

GEOCRES No. 31E-125

DIST. 52 REGION

W.P. No. 462-93-00

CONT. No. _____

W. O. No. _____

STR. SITE No. _____

HWY. No. 11

LOCATION Jessop's Creek Culvert

No. of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. _____

REMARKS: _____

**FINAL FOUNDATION INVESTIGATION AND DESIGN REPORT FOR
PROPOSED JESSOP'S CREEK CULVERT
HIGHWAY 11 FOUR LANING
6.7 km NORTH OF HWY 60 NORTHERLY 13 km
W.P. 462-93-00,
DISTRICT 52, HUNTSVILLE**

Report

to

McCormick Rankin Corporation

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February 3, 1999
File: 19-1351-7h

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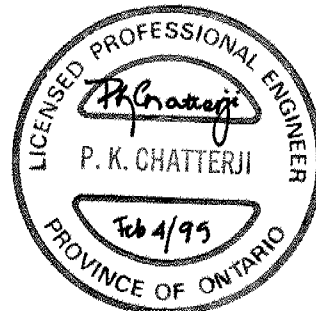


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Figure 3 Configuration of Rock Fill with Trapezoidal Wedge Gap and
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APPENDICES

Appendix A Borehole Logs
Appendix B Laboratory Test Results

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1. INTRODUCTION

This report presents the results of the foundation investigation and analysis carried out by Thurber Engineering Ltd. (Thurber) at the site of the proposed culvert which will carry Jessop's Creek under Highway 11 in Huntsville, Ontario. The purpose of the investigation was to explore the subsurface soil and groundwater conditions at the site and based on the data obtained provide borehole logs, soil profile and a written description of the subsurface conditions. The purpose of the analysis of the data obtained during the investigation was to produce recommendations for the design and construction of the culvert and associated backfill.

Thurber carried out the investigation as a sub-consultant to McCormick Rankin Corporation (MRC) under Ministry of Transportation (MTO) Agreement 9750 - 7424 - 5262.

2. SITE DESCRIPTION

2.1 Site Location

The subject site lies in the Right-of-Way (ROW) of the four-laning of Highway 11 north of Huntsville and is located where the ROW crosses Jessop's Creek. The site lies at Station 22+780, approximately, C/L Median Highway 11, on the north side of the community of Melissa.

At the site, the existing highway is carried on an embankment which is contiguous to, and continuous with the embankment immediately to the west which carries the CN Rail tracks. Jessop's Creek is carried under the

two embankments in a continuous culvert. The existing culvert under the highway is a 3.5 m diameter CSP. The remnants of an older concrete structure, apparently built to carry an earlier highway alignment over the creek, was observed partially buried between the existing highway and the railway embankment.

To the east of the existing embankment Jessop's creek flows across the flood plain to join with the Little East River.

2.2 Physiography

Based on The Physiography of Southern Ontario, 3rd Edition, by Chapman and Putnam, the region surrounding the site consists of bedrock ridges with shallow overburden. The bedrock is undifferentiated igneous and metamorphic rock of early Precambrian age and is generally hard and massively jointed.

Along this part of the Highway 11 corridor, the troughs between the exposed bedrock ridges are generally filled with fine sand deposits and muskeg.

The area of interest is bounded to the west by the railway embankment and to the east by the flood plain of the Little East River. The flood plain is underlain by a deposit of fine sand and silts typical of the area. Further east rock outcrops rise steeply above the flood plain. The area east of the existing highway is generally wooded.

3. INVESTIGATION PROCEDURES

3.1 Field Investigation

On November 12 and 13, 1998, a Nodwell track mounted auger rig was used on site for drilling and Standard Penetration Testing (SPT, following the procedures of ASTM D 1586). One sampled borehole (H-98-1) was drilled from the bench located approximately half way up the railway embankment to sample both the soils in the embankment and the underlying native soils. A second hole (H-98-2) was drilled on the flood

plain to the east of the existing highway for the purposes of exploring the native soil which will underlie the proposed culvert. The approximate locations of the boreholes are shown on Drawing 19-1351-7h-01.

The holes were advanced using hollow stem augers which were kept full of drilling mud at all times to counteract the effect of an unbalanced head of groundwater in the fine sand and silt soils encountered at the site.

The depths of sampling in the two boreholes were as follows:

Borehole No.	Depth of Sampling (m)
H-98-1	17.4
H-98-2	8.2

Samples were recovered at selected intervals throughout the depths of the boreholes in conjunction with Standard Penetration Tests (SPT) (following the test procedure outlined in ASTM D 1586). Samples were generally recovered at intervals of 0.75 m in the upper 3.0 m and thereafter at intervals of 1.5 m.

