the spring snow melt or after severe rainfall events.

6 MISCELLANEOUS

Borehole locations were selected by Thurber Engineering Ltd. Surveyors from Marshall Macklin Monaghan Ltd. staked these locations in the field, confirmed the co-ordinates and obtained the ground surface elevations.

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

Eastern Ontario Soil Investigations of Hawkesbury supplied a track mounted CME 75 drill rig and conducted the drilling, sampling and in-situ testing operations.

The field program was supervised on a full time basis by Mr. George Azzopardi of Thurber.

Routine laboratory testing was carried out by Thurber Engineering Ltd.

Overall supervision of the field program, interpretation of the data 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.

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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.

It is understood that Highway 11 NBL will cross over Ottawa Avenue via a single span structure spanning 28 m between abutments. The proposed finished grade at the structure will be about Elevation 349.2 m at the north abutment and the original ground surface is at average Elevation 344.1, resulting in an approach embankment up to 5.1 m high. At the south abutment, the finished grade will be at Elevation 349.9 and the original ground surface is at average Elevation 341.8, resulting in an approach embankment up to 8.1 m high.

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 the investigations.

8 STRUCTURE FOUNDATIONS

The proposed bridge is a single-span overpass structure with two abutments as foundation elements.

The stratigraphy encountered at the abutment locations consist of 29 m to 32 m of sands and silts overlying bedrock. The groundwater level lies at elevations ranging from Elev. 340.9 m to Elev. 341.5 m, within 0.4 to 2.6 metres of ground surface.

Initial consideration was given to the following foundation types:

- Spread footings
- Augered Caissons (drilled shafts)
- Driven piles

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

8.1 Spread Footings on Native Ground

The upper 4 to 6 m of soil at the abutments consists of loose to compact sand. The geotechnical resistance of these deposits is relatively low and settlements in these deposits under the footing load will be relatively large, compared to the requirements for highway bridge design. Moreover, in order to maintain the proposed profile grade approximately 8 m to 9 m high abutment stems will be required if footings are placed on these native deposits. Consequently, spread footings on native ground are not considered to be a feasible foundation alternative and are not recommended.

8.2 Spread Footings on Engineered Fill

Consideration was also given to placing spread footings on an engineered fill pad but the geotechnical resistance of the underlying soils is relatively low and settlements under the footing load will be relatively large. Therefore, the option of supporting the main structure on spread footings on an engineered fill pad is not recommended.

8.3 Augered Caissons (Drilled Shafts)

Augered caisson foundations were also considered for the support of the structure.

The soils in which caissons might be founded are non-cohesive and possible founding levels are several metres below the groundwater level. Consequently, the caissons would have to be installed using slurry techniques and liners to control the inflow of groundwater. Even with these precautions, it would be difficult for contractors to maintain a stable base in the caisson excavation and the geotechnical resistance would have to be based on skin friction, with allowance made for disturbance of the soil around the shaft.

Alternatively, caissons could be founded on the bedrock, in which case high geotechnical resistance would be available. However, the permeable nature of the overburden soil and the presence of boulders above the bedrock would make it difficult to seal the bottom of the liner at the bedrock interface. Unwatering of the caisson would be impractical and attempts to do so might result in continued flow of fines into the excavation.

Installation of deep caissons to bedrock is also expected to be a more expensive option than driven piles.

For these reasons, the use of a caisson foundation is not recommended.

8.4 Driven Steel Piles

The subsurface conditions at the site are considered suitable for the design of foundations supported on steel H-piles driven to achieve resistance in the very dense soil or on bedrock.

The bedrock is overlain by a layer of very dense sand with gravel, cobbles and boulders that varied in thickness up to 6 m at the north abutment and to between 4 and 6 m at the south abutment. The presence of this variable layer makes it difficult to predict the depth at which piles will achieve resistance. In some cases, resistance will be developed in the very dense soil but in other cases, piles may fully penetrate the soil layer and achieve resistance on the underlying bedrock. Consequently, it is recommended that the foundations be designed on the basis of the geotechnical resistance achieved by piles driven into the very dense soil. Piles reaching bedrock will develop a higher axial resistance than those stopping in the soil and thus the design assumption is safe.

Two steel pile sections believed to be currently available have been considered for use in the proposed foundations.

8.4.1 Axial Resistance

The factored, vertical, axial, geotechnical resistances for these pile sections when driven into very dense soil, and their approximate tip elevations, are presented in Table 8.1.

Table 8.1 – Axial Resistance of Various Pile Sections

Pile Section	Piles Driven Into Sand with Cobbles and Boulders			
	ULS (Factored)	SLS (25 mm Settlement)	Estimated Pile Tip Elevation	
			N. Abutment	S. Abutment
HP 310 X 110	1,800 kN	1,600 kN	319.0	313.5
HP 310 X 152	1,900 kN	1,700 kN	319.0	313.5

The slightly larger cross-section of the HP 310 X 152 results in an additional 100 kN in the axial resistance compared to the HP 310 X 110 cross-section, based on rounded values.

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

The pile tip elevations shown in Table 8.1 should be used for cost estimating purposes only. The actual pile tip elevations will be controlled as described in Section 8.4.4 Pile Driving.

8.4.2 Pile Tips

Due to the presence of cobbles and boulders in the expected founding layer, the tips of all piles should be fitted with cast steel, H-section rock points from an approved manufacturer such as Titus Steel (Standard H-point) or APF hard Bite or approved equivalent.

The use of rock points is recommended for the following reasons:

- The piles will be driven into soil containing cobbles and boulders, which requires a higher level of protection than driving into soils containing only smaller particle sizes
- Some piles may achieve refusal on large boulders, which will require the same pile tip protection and reinforcement as founding on bedrock

- Some piles may achieve resistance on the bedrock.

8.4.3 Pile Installation

Pile installation should be in accordance with Special Provision No. 903S01.

The Contract Documents should contain a NSSP alerting the Bidders to:

- The presence of cobbles and boulders in the expected bearing stratum.
- The possibility of piles within a group achieving the specified resistance at different elevations.
- The possibility of some piles meeting refusal on a large boulder.

Suggested wording for the NSSP is included in Appendix D.

8.4.4 Pile Driving

Pile driving must be controlled by the Hiley Formula and an ultimate pile resistance to be specified by the designer in accordance with Clause 3.3.2 (b) Construction Stage of the Structural Manual. The Hiley formula need not be used until the piles are approaching the bearing stratum below Elevation 323 at the north abutment and 317 at the south abutment. The appropriate pile driving note is "Piles to be driven in accordance with Standard SS 103-11 using an ultimate resistance of "R" kN per pile".

"R" must have the minimum values shown in Table 8.2.

Table 8.2 – Ultimate Geotechnical Resistance of Piles

Pile	Ultimate Resistance (R) (kN)
HP 310X110	3,600 kN
HP 310X152	3,800 kN

The NSSP referenced in 8.4.3 should also require the QVE to terminate driving before the pile is damaged by overdriving.

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

8.4.5 Downdrag

Downdrag on the piles is not considered to be an issue at this site.

8.4.6 Integral Abutment Considerations

The ground conditions at this site are considered suitable for an integral abutment design.

The integral abutment design requires that the piles possess flexibility in the upper 3 m of the pile length. The near surface, native soils at this site are loose to compact and the lateral resistance of a pile in this soil might provide sufficient flexibility. However, the upper 3 m of the pile may lie partially within the compacted fill of the approach embankment and partially in the underlying native soils, which may become densified under the embankment loading. Accordingly, to provide the required flexibility in the piles, the upper 3 m of the piles should be surrounded by a 600 mm diameter CSP as specified by the integral abutment design procedures.

After the pile is driven, the space between the pile and the CSP should be filled with sand. An NSSP should be included in the contract drawings specifying the gradation of the sand according to Table 8.3.

Table 8.3 – Integral Abutment Sand Grading

MTO Sieve Designation		Percentage Passing
2 mm	#10	100%
600 µm	#30	80%-100%
425 µm	#40	40%-80%
250 µm	#60	5%-25%
150 µm	#100	0%-6%

8.4.7 Lateral Resistance

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

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

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

where z = depth of embedment of pile in metres

D = pile width in metres

n_h = coefficient of horizontal subgrade reaction (Table 8.4)

γ = unit weight (Table 8.4)

K_p = passive earth pressure coefficient

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, $K = k_s \times L \times D$ (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, P_{ult} , may be obtained from the expression, $P_{ult} = p_{ult} \times L \times D$.

Table 8.4 – Recommended Soil Parameters

Location	Elevation	n_h (kN/m ³)	K_p	Unit Weight* (kN/m ³)	Soil Conditions
North Abutment BH 414-109 BH 414-110	OGL to 343.0	1,200	2.8	20	Silt and Sandy Silt
	343.0 to 336.5	1,200	3.0	10	Silt
	336.5 to 325.0	3,000	3.0	10	Sand
	Below 325.0	5,000	3.3	12	Sand, silt, cobbles and boulders, very dense
South Abutment BH 414-107 BH 414-108	OGL to 339.0	1,200	2.8	11	Silty sand
	339.0 to 320.0	2,000	3.0	10	Clay, silty, firm to stiff
	320.0 to 315.0	3,000	3.0	10	Silt, compact
	Below 315.0	5,000	3.3	12	Sand, silt, cobbles and boulders, very dense

*Buoyant unit weight below the water table.

Pile interaction should be considered with reference to CHBDC Clause 6.8.9.2.

For lateral soil/pile group interaction analysis, the equation for k_s quoted above may be used in conjunction with appropriate reduction factors.

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.

For conventional abutments, the lateral resistance may be provided by battered piles.

8.5 Recommended Foundation

The use of H-piles at the abutments allows for the design of an integral abutment structure. From a geotechnical point of view, it is recommended that all foundations for the main bridge structure be supported on steel H-piles driven into the very dense soil or to bedrock.

8.6 Frost Cover

Pile caps and footings should be provided with a minimum of 1.9 m of earth cover over the footing base (founding elevation).

9 EXCAVATION

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the 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. This classification is based on the lack of cohesion in the soils and the resulting possibility that excavation slopes will slough if excavated vertically for the lower 1.2 m. Excavation slopes should not exceed 1V:1H above the groundwater level.

Excavation below the groundwater level without prior dewatering is not recommended since the inflow of groundwater will cause boiling and sloughing of the soil below the water table making it difficult to maintain a dry, sound base on which to work.

Prior to excavation below the natural groundwater level, the groundwater must be depressed to a level below the deepest excavation level sufficient to maintain a stable base and prevent soil disturbance by construction traffic.

10 UNWATERING

Based on the preliminary GA for the bridge structure and the use of a piled foundation, it is not expected that work at the abutments will require excavation below the groundwater level. However, the Contractor should be prepared to pump from sumps to remove any seepage water or surface water collecting in an excavation. The Contract Documents should also contain a NSSP alerting the Contractor to the risks associated with excavation below the groundwater level without prior dewatering. Suggested wording is included in Appendix D.

The design of the dewatering system that may be required should be the responsibility of the Contractor and the Contract Documents should alert him to this responsibility and the need to engage a dewatering specialist. While the responsibility for dewatering should remain with the Contractor, suitable systems that might be employed include pumping from filtered sumps for penetration of no more than 0.5 m below the groundwater level and the use of vacuum wellpoints for deeper penetration below the groundwater level.

11 APPROACH EMBANKMENTS

Approach embankment construction using either earth fill or rock fill is feasible on the foundation soils encountered at this site. Settlement in the order of 100 to 150 mm will occur under the loading imposed by approximately 5 to 8 m of approach fill but due to the non-cohesive nature of the foundation soils, the settlement will be immediate and essentially complete when construction of the fill is completed.

The immediate approach embankments should be constructed using non-cohesive fill. The embankments will also experience settlement resulting from consolidation of the fill. This settlement is expected to be approximately 50 to 80 mm for 5 to 8 m high embankments. The settlement within the non-cohesive fill should be immediate in nature and essentially be complete shortly after construction has been completed.

The global, internal and surficial stability of the approach embankment fill will depend on the slope geometry and also to a large degree on the material used to construct the embankment. If the embankment is constructed of blast rock fill, it may be assumed that the side slopes will be stable at inclinations up to 1.25H:1V. Embankments constructed using granular material, select subgrade material or non-cohesive earth fill will have stable side slopes at inclinations of up to 2H:1V.

For the purpose of embankment stability analyses, the commercially available slope stability program GSLOPE developed by Mitre Software Inc. was used. The Bishop's simplified method for stability analysis was employed.

Global stability analyses were conducted for 2H:1V SSM or earth fill embankments and for 1.25H:1V rock fill embankments. The stability of the embankments was also checked under seismic loading assuming an acceleration of 0.18g. The computed factors of safety are as shown in Table 11.1.

Table 11.1 Computed Factors of Safety

Location / Material	Condition	Factor of Safety	Figure
South Approach			
Rock Fill	Normal	1.6	E1
Rock Fill	Seismic = 0.18g	1.2	E2
Earth Fill	Normal	1.5	E3
Earth Fill	Seismic = 0.18g	1.0	E4
North Approach			
Rock Fill	Normal	1.8	E5
Rock Fill	Seismic = 0.18g	1.3	E6
Earth Fill	Normal	1.7	E7
Earth Fill	Seismic = 0.18g	1.1	E8

In each case of normal loading, the factor of safety against global failure was greater than 1.5. Under the assumed seismic loading, the minimum factor of safety calculated was 1.0. These

factors of safety are considered to be acceptable for the proposed embankment bearing on non-cohesive soil.

It is recommended that all topsoil and peat be stripped prior to constructing the approach fills. Embankment construction should be in accordance with OPSS 206, as amended by Special Provision "Amendment to OPSS 206, December 1993", dated November 2002.

The approach fills should be constructed in advance of pile driving operations.

Where earth fill embankments are higher than 6 m, mid-height berms should be incorporated in the design. The berms should:

- extend for the length through which the embankment height exceeds 6 m
- be at least 2 m wide
- have 2% positive drainage to shed run-off water.

Where rock fill embankments are higher than 6 m, mid-height berms should be incorporated in the design. The berms should:

- extend for the length through which the embankment height exceeds 6 m
- be at least 2 m wide

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

12 RETAINED SOIL SYSTEMS

The General Arrangement drawings indicate that retaining walls may be used beyond both abutments to retain the forward slope of the approach fills. Retained soil system (RSS) walls may be used subject to the requirements presented in this section.

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

12.1 Foundation

The performance of a 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.

At this site, the foundation conditions under the entire width of the RSS mass must be considered, i.e. from the face of the wall to the furthest extent of the reinforcement.

All topsoil, peat and organic soil must be stripped from the area extending beyond the footprint of the RSS mass for a distance equal to the height of thickness of any engineered fill placed below the RSS.

The native soils above the following elevations are considered to be too loose to support the RSS without risk of unacceptable settlement:

- North abutment – 344.0 (or ground surface if it is lower)
- South abutment – 340.0

All native soil lying at higher elevations must be recompacted to 100% of its Standard Proctor maximum dry density (SPMDD) or it must be removed and be replaced by engineered fill.

To provide an acceptable foundation for this RSS mass, the following conditions must be met:

- All fill placed below the RSS mass must be OPSS Granular A engineered fill compacted to 100% of its SPMDD at $\pm 2\%$ of optimum moisture content.
- The underside of the fill must be placed no higher than Elev. 343.5 m at the north abutment and Elev. 340.0 m at the south abutment.
- The geometry of the engineered fill must conform to the limits illustrated in Figure 1.

The subgrade should be competent and free of organics, soft or deleterious soils. The native soil under the RSS foundation should be re-compacted

Dewatering may be required to prepare the subgrade for placement and compaction of the engineered fill pads. The contractor must be prepared to install an appropriate dewatering system and to maintain the excavation in an unwatered condition.

The following parameters may be used for the design of the RSS:

- Ultimate coefficient of sliding resistance of cast in-situ concrete levelling pad on Granular A = 0.7
- Ultimate coefficient of sliding resistance of RSS mass on Granular A = 0.7

All topsoil, organics and soft soils should be removed from the subgrade of the RSS wall prior to placement of the fill.

The entire block of reinforced earth must also be designed against various modes of failure including sliding and overturning.

Settlement under a RSS mass constructed as outlined above is expected to be less than 25 mm and to occur essentially as the RSS is constructed.

12.2 Global Stability

The global stability of the RSS wall is dependent on the characteristics of the embankment fill and the foundation soils, the geometry of the embankment and location of the RSS within the embankment.

RSS walls are likely to be used beyond both abutments. It is envisaged that the RSS will be founded on an engineered fill core as shown in Figure 1 and will be located up the slope in the embankment fill.

Stability analyses were carried out based on the following variables:

- RSS founded on engineered fill – Granular A or Granular B Type II compacted to 100% SPMDD at $\pm 2\%$ optimum moisture content, with a slope of 2H:1V (angle of internal friction, ϕ , of 35° , cohesion of 0, and unit weight, γ , of 22.8 kN/m^3) immediately below the levelling pad and the RSS mass and Granular B as abutment backfill.
- Embankment fill behind the RSS is inclined at 2H:1V.
- Groundwater level at approximate Elev. 341 m

Analysis carried out on RSS walls located in the embankment fill indicates that the factor of safety for global stability is 1.2. This is the worst case, highest wall and an assumption of infinitely long wall. In reality the wall is of finite length, reducing in height, built against the abutment and returning on itself. With these conditions, the global stability of the RSS wall is considered to be satisfactory.

The internal stability of the RSS wall should be analyzed by the supplier/designer of the proprietary product selected for this site.

13 BACKFILL TO ABUTMENTS

In the case of integral or semi-integral abutments, backfill to the abutment should be granular material.

In the case of a conventional abutment, granular backfill is recommended but rock backfill can be permitted.

In all cases where the approach embankment consists of rock fill and granular backfill to the abutment wall is used, the granular backfill must consist of OPSS Granular "B" Type II.

The backfill to the abutment walls must be in accordance with OPSS 902 as amended by Special Provision 902S01. Granular backfill must be placed to the extents shown in OPSD 3501.000, and rock backfill must be placed to the extents shown in OPSD 3505.000. All granular material should meet the requirements of SP 110F13 Amendment to OPSS 1010, March 1993.

Compaction equipment to be used adjacent to retaining structures must be restricted in accordance with SSP 105S10.

The design of the abutment must incorporate a subdrain as shown in OPSD 3501.000 or OPSD 3505.000, as applicable.

14 EARTH PRESSURE

Earth pressures acting on the structure may be assumed to be triangular and to be governed by the characteristics of the abutment backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

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

Where:

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

K = 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 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 shown in Table 14.1.

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) might be preferred as it results in lower earth pressures acting on the wall. In the case of integral abutments, material with a lower passive pressure coefficient (e.g. Granular B Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass.

The factors in Table 14.1 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 Canadian Highway Bridge Design Code.

Table 14.1 – 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 $\phi = 32^\circ, \gamma = 21.2 \text{ kN/m}^3$		Rock Fill (Limited to 300 mm size) $\phi = 42^\circ, \gamma = 19 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall(2H:1 V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall(2H:1 V)
Active (Unrestrained Wall)	0.27	0.40*	0.31	0.48*	0.2	.28*
At rest (Restrained Wall)	0.43	-	0.47	-	.33	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-	5.0	-

* For wing walls.

15 SEISMIC CONSIDERATIONS

15.1 Seismic Design Parameters

The site is treated as lying in Seismic Zone 2. The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 2
- Zonal Velocity Ratio 0.1
- Acceleration Related Seismic Zone 2
- Zonal Acceleration Ratio 0.1
- Peak Horizontal Acceleration 0.11

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

15.2 Liquefaction Potential

The potential for liquefaction of the foundations soils was assessed using the Seed and Idriss (1971) method¹.

Using this method, it is estimated that under the existing conditions the foundation soils at both abutments are not prone to liquefaction. At the abutments, the approach embankments will increase the effective stress on the soil under the embankment and around the piles and as a result, liquefaction at the foundation is not considered to be likely.

The foundation loads will be transferred by steel piles to very dense sand with cobbles and boulders, or possibly to bedrock. In either case, it is not considered likely that the vertical geotechnical resistance of the piles will be compromised.

15.3 Retaining Wall Dynamic Earth Pressures

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.

In calculating the active, passive and at rest earth pressure coefficients the angle of friction between the wall and backfill material is assumed to be 0.5ϕ . For the design of retaining walls, the coefficients of horizontal earth pressure in Table 15.1 may be used:

Table 15.1 – Earth Pressure Coefficient for Earthquake Loading

Earth Pressure Coefficient (K) for Earthquake Loading						
Wall Condition	Granular A or Granular B Type II $\phi = 35^\circ$; $\delta = 17.5^\circ$ $\gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ$; $\delta = 16^\circ$ $\gamma = 21.2 \text{ kN/m}^3$		Rock Fill $\phi = 42^\circ$; $\delta = 21^\circ$ $\gamma = 19.0 \text{ kN/m}^3$	
	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 (K_{AE})*	0.3	0.45	0.33	0.54	0.23	0.31
Passive (K_{PE})	6.3	6.3	5.4	5.4	12.0	12.0
At Rest (K_{OE})**	0.59		0.63		0.33	

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

** After Woods

¹ Seed, H.B. and Idriss, I.M. 1971, "Simplified Procedure for Evaluating Soil Liquefaction Potential" *Journal of Soil Mechanics and Foundations Division*, ASCE, Vol. 101, No. SM9, September, pp. 1249-1273.

16 CONSTRUCTION CONCERNS

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

- The possibility of piles reaching refusal on large boulders. In this case, the Hiley formula is not appropriate and site staff must make a decision regarding pile resistance and the appropriateness of continued driving.
- The potential variability of pile lengths at refusal.
- Unwatering in the case of excavations that must penetrate below the groundwater level

17 CLOSURE

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

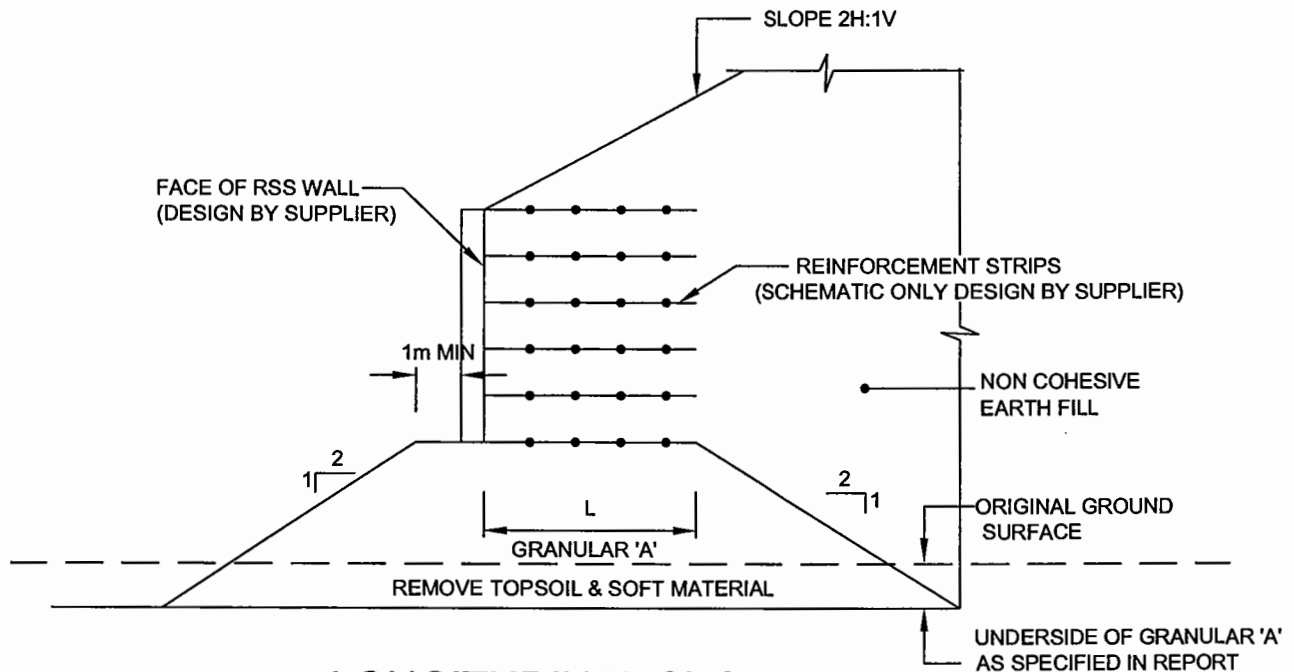
The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.

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



Report reviewed by:
P.K. Chatterji, P.Eng., Ph.D.
Review Principal

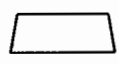


LONGITUDINAL SECTION

NOT TO SCALE

NOTES:

1. REMOVE TOPSOIL AND OR SOFT SUBSOIL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH FILL.
2. PLACE GRANULAR 'A' BELOW PLAN AREA OF RSS MASS.
3. CONSTRUCT RSS MASS
4. PLACE REMAINDER OF GRANULAR 'A' AND EARTH FILL AS REQUIRED.
5. MODIFIED FROM M.T.C 1982.