A standpipe piezometer was installed in each borehole on completion of sampling.

All recovered samples were examined, identified and logged in the field and were transported to Thurber's Toronto laboratory by the field supervisor for further examination and laboratory analysis.

The results of the drilling and sampling are summarized on the borehole logs in Appendix A.

3.2 Laboratory Analysis

Geotechnical laboratory testing consisted of natural moisture content determinations and visual classifications of all recovered samples. In addition, grain size analyses were conducted on selected samples. The

results of the laboratory testing are presented on the borehole logs in Appendix A, and in Figures B1 and B2 in Appendix B. The results of pH and sulphate testing are presented in Table B1 in Appendix B.

4. DESCRIPTION OF SUBSURFACE CONDITIONS

4.1 Subsurface Soil Conditions

The soils encountered on this site consist of the native soil and the fill placed to construct the existing railway and highway embankments. It is apparent from visual examination that the embankments were constructed using locally obtained fill. In Borehole H-98-1, it was not possible to make a clear differentiation between the embankment fill and the underlying native soil either by visual examination or on the basis of grain size distribution. The native soil encountered in the borehole to the east of the highway, Borehole H-98-2, was also indistinguishable from the soils encountered in Borehole H-98-1. Consequently, the same description has been applied to all the soils on this site.

The soils lie in a narrow envelop in the fine silty sand to sand and silt range. Only one of the tested samples, Sample 10 at a depth of 11 m in Borehole H-98-1 exhibited trace gravel. The sand fill in the embankment is in a compact to dense state, based on SPT values ranging from 20 to 42, though one value of 69 was recorded which indicates locally very dense conditions. In the embankment fill, measured moisture contents ranged from 5 to 9%.

The native sand was found to be in a loose to compact state based on SPT values of 4 to 22. A locally dense zone in the native soil under the embankment was indicated by an SPT value of 31 at a depth of 14 m. The measured moisture contents in the native soil generally ranged from 19 to 27%, with two higher values near the surface in Borehole H-98-2 due to the presence of organic material.

4.2 Groundwater

On completion of drilling and sampling, groundwater was noted at the following depths in the open boreholes:

Borehole Number	Depth to Water (m)
H-98-1	7.26
H-98-2	0.25

The water levels noted in the standpipe piezometers installed in the boreholes are as follows:

Date	Water Level Depths (Elevations) (m)	
	H-98-1	H-98-2
November 12, 1998	N/A	0.19 (303.1)
November 13, 1998	8.52 (306.2)	0.18 (303.1)
November 16, 1998	8.81 (305.9)	0.21 (303.1)
November 17, 1998	8.78 (305.9)	0.22 (303.1)
November 18, 1998	8.74 (306.0)	0.20 (303.1)
December 3, 1998	9.06 (305.6)	0.22 (303.1)

The groundwater levels at the site are subject to fluctuation and will be influenced by such factors as recent rainfall and the level of the creek.

4.3 Disturbance in Excavation

The soils encountered at this site will be susceptible to base boiling and disturbance in any excavations which may be carried down below the groundwater table prevailing at the time of construction. This condition will be due to the unbalanced hydrostatic head as the excavation is taken below the water table. If any such excavation is required, consideration should be given to prior unwatering to lower the water table to at least 1.0 m below the base of the excavation.

5. RECOMMENDATIONS FOR STRUCTURE FOUNDATIONS

5.1 Type of Structure

The proposed structure consists of a concrete box culvert measuring 3.7 x 3.9 m. The culvert will extend for 33 m from a median opening under the proposed NBL embankment and to a discharge point on the east side of the embankment. To the west, the new culvert will replace a portion of an existing CSP culvert which extends under the existing highway embankment and adjacent railway embankment to the west. The new culvert will connect with the existing culvert under the railway embankment approximately at the property line and extend for 48 m eastward to discharge in the median gap. The maximum depth of cover over the culvert will be in the order of 5.0 m.

5.2 Construction Within the New Embankment

It is understood that the Geotechnical/Pavement Design Report recommends construction of a rock fill embankment for the proposed NBL to limit environmental impacts.

Settlement of the loose to compact foundation soils under the weight of the embankment is one of the principal issues related to the performance of a concrete box section within a new embankment. A settlement analysis, modelling the culvert within the proposed new embankment, indicated that settlement at the middle of the culvert will be in the order of 130 to 160 mm, as shown in Figure 2.

Two approaches to mitigating the effects of the predicted settlement have been considered:

- designing the culvert to accommodate the settlement
- preloading the site and allowing the settlement to occur prior to constructing the culvert.