ENGINEER	AEG	RSS MASS ON COMPACTED FILL SHOWING GRANULAR A CORE	 THURBER
DRAWN	SS		
DATE	MAR. 2006		
APPROVED	PKC		
SCALE	NTS		
			DWG. NO. FIGURE 1

Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

NOTE: Hierarchy of Soil Strength Prediction

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

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)


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

5. LEGEND FOR RECORDS OF BOREHOLES

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

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

 Water Level

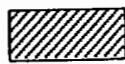
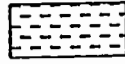
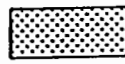


 Shear Strength Determination by Pocket Penetrometer

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

UNIFIED SOILS CLASSIFICATION

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

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION		SYMBOLS	
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)

DISCONTINUITY SPACING		STRENGTH CLASSIFICATION			
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				
Very thinly bedded	20 to 60mm	Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Laminated	6 to 20mm				
Thinly Laminated	Less than 6mm	Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.

TERMS					
Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.	Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
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.				

METRIC

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	"N" VALUES		SHEAR STRENGTH kPa	WATER CONTENT (%)			
341.8										
0.0	TOPSOIL (150mm)									
0.2	Silty SAND , fine grained Compact to Very Loose Brown Moist		1	SS 10					1 5 89 5	
			2	SS 3						
			3	SS 17						
338.8										
3.1	SILT , trace to some sand, trace clay, trace gravel Brown Compact Wet		4	SS 24						
336.4										
5.5	Silty SAND , trace clay Very Dense Brown Wet		5	SS 50/ .150						
	Becoming Compact		6	SS 20						
			7	SS 22						

+ ³, × ³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 414-107

2 OF 4

METRIC

W.P. 750-93-01 LOCATION N 5 077 433.6 E 312 892.2 Ottawa Avenue NBL ORIGINATED BY MEF
 HWY 11 BOREHOLE TYPE NW Casing/NQ Core Barrel/Dynamic Cone COMPILED BY HS
 DATUM Geodetic DATE 05.11.05 - 06.11.05 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		WATER CONTENT (%)				
								20 40 60 80 100						
								○ UNCONFINED + FIELD VANE						
								● QUICK TRIAXIAL × LAB VANE						
								20 40 60 80 100						
			8	SS	8									
							331							
							330							0 64 36 (SI+CL)
							329							
			9	SS	27									
							328							
							327							
							326							
			10	SS	34									0 77 23 (SI+CL)
							325							
							324							
							323							
			11	SS	29									
							322							

Continued Next Page

+ 3 . × 3 : Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 414-107

4 OF 4

METRIC

W.P. 750-93-01 LOCATION N 5 077 433.6 E 312 892.2 Ottawa Avenue NBL ORIGINATED BY MEF
 HWY 11 BOREHOLE TYPE NW Casing/NQ Core Barrel/Dynamic Cone COMPILED BY HS
 DATUM Geodetic DATE 05.11.05 - 06.11.05 CHECKED BY AEG

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									
								20	40	60	80	100					
311.8 30.0	BOULDER																
311.1			3	RUN													
310.8 31.0	BOULDER																
309.5			4	RUN													
32.3	GNEISS BEDROCK Fresh to slightly weathered, thinly bedded, pink with black subvertical banding																
			5	RUN													
			6	RUN													
306.7																	
35.2	END OF BOREHOLE AT 35.18m. BOREHOLE GROUTED WITH BENTONITE TO SURFACE.																

+³, ×³: Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

METRIC

SOIL PROFILE	CAMDEN			DYNAMIC CONE PENETRATION			
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+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 414-108

2 OF 4

METRIC

W.P. 750-93-01 LOCATION N 5 077 437.9 E 312 903.6 Ottawa Avenue NBL ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Core Barrel/Dynamic Cone COMPILED BY HS
 DATUM Geodetic DATE 21.10.05 - 22.10.05 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)			
								○ UNCONFINED + FIELD VANE								W _p — W — W _L			
								● QUICK TRIAXIAL × LAB VANE											
							20	40	60	80	100	20	40	60	kN/m ³	GR SA SI CL			
323.1	Dense		10	SS	66		331											0 80 20 (SI+CL)	
			11	SS	52		330												
			12	SS	37		329												
			13	SS	62		328												
			14	SS	42		327												
			15	SS	36		326												
18.6	SILT, trace sand, trace clay Dense Brown Wet						325										0 71 29 (SI+CL)		
			16	SS	42		324										0 4 94 2		

Continued Next Page

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

ONTMT4S 414.GPJ 24/11/05

METRIC

+³, ×³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 414-108

4 OF 4

METRIC

W.P. 750-93-01 LOCATION N 5 077 437.9 E 312 903.6 Ottawa Avenue NBL ORIGINATED BY GA
HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Core Barrel/Dynamic Cone COMPILED BY HS
DATUM Geodetic DATE 21.10.05 - 22.10.05 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			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				W P	W	W L		
								20 40 60 80 100								
								○ UNCONFINED + FIELD VANE								
								● QUICK TRIAXIAL × LAB VANE								
								20 40 60 80 100								
310.3			20	SS	100		311									
31.4	GNEISS BEDROCK Fresh to slightly weathered, thinly bedded, pink with black subvertical banding		1	RUN	.00		310									RUN 1# TCR=100%, SCR=100%, RQD=100%, UCS=165MPa
			2	RUN			309									RUN 2# TCR=100%, SCR=100%, RQD=100%, UCS=147MPa
307.3							308									
34.4	END OF BOREHOLE AT 34.37 m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.05m slotted screen. WATER LEVEL READINGS: DATE DEPTH(m) Oct. 24, 05 0.30 Oct. 26, 05 0.32 Oct. 27, 05 0.31 Nov. 5, 05 0.23 Nov. 7, 05 0.20 Nov. 15, 05 0.40 Nov. 17, 05 0.35															

+³, ×³: Numbers refer to
Sensitivity

20
15 10 5
(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 414-109

1 OF 4

METRIC

W.P. 750-93-01 LOCATION N 5 077 463.3 E 312 881.2 Ottawa Avenue NBL ORIGINATED BY MEF
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Core Barrel/Dynamic Cone COMPILED BY HS
 DATUM Geodetic DATE 31.10.05 - 01.11.05 CHECKED BY AEG

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				
344.6							20 40 60 80 100	○ UNCONFINED + FIELD VANE	PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
0.0	Sandy SILT Loose Brown Dry		1	SS	7		20 40 60 80 100	● QUICK TRIAXIAL × LAB VANE	WATER CONTENT (%)			
343.8												
0.8	SILT, trace to some sand, trace clay Dense Brown Dry		2	SS	32		344					
			3	SS	30		343					0 13 84 3
	Becoming Wet, Compact		4	SS	26		342					
			5	SS	14		341					
			6	SS	24		340					
			7	SS	19		339					0 5 91 4
			8	SS	19		338					
							337					
336.4												
8.2	SAND, fine grained, trace to some silt Dense Brown Wet		9	SS	35		336					
							335					

Continued Next Page

+³ ×³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 414-109

2 OF 4

METRIC

W.P. 750-93-01 LOCATION N 5 077 463.3 E 312 881.2 Ottawa Avenue NBL ORIGINATED BY MEF
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Core Barrel/Dynamic Cone COMPILED BY HS
 DATUM Geodetic DATE 31.10.05 - 01.11.05 CHECKED BY AEG

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						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
							WATER CONTENT (%)							
							20 40 60							
							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _P W W _L							
			10	SS	34		334							0 81 19 (SI+CL)
	Becoming Compact		11	SS	20		333							
			12	SS	14		331							
	Becoming Loose		13	SS	7		330							
			14	SS	31		328							
	Becoming Dense		15	SS	37		327							
			16	SS	29		325							0 91 9 (SI+CL)
	Becoming Compact													

Continued Next Page

+³ × 3³ : Numbers refer to
Sensitivity

20
15 5
10 (%) STRAIN AT FAILURE

METRIC

[illegible]

+ 3, × 3: Numbers refer to Sensitivity

ONTMT4S 414.GPJ 24/11/05

METRIC

+ 3, × 3: Numbers refer to Sensitivity

METRIC

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

DNTMT4S 414.GPJ 24H1/05

METRIC

COIL PROFILE	SAMPLE NO.		DYNAMIC CONE PENETRATION			
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+³, ×³: Numbers refer to Sensitivity

ONTMT4S 414.GPJ 24/11/05

METRIC[illegible]Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 414-110

4 OF 4

METRIC

W.P. 750-93-01 LOCATION N 5 077 465.3 E 312 889.6 Ottawa Avenue NBL ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing/NQ Core Barrel/Dynamic Cone COMPILED BY HS
 DATUM Geodetic DATE 24.10.05 - 25.10.05 CHECKED BY AEG

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					
			1	RUN			313									1	RUN 2# TCR=100%, SCR=83%, RQD=78%, UCS=124MPa
																1	
																0	
			2	RUN			312									0	
																2	
311.6																1	
32.0	END OF BOREHOLE AT 32.00m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.05m slotted screen.																
	WATER LEVEL READINGS: DATE DEPTH(m) Oct. 26, 05 2.91 Oct. 27, 05 2.78 Nov. 5, 05 2.50 Nov. 7, 05 2.40 Nov. 15, 05 2.68 Nov. 17, 05 2.63																

RECORD OF BOREHOLE No 414-111

1 OF 2

METRIC

W.P. 750-93-01 LOCATION N 5 077 417.1 E 312 904.6 Ottawa Avenue NBL ORIGINATED BY GA
HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing COMPILED BY HS
DATUM Geodetic DATE 23.10.05 - 23.10.05 CHECKED BY AEG

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			20 40 60 80 100				
								SHEAR STRENGTH kPa				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				
						WATER CONTENT (%)						
						PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT						
						w _p w w _L						
343.0												
0.0	TOPSOIL (50mm)											
0.0	SAND, fine to medium grained		1	SS	14							
	Compact											
	Brown											
	Dry											
	(FILL)											
341.5			2	SS	10							
1.5	SILT, trace sand, trace clay											
	Loose		3	SS	8							
	Brown											
	Moist											
340.8												
2.3	SILT and SAND, trace clay											
	Compact to Loose		4	SS	15							
	Brown											
	Wet											
			5	SS	9							
339.1												
4.0	SILT, trace sand, trace clay											
	Compact to Very Dense		6	SS	27							
	Brown											
	Wet											
			7	SS	31							
			8	SS	76							
334.5												
8.5	Silty SAND, fine to medium grained											
	Very Dense		9	SS	57							
	Brown											
	Wet											

Continued Next Page

+³ × 3³: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

METRIC

ELEV DEPTH	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 (%)
	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100		SHEAR STRENGTH kPa	20 40 60		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					
								20 40 60 80 100					

[illegible]

RECORD OF BOREHOLE No 414-112

1 OF 2

METRIC

W.P. 750-93-01 LOCATION N 5 077 480.3 E 312 880.9 Ottawa Avenue NBL ORIGINATED BY GA
 HWY 11 BOREHOLE TYPE Hollow Stem Augers/NW Casing COMPILED BY HS
 DATUM Geodetic DATE 26.10.05 - 26.10.05 CHECKED BY AEG

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		
343.3														
0.0	SILT, trace to some sand, trace clay Compact Brown Dry		1	SS	10		343							
			2	SS	18		342							
			3	SS	25									
341.0														
2.3	SAND, trace to some silt Compact Brown Wet		4	SS	15		341							
			5	SS	10		340							
339.3														
4.0	SILT, trace to some sand, trace clay Dense Brown Wet		6	SS	34		339							
			7	SS	37		338							
			8	SS	30		337							
			9	SS	50/ .150		336							
334.1							335							
9.1	SAND and SILT, trace clay Very Dense Brown Wet						334							

Continued Next Page

+³ ×³: Numbers refer to
Sensitivity

20
15
10
(%) STRAIN AT FAILURE

METRIC

[illegible]

Appendix B

Laboratory Test Results

FIGURE B1

Size of openings, inches

U.S.S. Sieve size, meshes/inch

6" 4 1/4" 3" 1 1/2" 1" 3/4" 1/2" 3/8" 3 4 8 10 16 30 40 50 60 100 200

100 10 1 0.1 0.01 0.001 0.0001

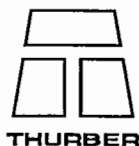
PERCENT FINER THAN

GRAIN SIZE, mm

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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	414-107	2.77	339.08

Date March 2006
Project 749-93-01



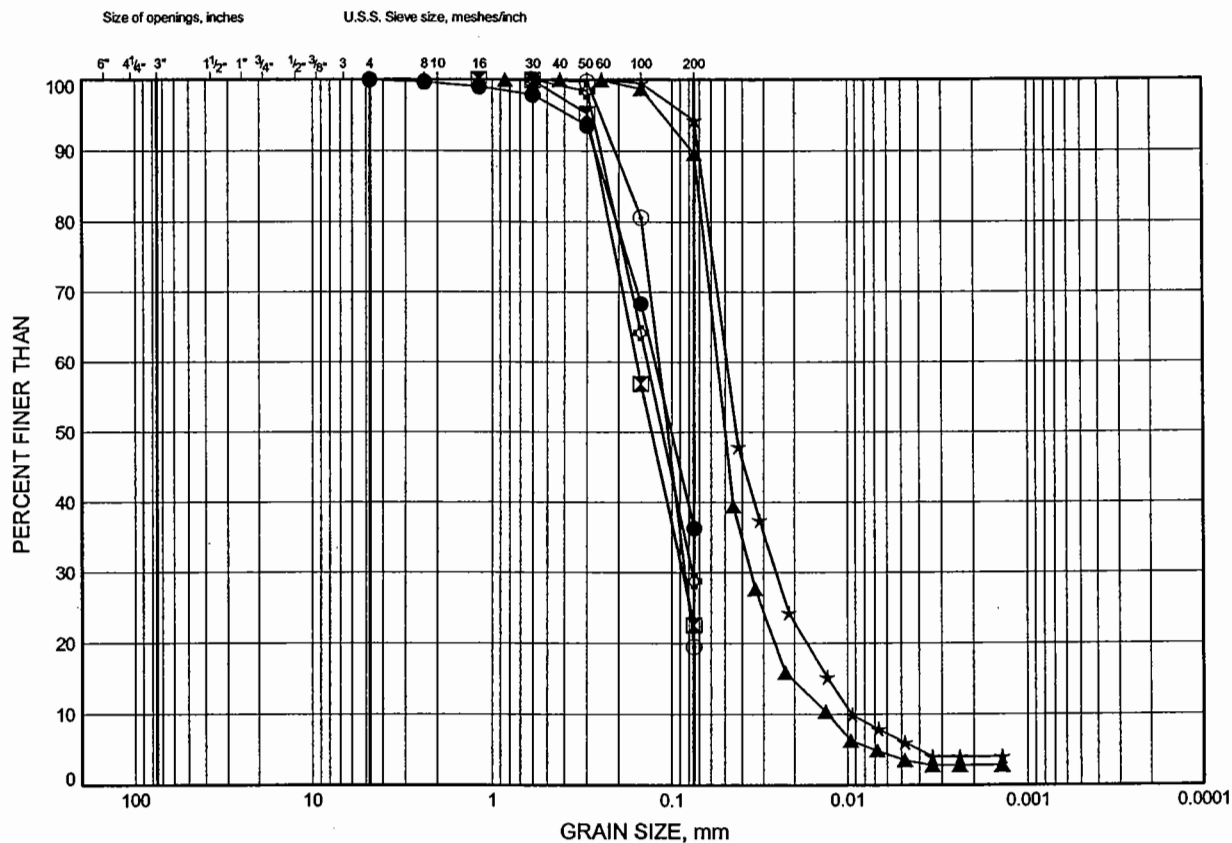
Prep'd JHL
Chkd. MEF

Hwy 11 Four Laning

GRAIN SIZE DISTRIBUTION

FIGURE B2

SILT TO SILTY SAND

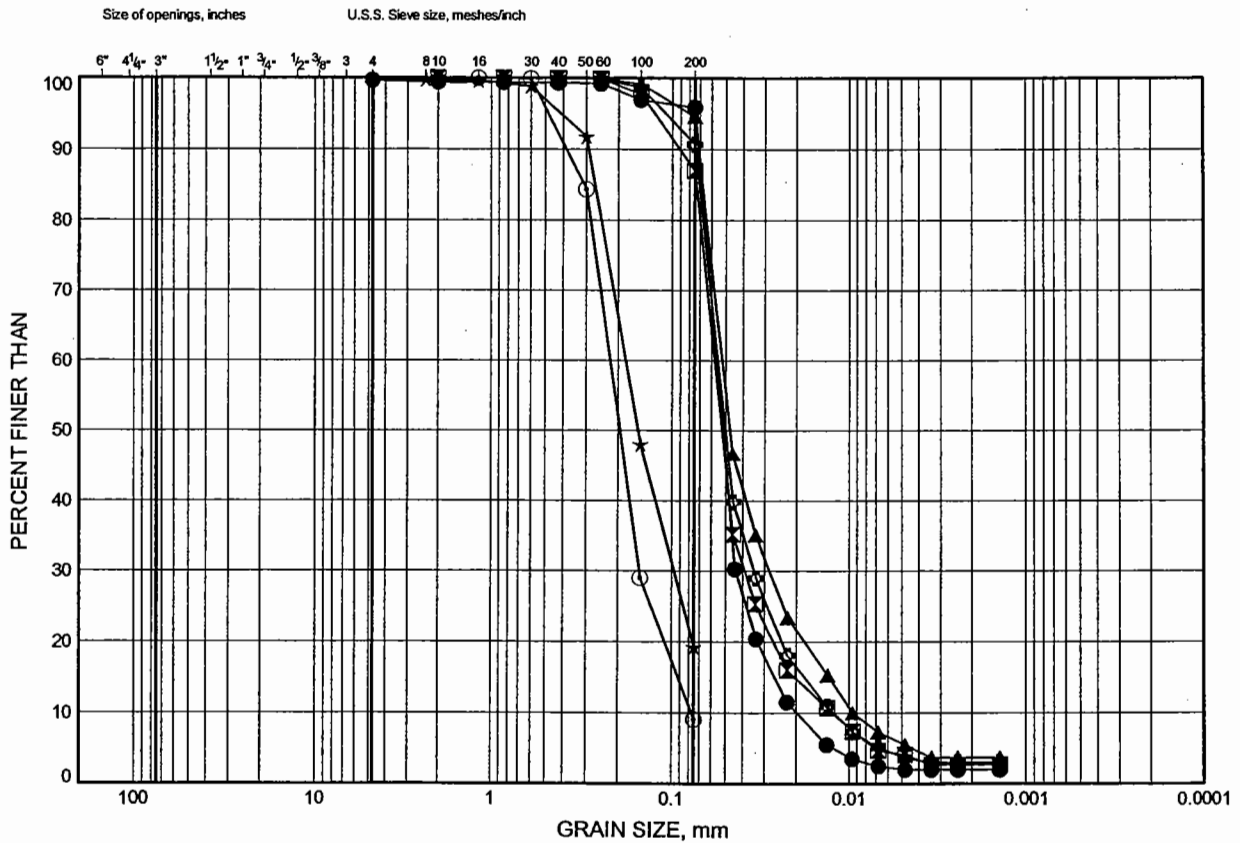


Hwy 11 Four Laning

GRAIN SIZE DISTRIBUTION

FIGURE B3

SILT TO SILTY SAND

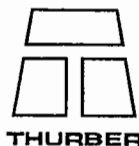


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	414-108	19.43	322.22
⊠	414-109	1.83	342.77
▲	414-109	5.89	338.71
★	414-109	10.46	334.14
⊙	414-109	19.61	324.99
⊛	414-110	1.83	341.77

Date March 2006

Project 749-93-01



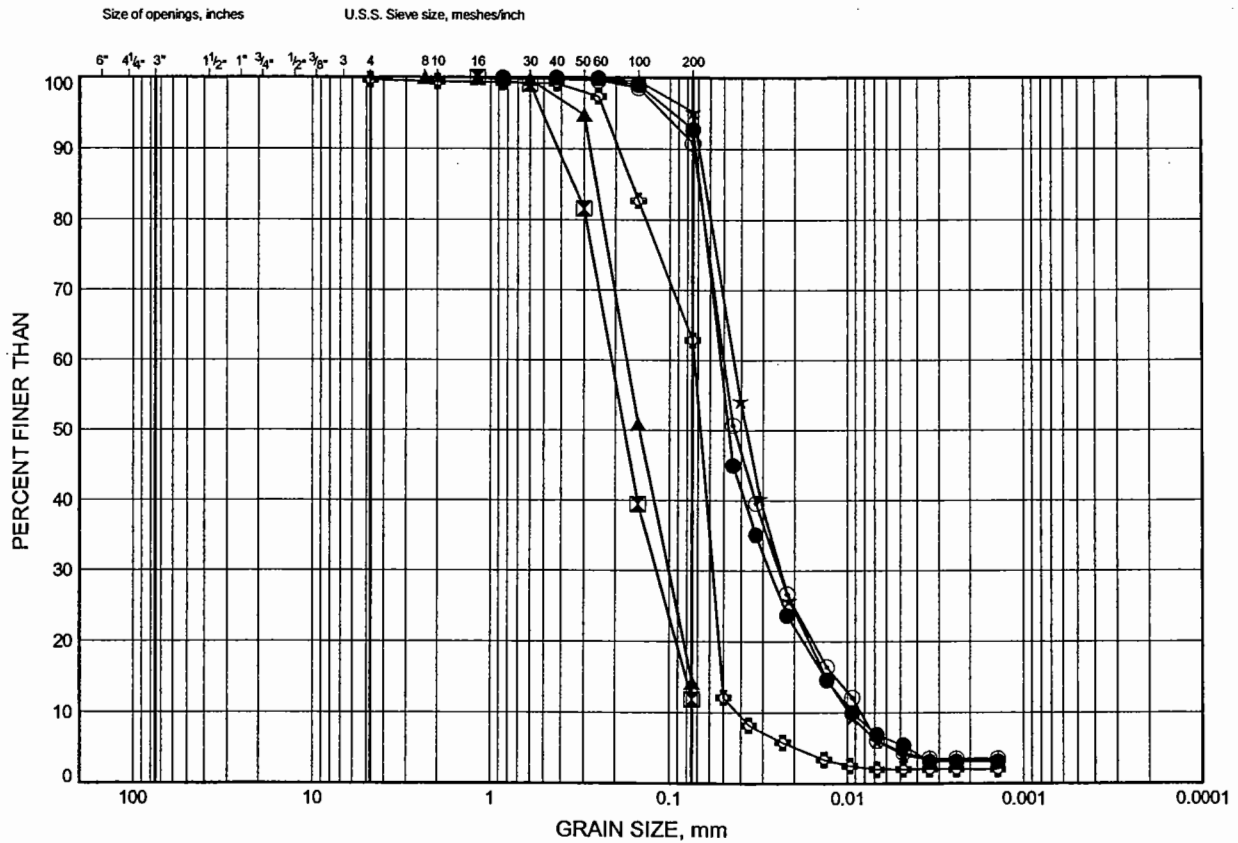
Prep'd JHL

Chkd. MEF

Hwy 11 Four Laning GRAIN SIZE DISTRIBUTION

FIGURE B4

SILT TO SILTY SAND

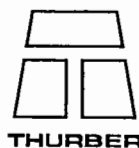


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	414-110	5.72	337.89
⊠	414-110	11.81	331.79
▲	414-110	17.91	325.69
★	414-110	22.48	321.12
⊙	414-111	1.83	341.21
⊕	414-111	3.35	339.69