These two options are presented below, though it is understood that the preloading option may be precluded by the need to limit environmental impacts.

5.2.1 Construction to Tolerate Settlement

If it is decided to construct the culvert under the new embankment without first preloading the underlying foundation soil, the following construction sequence is recommended:

1. Temporarily divert the creek from the alignment of the new construction.
2. Lower the groundwater to at least 1.0 m below the lowest required level of excavation.
3. Excavate to the required subgrade level to accommodate a minimum thickness of 300 mm of Granular A bedding (see discussion on bedding thickness below). All topsoil, peat, soft and otherwise deleterious material must be stripped from below the culvert to ensure founding on uniform, competent foundation soil. If this results in over excavation, backfill the over excavation with compacted Granular A.
4. Place required thickness of Granular A bedding.
5. Construct and backfill the box culvert. No specific standards exist, but reference should be made to OPSS 422 and OPSD 803.02 for guidance.
6. Backfill around the culvert using well graded Granular B placed to the minimum extent illustrated in Figure 1.
7. Return the creek flow to the culvert and construct the remainder of the embankment.

The dewatering system should be maintained in operation until the fill specified in Step 6 above has been placed to at least 1.0 m above creek level.

If this method of construction is used, the concrete box culvert must be designed to accommodate the estimated settlements which range up to

160 mm under the centreline of the new embankment and 97 mm at the end of the culvert, as illustrated in the attached Figure 2. It is understood that this can be accommodated by constructing the culvert in sections with articulated construction joints.

Figure 2 shows the estimated settlements along a half-length of the culvert for two situations:

1. the minimum 300 mm Granular A bedding
2. replacement of 2.0 m of soil below the culvert with Granular A.

The extra 1.7 m depth of Granular A reduces the total settlement at the centre of the culvert from 160 to 130 mm and reduces the differential settlement along the half-length of the culvert by 20 mm. Thus, it is theoretically possible to achieve a reduction in the total settlement of about 20% but against that must be set the added cost of deeper excavation and the difficulty and cost of further depressing the water table in relatively pervious soils.

5.2.2 Preloading Option

The following construction sequence is recommended only if it is feasible to divert the creek to the extent required. The objective is to preload the underlying foundation soil to facilitate completion of the majority of the settlement prior to placing the structure:

1. Temporarily divert the creek.
2. Strip topsoil and place a Granular A base as shown in Figure 3.
3. Construct the rockfill embankment to its full height to at least 10 m beyond the culvert, leaving a trapezoidal wedge shaped gap at the location of the box section, as illustrated in Figure 3.
4. Chink the surface of the rock fill in accordance with normal practice. The chinking should cover the top of the embankment plus the sides of the wedge. Chinking must effectively prevent future loss of road sub-base material into the interstices of the rock fill.
5. Backfill the wedge with well graded Granular B material, compacting to at least 95% SPMDD.

6. Overbuild to a height of 1.0 m above finished grade for the full width of the embankment and for a distance of 10 m to either side of the centreline of the box section
7. Allow all fill to remain in place for long enough to allow most of the settlement to occur (expected to require several weeks, see below).
8. After settlement has reached a satisfactory stage, lower the groundwater to a level at least 1.0 m below the lowest required level of excavation.
9. Excavate, as required, to accommodate construction of the box culvert and at least 300 mm of Granular A bedding. Stockpile clean granular material excavated from the wedge for use as backfill above the box culvert.
10. Construct the culvert ensuring a minimum of 300 mm of Granular A bedding below the base slab. No specific standards exist, but reference should be made to OPSS 422 and OPSD 803.021 for guidance.
11. The culvert bedding should be placed on uniform and competent foundation soils.
12. Backfill around and above the box section using the stockpiled granular material, compacting to 98% SPMDD.
13. Complete backfilling to subgrade level using the excavated material.

At Step 7, after the earth fill embankment has been completed to the over-built height, settlements should be monitored by a Geotechnical Engineer. Excavation and construction of the culvert may begin after the Engineer is satisfied that settlement under the new embankment has reached at least 90% of its estimated final value. Settlement may be monitored by means of suitable settlement points installed at the intersection of the centreline of the culvert and centreline of the embankment. Level surveying to ± 5 mm should be sufficiently accurate.

Some small rebound and recompression will occur during the excavation and backfilling to construct the box section, but this is expected to be too small to affect the structure.

The dewatering system should be maintained in operation until the fill

specified in Step 12 above has been placed to at least 1.0 m above creek level.

5.3 Construction Within the Existing Embankment

Based on the results of visual inspection on site, it appears that the existing embankment is constructed using earth fill.