Date March 2006

Project 749-93-01



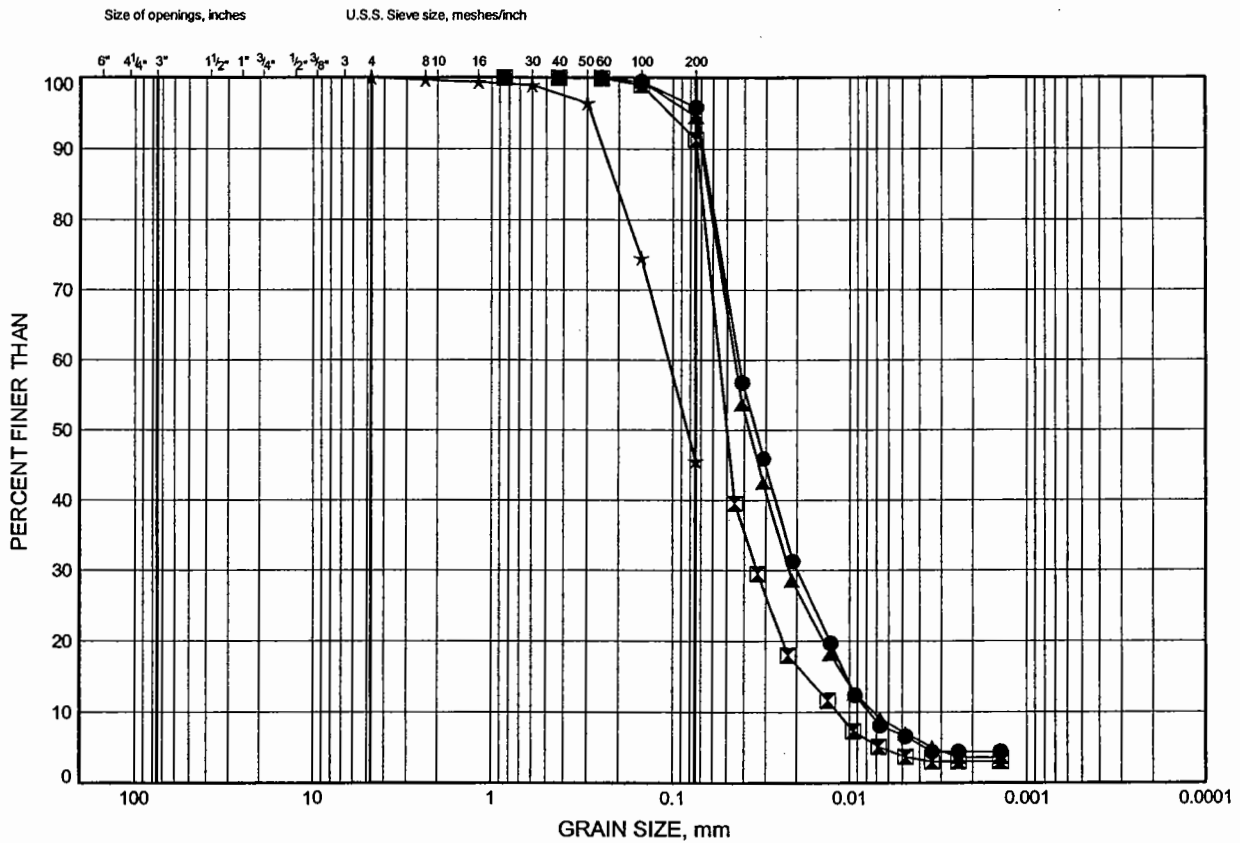
Prep'd JHL

Chkd. MEF

Hwy 11 Four Laning GRAIN SIZE DISTRIBUTION

FIGURE B5

SILT TO SILTY SAND



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	414-111	7.24	335.80
⊠	414-112	1.83	341.46
▲	414-112	5.72	337.57
★	414-112	10.29	333.00

Date March 2006

Project 749-93-01



Prep'd JHL

Chkd. MEF

**TABLE B1 - Point Load Test Results
Ottawa Avenue Overpass NBL**

RUN	Depth			Is50	UCS (MPa)				
	feet	Inches	m						
414-107									
4	106	4	32.41	8.67	208.14	Total Rock Core Average Minimum Maximum MPa 154 120 208			
5	108	2	32.97	5.20	124.89				
	109	9	33.45	5.75	137.90				
	110	8	33.73	5.53	132.69				
	111	6	33.99	7.59	182.13				
6	112	9	34.37	7.26	174.32	Run #	Average		
	115	3	35.13	4.99	119.68	4	208.14		
						5	144.40		
						6	147.00		
RUN	Depth			Is50	UCS (MPa)				
	feet	Inches	m						
414-108									
1	103	7	31.57	6.50	156.11	Total Rock Core Average Minimum Maximum MPa 159 114 208			
	104	7	31.88	4.77	114.48				
	105	7	32.18	6.83	163.91				
	106	10	32.56	7.91	189.93				
	107	9	32.84	8.67	208.14				
	108	10	33.17	6.50	156.11	Run #	Average		
2	109	2	33.27	6.07	145.70	1	164.78		
	110	6	33.68	6.94	166.52	2	148.95		
	111	6	33.99	5.31	127.49				
RUN	Depth			Is50	UCS (MPa)				
	feet	Inches	m						
414-109									
6	124	3	37.87	6.83	163.91	Total Rock Core Average Minimum Maximum MPa 172 62 221			
	125	7	38.28	9.21	221.15				
	126	4	38.28	9.11	218.55				
8	129	11	38.51	2.60	62.44				
	131	1	39.60	6.46	155.07				
	132	0	39.95	7.37	176.92	Run #	Average		
	132	6	40.23	8.72	209.18	6	201.21		
						8	150.90		
RUN	Depth			Is50	UCS (MPa)				
	feet	Inches	m						
414-110									
1	96	6	29.41	6.85	164.43	Total Rock Core Average Minimum Maximum MPa 142 96 179			
	97	3	29.64	5.64	135.29				
	98	0	29.87	5.64	135.29				
	98	10	30.12	6.50	156.11				
	100	0	30.48	6.42	154.03				
	100	10	30.73	7.46	179.00	Run #	Average		
2	101	6	30.94	6.50	156.11	1	154.03		
	102	8	31.29	5.98	143.62	2	124.37		
	103	9	31.62	4.25	101.99				
	104	4	31.80	3.99	95.75				

Appendix C

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Foundation Element	Driven Piles	Augered Caissons	Footings on Native Soil	Footings on Engineered Fill
North & South Abutment	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance available by driving piles to achieve resistance in the very dense soil overlying bedrock. ii. Allows choice of conventional, integral or semi-integral abutment design. iii. Readily installed. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit cost compared to footings. ii. Construction concerns related to the possibility of piles being obstructed by a boulder during driving. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. High geotechnical resistance available for units founded in very dense soil or on bedrock. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Higher unit cost compared to other footing options such as driven piles. ii. High risks associated with inflow of groundwater and soil fines. iii. Relatively high construction effort required to install caissons to bedrock compared to driven piles. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Economical to install. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Comparatively longer abutment stem. ii. Low geotechnical resistance may result in uneconomically large footing sizes. iii Potential for unacceptable settlements. 	<p>Advantages:</p> <ul style="list-style-type: none"> i. Possibility of shortening the abutment height. ii. Higher geotechnical resistance than is available on native soil. iii. Lower cost compared to deep foundations. iv. Allows use of perched abutments. v. Allows choice of semi-integral abutment. <p>Disadvantages:</p> <ul style="list-style-type: none"> i. Lower geotechnical resistance than piles. ii. High cost of constructing engineered fill. iii Footings have to be placed further away from Ottawa Ave. to accommodate forward slope of fill. iv. Potential for unacceptable settlements.
	RECOMMENDED	NOT RECOMMENDED	NOT RECOMMENDED	NOT RECOMMENDED

Appendix D

Special Provisions and NSSP's

The following Special provisions are referenced in this report:

- Amendment to OPSS 206, December 1993
- Special Provision 599S22
- Special Provision No. 902S01
- Special Provision No. 903S01

Suggested text for a NSSP on Pile Installation should contain the following:

“The soil overlying the bedrock contains cobbles and boulders, particularly below Elevation 275. The presence of cobbles and boulders will potentially have an impact on the installation of piles at the site. Some possible impacts that must be taken into consideration include, but are not necessarily limited to:

- *The cobbles and boulders may impede the driving of the piles resulting in more arduous driving*
- *Some piles may meet refusal on boulders that are large enough not to be dislodged or broken by the pile driving*
- *As a result of the presence of boulders, piles may meet refusal at varying depths*
- *Pile driving must be controlled according to the criteria specified for the site*
- *If a pile meets refusal at a depth less than the anticipated depth, the QVE must terminate driving before the pile is damaged due to over-driving*

Suggested text for a NSSP on Dewatering should contain the following:

“The soils underlying this site are cohesionless in nature and the observed groundwater table lies close to the surface. Excavation below the groundwater level is expected to lead to instability and slough of the sides of the excavation and boiling of the base, accompanied by loss in geotechnical resistance of the soils. If excavation is required to be carried out below the groundwater level prevailing at the time of construction, appropriate means of dewatering must be implemented to depress the groundwater level sufficiently far below the base of the excavation to prevent any instability, sloughing, or boiling and so as to preserve the stability of the excavation and to allow the work to proceed in the dry.”

Appendix E

Slope Stability Output

Thurber Engineering Ltd. - Toronto
 19-1423-12 Hwy 11 Burk's Falls
 Ottawa Ave. NBL
 March 9, 2006
 South Approach Rock fill

	Gamma C	Phi	Min	Piezo
	kN/m ³	deg	c/p	Surf.
Rock Fill	20	45	0	1
Silty Sand	21	30	0	1

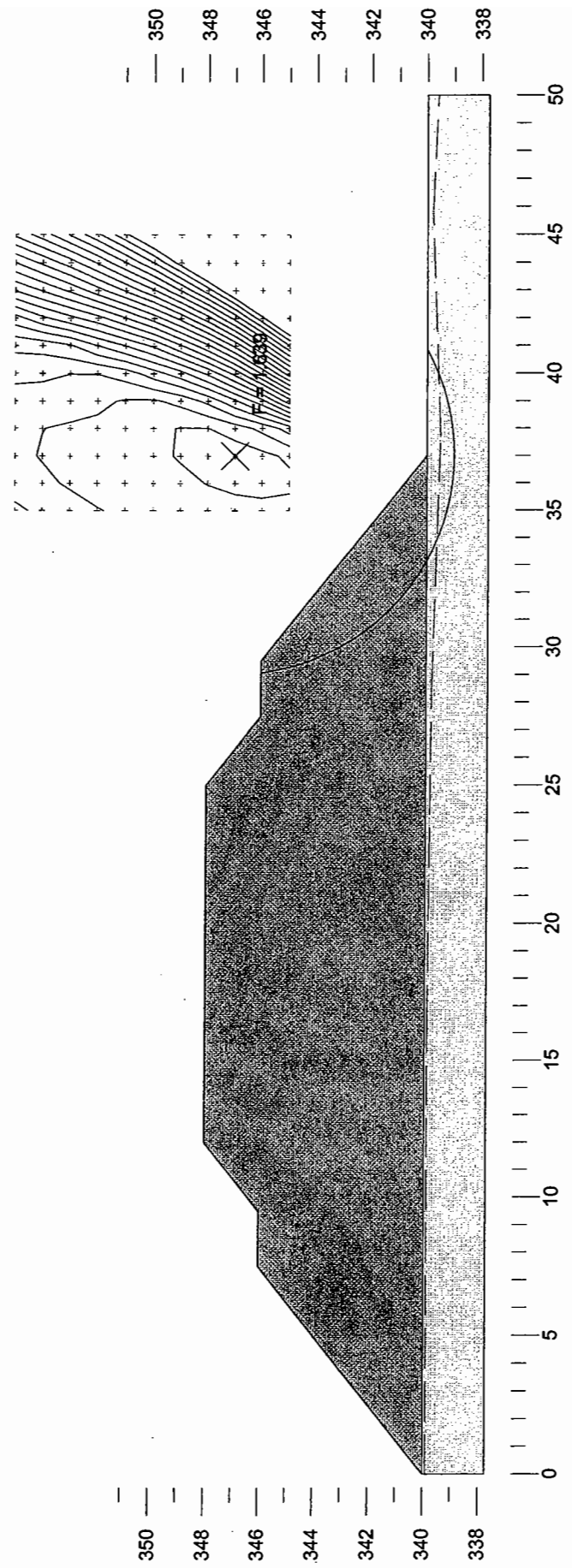


Figure E1

Thurber Engineering Ltd. - Toronto
 19-1423-12 Hwy 11 Burk's Falls
 Ottawa Ave. NBL
 March 9, 2006
 South Approach Rock fill
 Seismic = 0.18

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Rock Fill	20	45	0	1
Silty Sand	21	30	0	1

Seismic coefficient = 0.18

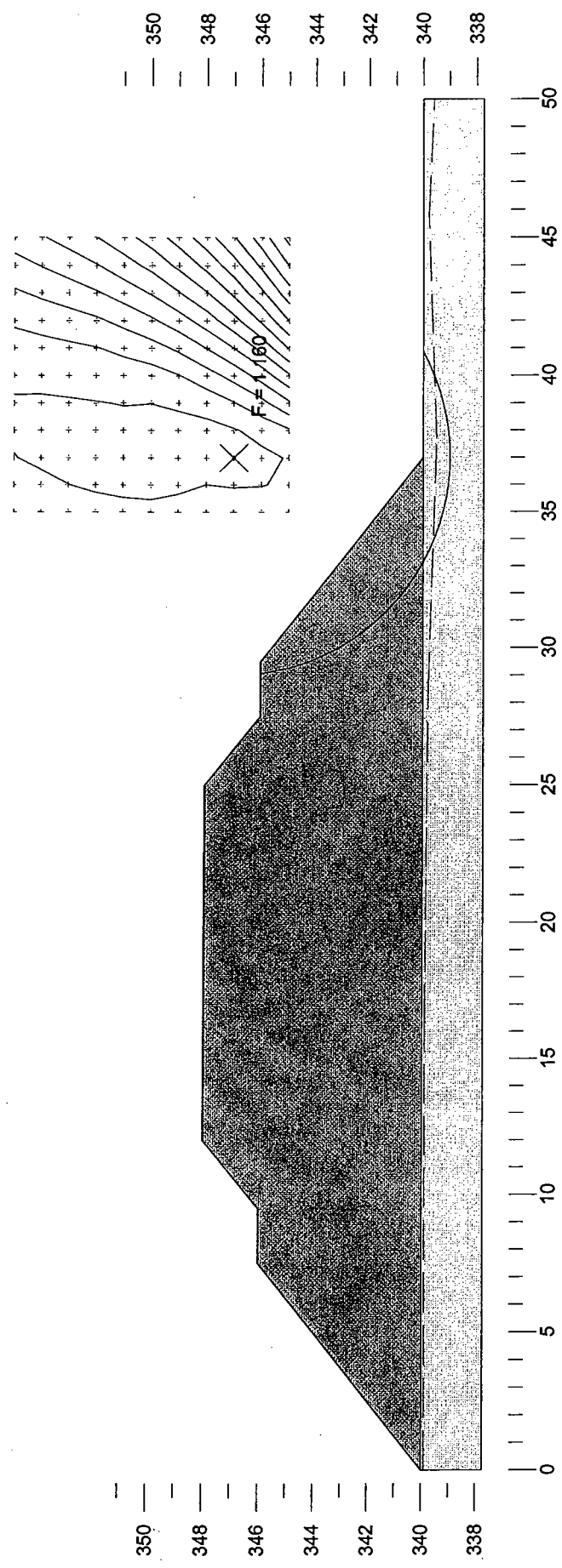


Figure E2

Thurber Engineering Ltd. - Toronto
 19-1423-12 Hwy 11 Burk's Falls
 Ottawa Ave. NBL
 March 9, 2006
 South Approach Earth Fill

Gamma C	Phi	Min	Piezo
kN/m ³	deg	c/p	Surf.
21	0	0	1
21	0	0	1

Earth Fill
 Silty Sand

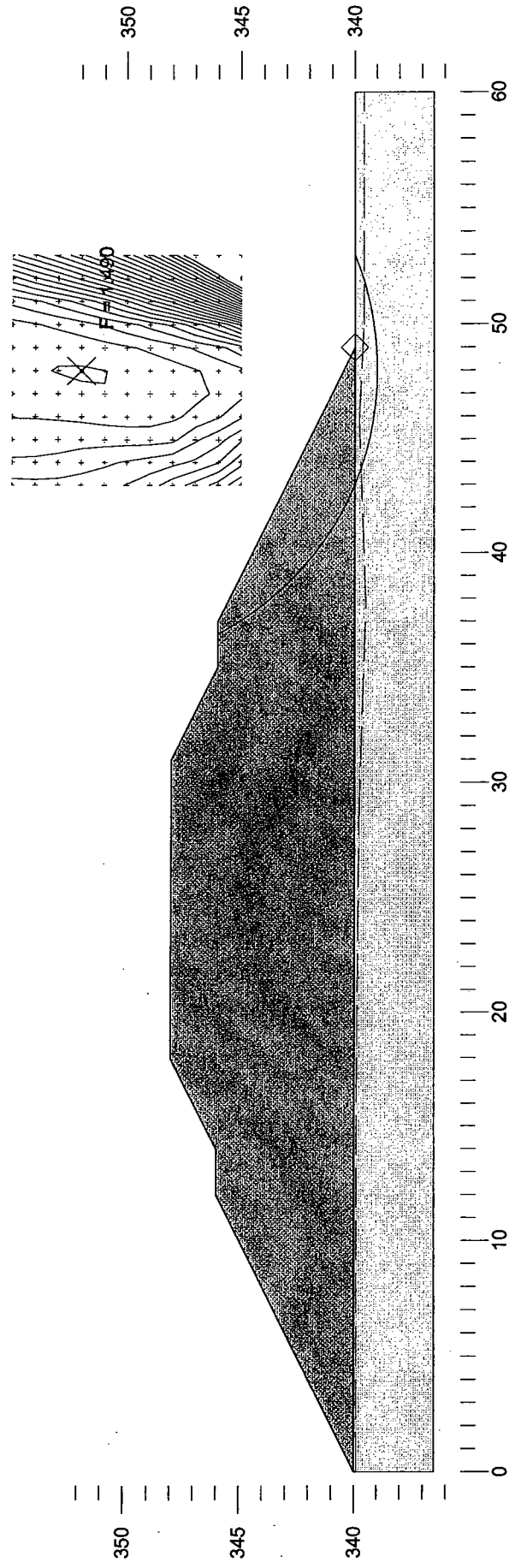


Figure E3

Thurber Engineering Ltd. - Toronto
 19-1423-12 Hwy 11 Burk's Falls
 Ottawa Ave. NBL
 March 9, 2006
 South Approach Earth Fill
 Seismic 0.18

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Earth Fill	21	30	0	1
Silty Sand	21	30	0	1

Seismic coefficient = 0.18

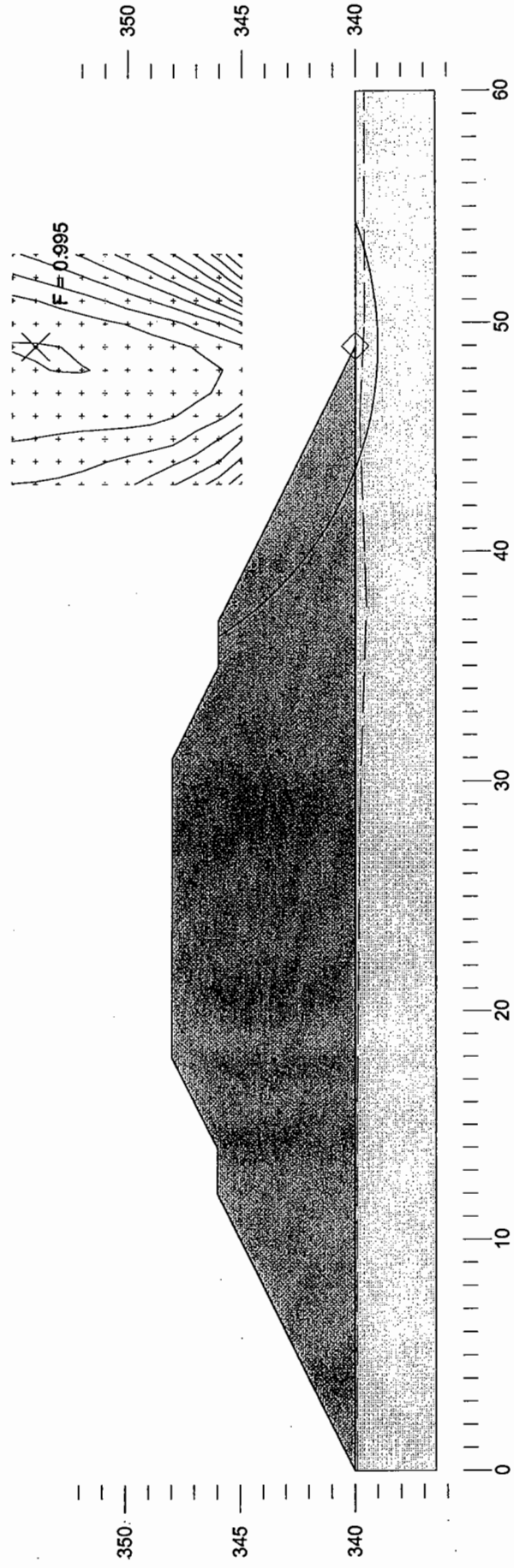


Figure E4

Thurber Engineering Ltd. - Toronto
 19-1423-12 Hwy 11 Burk's Falls
 Ottawa Ave. NBL
 March 9, 2006
 North Approach Rock fill

	Gamma C	Phi	Min	Piezo
	kN/m ³	deg	c/p	Surf.
Rock Fill	20	45	0	1
Silty Sand	21	30	0	1

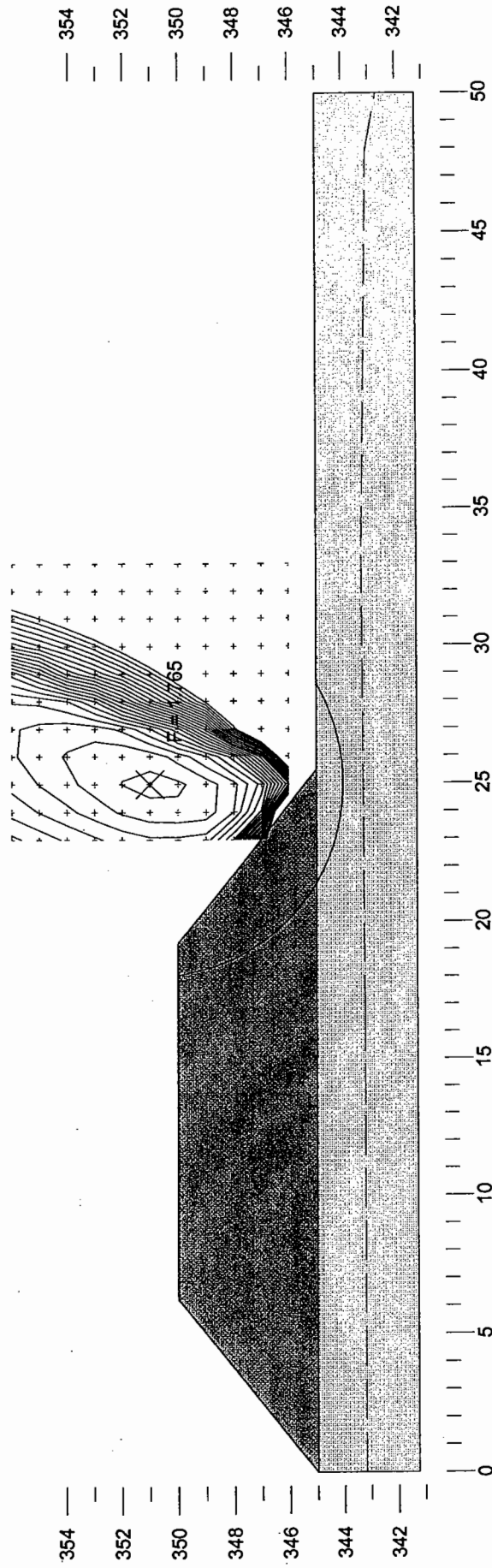


Figure E5

Thurber Engineering Ltd. - Toronto
 19-1423-12 Hwy 11 Burk's Falls
 Ottawa Ave. NBL
 March 9, 2006
 North Approach Rock fill
 Seismic 0.18

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Rock Fill	20	45	0	1
Silty Sand	21	30	0	1

Seismic coefficient = 0.18

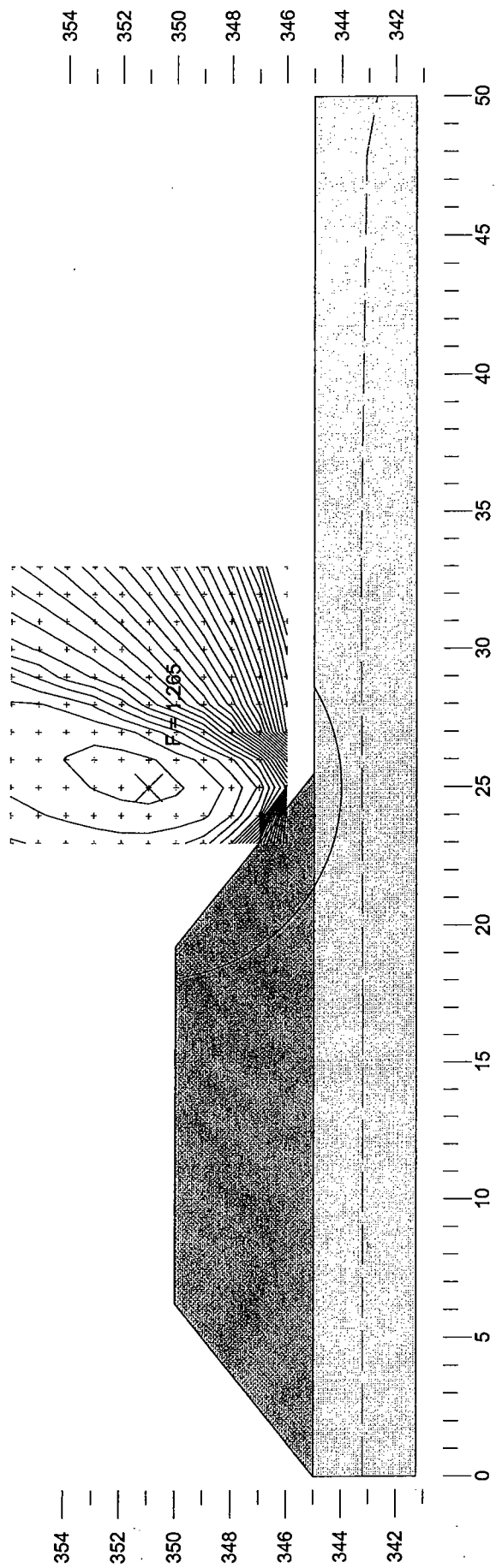


Figure E6

Thurber Engineering Ltd. - Toronto
 19-1423-12 Hwy 11 Burk's Falls
 Ottawa Ave. NBL
 March 9, 2006
 North Approach Earth Fill