The effect of constructing the 3.7 x 3.9 m box section will result in essentially no change in the vertical stress on the soil below the culvert. Therefore, there should be no new settlement of the underlying soils as a result of the construction of the new box culvert. Some elastic rebound and recompression may occur due to the process of excavation and backfilling. Since the unloading and reloading will occur within a confined area, the reconsolidation as backfill is placed will likely be minor and within tolerable limits for the structure.

It is important that the proposed box culvert be constructed on a dry, stable base. To achieve this condition, the existing creek must be temporarily diverted and the groundwater lowered at least 1.0 m below the base of excavation.

It is recommended that the structure be placed in an open excavation within the existing embankment following the procedure set out below :

1. Temporarily divert the creek.
2. Lower the groundwater level to at least 1.0 m below the base of the deepest required level of excavation.
3. Excavate across the embankment to accommodate the box section. It is recommended that the base of the excavation be at least 1.0 m wider than the box section on each side and be carried down to a level at least 300 mm below the underside of the base of the box section.
4. It is anticipated that the excavated material will be suitable for re-use as earth fill and it should be stockpiled on site if it meets all other requirements of the project specifications.
5. Place Granular A up to the underside of the box as a bedding layer.

6. Construct the box section. No specific standards exist, but reference should be made to OPSS 422 and OPSD 803.021 for guidance.
7. Backfill around and above the box culvert using Granular B, Type I compacted to 98% SPMDD. Backfill should be placed uniformly on both sides of the culvert.
8. Complete backfilling to subgrade level using the earth material previously stockpiled, if acceptable.

The dewatering system must be kept in operation until the backfill has reached to the top of the culvert.

5.4 Geotechnical Design Considerations

A cut-off wall should be constructed under the inlet end of the culvert under the new embankment. It is recommended that this wall penetrate 1.2 m below the underside of the box section.

If preloading of the foundation soil is not carried out, the box culvert should be cambered to account for the calculated settlement or, alternatively, the baffles required inside the culvert should be designed with sufficient height to compensate for the settlement.

The treatment of the stream bed at inlet and outlet is understood to be governed by fisheries concerns. It is assumed that the design of the stream bed at the inlet and outlet, as prepared by others, will take account of erosion prevention.

The new embankment will consist of rock fill with well graded Granular B around the culvert. Consequently, there are no frost heave concerns at this location. Within the existing embankment, it is understood that the top of the culvert will be below the frost line and no special frost treatment will be required.

5.5 Settlements Between Structures

Since the structures within the two highway embankments will be separated by a median gap, any differential movement which may occur between the two structures is not an issue from a geotechnical perspective.

There may however, be some relative movement between the new box culvert and the existing CSP under the railway embankment. The connection between the two should be flexible enough to accommodate some movement.

6. EARTH PRESSURE

The lateral earth pressures to be used in design of the box culvert and associated wing walls should be computed in accordance with Section 6-7 of the OHBDC .

The granular backfill should conform to Ontario Provincial Standard Specifications (OPSS) 1010 for Granular B, Type 1 and the walls of the box section may be designed based on the following unfactored earth pressure distribution:

$$P_h = K \gamma h$$

where;

K = earth pressure coefficient, use value from table on the following page

γ = unit weight of soil, = 21.2 kN/m³ for Granular B

h = depth below top of wall, m

Wall Type	Earth Pressure Coefficient (K)	
	OPSS Granular B $\phi' = 30^\circ$	
	Horizontal Ground Surface	Sloping Ground Surface (2H:1V)
Culvert Walls (And Restrained Walls)	0.50	-
Unrestrained Wing Walls	0.33	0.55

If the wing walls are cast integrally with the concrete box section, it is recommended that they be treated as restrained walls. Alternatively, it may be possible to construct the wing walls as Restrained Soil Systems (RSS) or other system which can tolerate some forward movement, in which case it is appropriate to treat the wing walls as unrestrained (active case).

Additional lateral pressure must be added to account for compaction induced forces. The additional pressure must be computed in accordance with Section 6-7.4.3 of the OHBDC.

7. TEMPORARY EXCAVATION

All excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of OHSA, the soils on this site are classed as Type 3 above the water table and Type 4 below the water table.

The sides of unsupported excavations and trenches above the water table must be sloped at 1H:1V for the full height of excavation, unless supported. If support is used, it must be in the form of shoring designed for the site and soil conditions by a professional engineer. If any excavation is carried out below the water table, unsupported slopes must be flattened to 3H:1V.