Earth Fill	Gamma C	Phi	Min	Plezo
Silty Sand	kN/m3	deg	c/p	Surf.
	21	30	0	1
	21	30	0	1

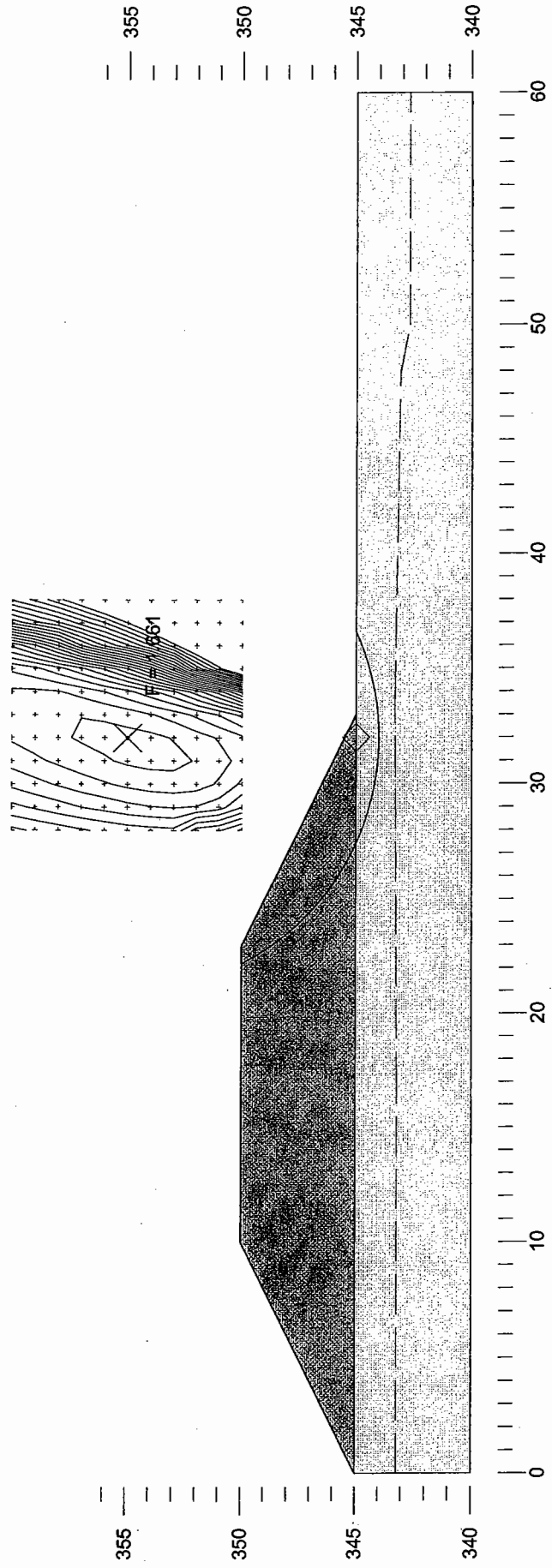


Figure E7

Thurber Engineering Ltd. - Toronto
 19-1423-12 Hwy 11 Burk's Falls
 Ottawa Ave. NBL
 March 9, 2006
 North Approach Earth Fill
 Seismic 0.18

	Gamma C	Phi	Min	Piezo
	kN/m3	deg	c/p	Surf.
Earth Fill	21	0	0	1
Silty Sand	21	0	0	1
Seismic coefficient = 0.18				

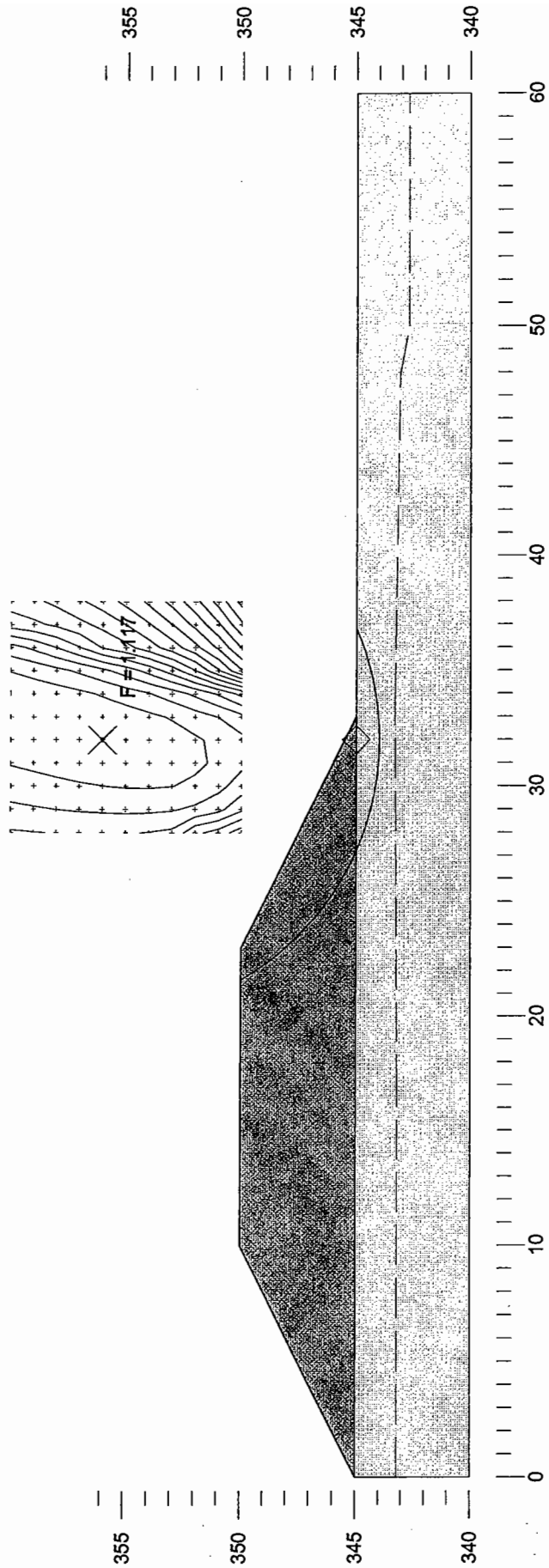


Figure E8

Thurber Engineering Ltd. - Toronto
 19-1423-12
 Ottawa Ave NBL
 March 31, 2006
 South Abutment RSS Walls
 RSS Height 2.7m

	Gamma C	Phi	Piezo
	kN/m ³	deg	Surf.
Gran A Cap	22.8	0	35
RSS Core	21.2	0	32
Approach Fill	21.2	0	32
Gran Pad	22.8	0	35
Sand & Silt	20	0	30
Slp Circ Limit	(Infinitely Strong)		

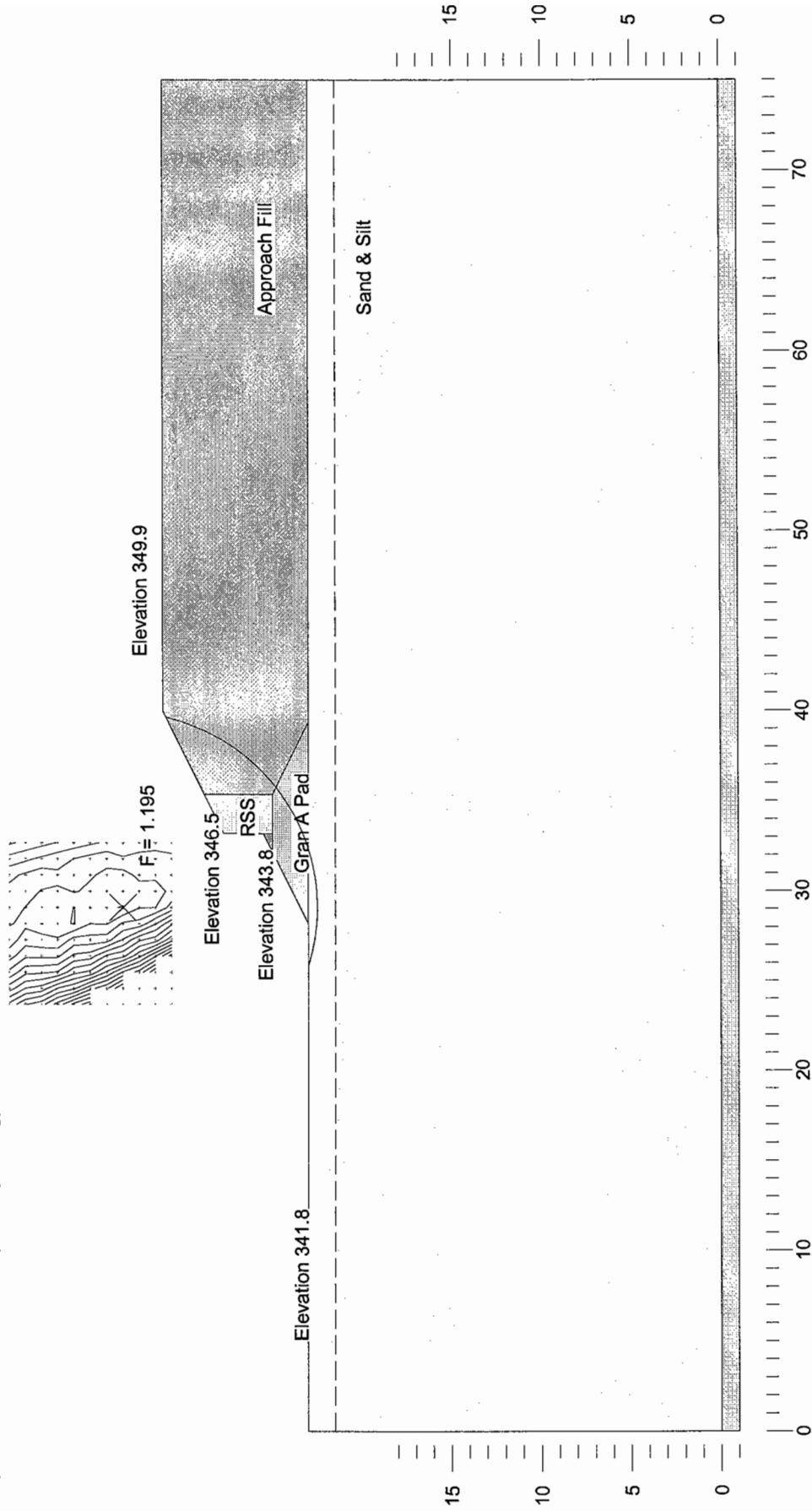
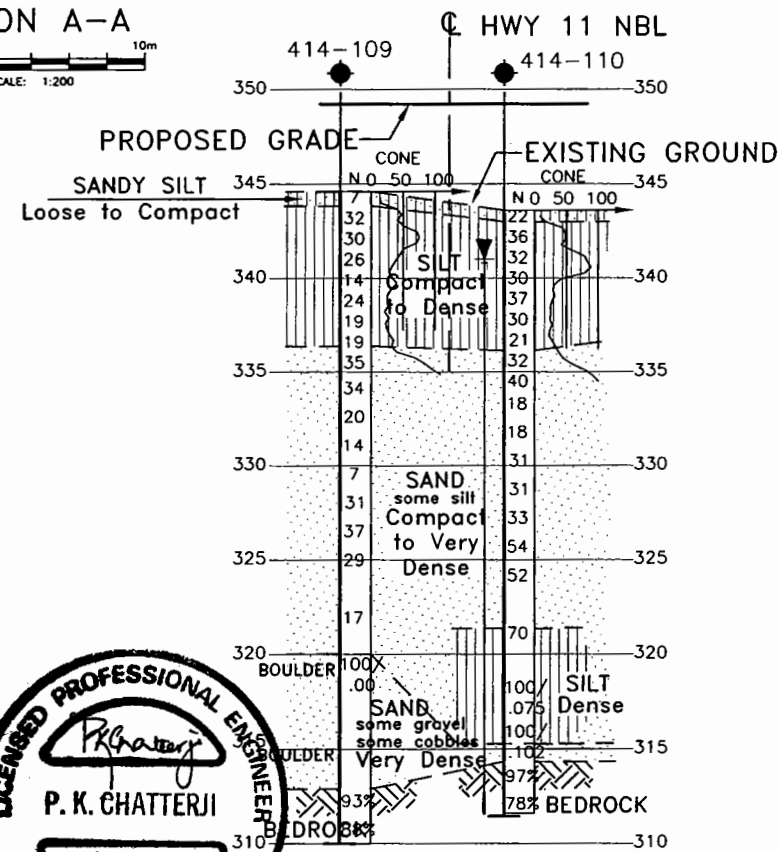
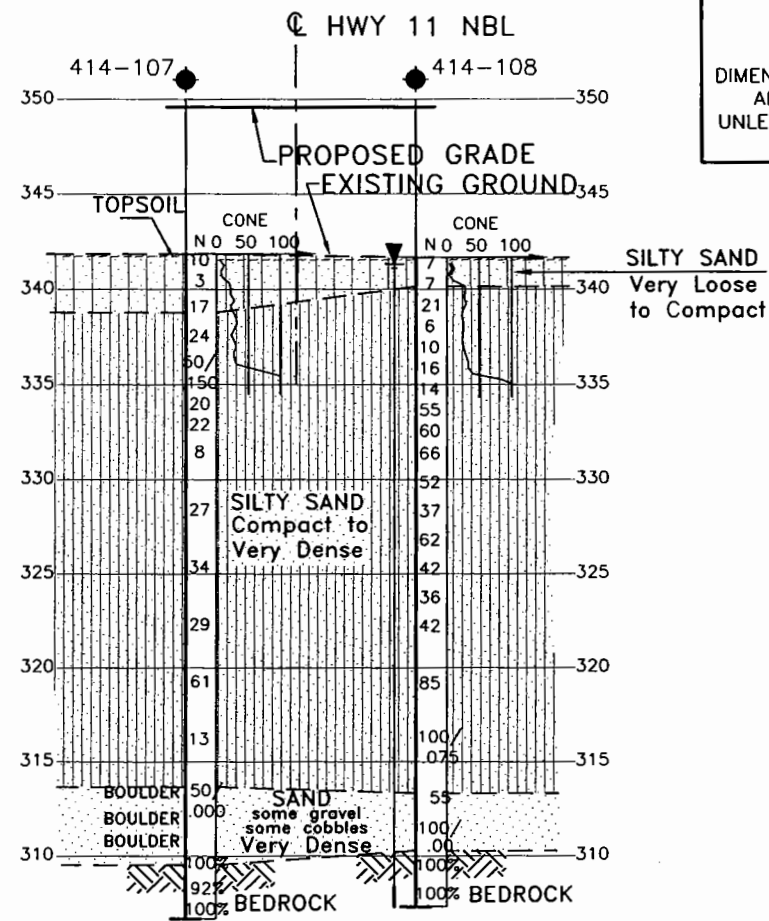
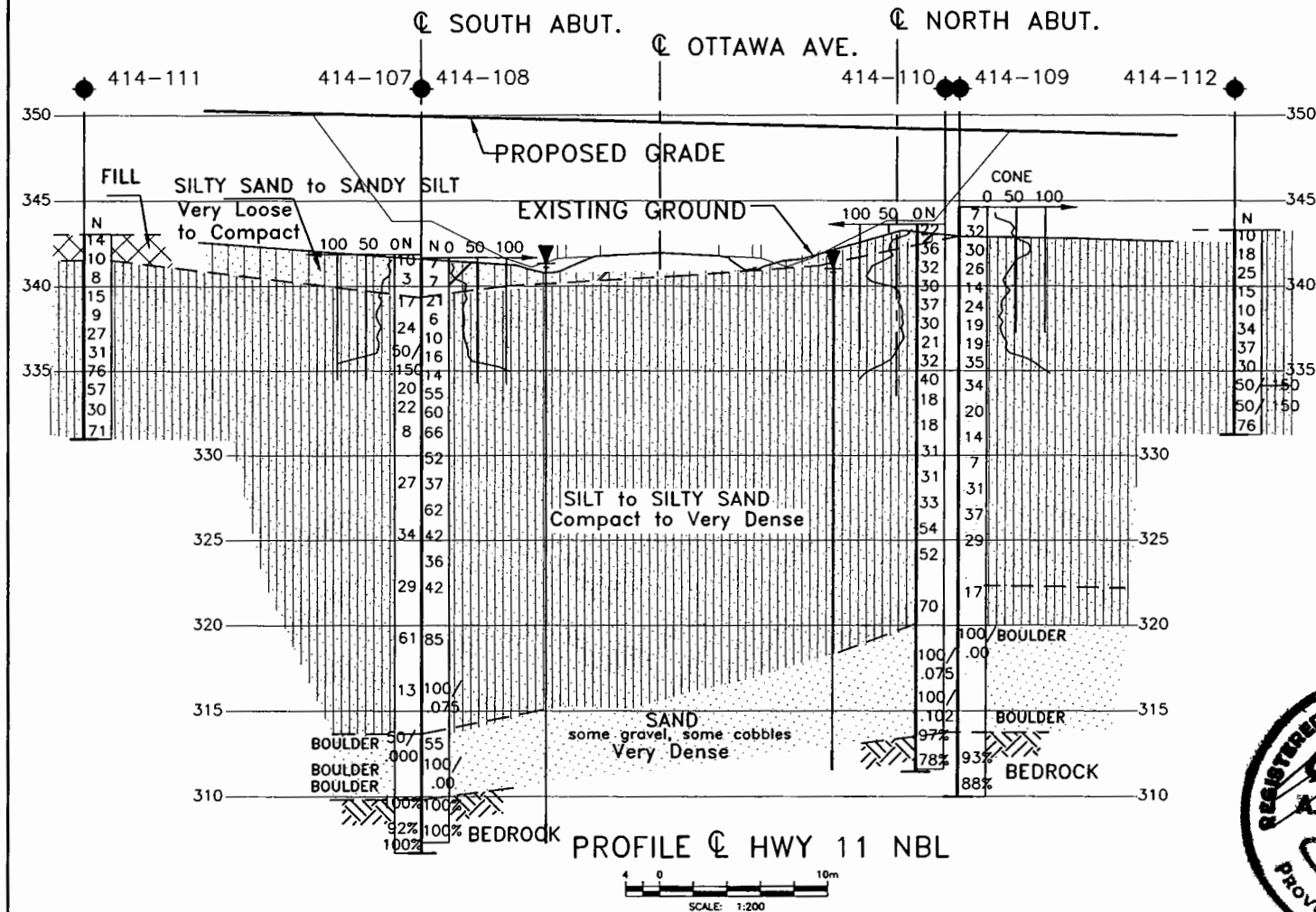
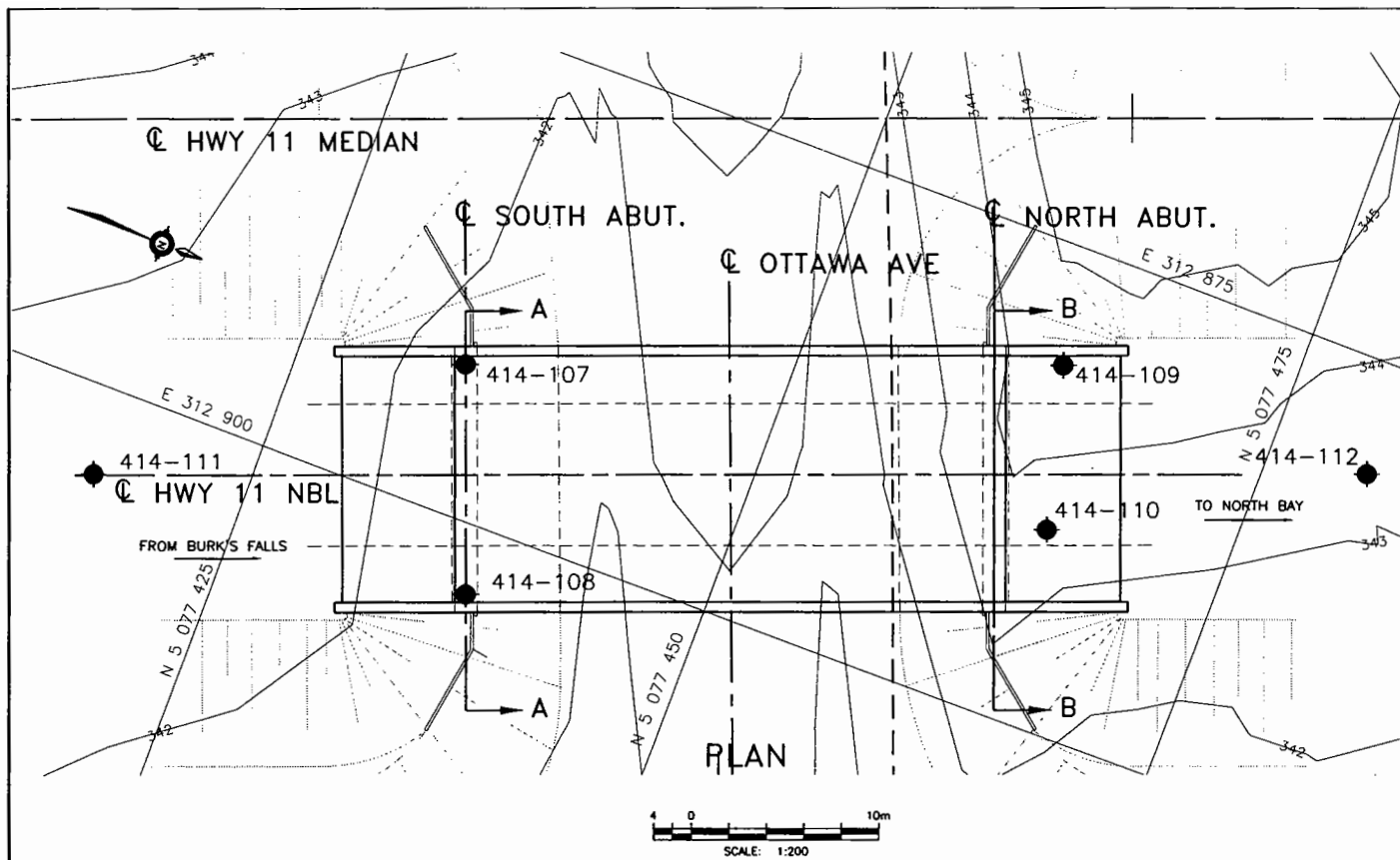


Figure E9

Appendix F

Drawing

Borehole Locations and Soil Strata



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

HWY 11
CONT No 2006-5151
WP No 749-93-01

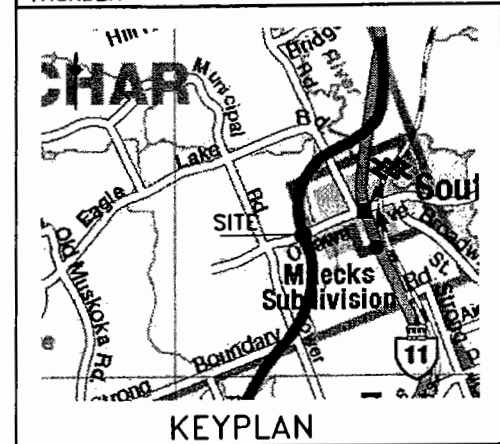


OTTAWA AVENUE I/C
OVERPASS NBL
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET
551



THURBER ENGINEERING LTD.



LEGEND

- Bore Hole By Thurber
- ⊙ Bore Hole By Golder
- N Blows/ 0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/ 0.3m (60° Cone, 475 J/blow)
- PH Pressure, Hydraulic
- WL at Time of Investigation
- Head Artesian Water
- Piezometer
- 90% Rock Quality Designation (RQD)

NO	ELEVATION	NORTHING	EASTING
414-107	341.8	5077433.6	312892.2
414-108	341.7	5077437.9	312903.6
414-109	344.6	5077463.3	312881.2
414-110	343.6	5077465.5	312889.6
414-111	343.0	5077417.1	312904.6
414-112	343.3	5077480.3	312880.9

NOTE
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.



BENCHMARK

SECTION B-B
DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

DATE	BY	DESCRIPTION
DESIGN AEG	CHK PKC	CODE CHBDC 2000/LOAD CL-625-001/DATE NOV 2005
DRAWN SS	CHK AEG	SITE 44-414/1/STRUCT. SCHEME DWG 2