As indicated in Section 4.3, basal boiling and disturbance is expected for any excavation carried below the groundwater level. Accordingly, it is recommended that no excavation be carried below the prevailing groundwater level unless prior

unwatering is carried out to depress the groundwater level at least 1.0 m below the base of the deepest excavation.

Temporary support of the sides of the excavation may be required where the proposed new culvert joins the existing culvert under the railway embankment. A suitable system of temporary support would be cantilevered driven steel H-section soldier piles and wood lagging. It should be noted, however, that open, augered holes normally used to install soldier piles will penetrate below the water table and will be subject to base heave and collapse of the hole. The design of the temporary shoring must address the installation of soldier piles in sand below the groundwater table.

The earth pressures to be used in the design of the temporary shoring may be computed using the formula and values given in Section 6 of this report. The additional parameters which may be required for the calculation of passive toe resistance in front of the soldier piles are:

$$\begin{aligned} K_p &= \text{passive earth pressure coefficient, use a value of 3.0} \\ Y_{\text{sub}} &= \text{submerged unit weight of soil, } = 11.0 \text{ kN/m}^3 \end{aligned}$$

8. CONSTRUCTION CONCERNS

Construction within the existing embankment may encounter soil conditions differing from those described in this report and upon which the design recommendations are based. The contractor should be directed to inspect the subgrade of the excavation and draw to the attention of the Engineer any deleterious conditions which are encountered. It is very important to found the box culvert on uniform and competent foundation soil conditions to minimize the differential settlement of the structure. Temporary diversion of the creek and effective lowering of the groundwater level are important aspects of construction at this site. Failure to carry out either or both of these requirements could have serious detrimental impact on the construction and the future performance of the proposed box culvert.

The possibility of excavation below the groundwater level prevailing at the time of construction is also a concern. The contractor should be warned not to excavate

to such levels without effective, prior unwatering. An NSSP to this effect should be included in the contract if necessary.

9. CONSTRUCTION INSPECTION AND MONITORING

During construction, all foundation installation, excavation and embankment construction activities should be monitored by geotechnical personnel to confirm that the foundation recommendations and design are being correctly implemented and that the soil conditions encountered do not differ materially from the interpretation used in this report.

STATEMENT OF GENERAL CONDITIONS

1. STANDARD OF CARE

This study and Report have been prepared in accordance with generally accepted engineering or environmental consulting practices in this area. No other warranty, expressed or implied, is made.

2. COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report which is of a summary nature and is not intended to stand alone without reference to the instructions given to us by the Client, communications between us and the Client, and to any other reports, writings, proposals or documents prepared by us for the Client relative to the specific site described herein, all of which constitute the Report.

IN ORDER TO PROPERLY UNDERSTAND THE SUGGESTIONS, RECOMMENDATIONS AND OPINIONS EXPRESSED HEREIN, REFERENCE MUST BE MADE TO THE WHOLE OF THE REPORT. WE CANNOT BE RESPONSIBLE FOR USE BY ANY PARTY OF PORTIONS OF THE REPORT WITHOUT REFERENCE TO THE WHOLE REPORT.

3. BASIS OF REPORT

The Report has been prepared for the specific site, development, design objectives and purpose that were described to us by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the document are only valid to the extent that there has been no material alteration to or variation from any of the said descriptions provided to us unless we are specifically requested by the Client to review and revise the Report in light of such alteration or variation.

4. USE OF THE REPORT

The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client. NO OTHER PARTY MAY USE OR RELY UPON THE REPORT OR ANY PORTION THEREOF WITHOUT OUR WRITTEN CONSENT. WE WILL CONSENT TO ANY REASONABLE REQUEST BY THE CLIENT TO APPROVE THE USE OF THIS REPORT BY OTHER PARTIES AS "APPROVED USERS". The contents of the Report remain our copyright property and we authorize only the Client and Approved Users to make copies of the Report only in such quantities as are reasonably necessary for the use of the Report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make the Report, or any portion thereof, available to any party without our written permission. Any use which a third party makes of the Report, or any portion of the Report, are the sole responsibility of such third parties. We accept no responsibility for damages suffered by any third party resulting from unauthorized use of the Report.

5. INTERPRETATION OF THE REPORT

a) Nature and Exactness of Soil and Contaminant Description: Classification and identification of soils, rocks, geological units, contaminant materials and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgemental in nature and even comprehensive sampling and testing programs, implemented with the appropriate equipment by experienced personnel, may fail to locate some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and all persons making use of such documents or records should be aware of, and accept, this risk. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. Where special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.

(see over...)

INTERPRETATION OF THE REPORT *(continued)*

- b) Reliance on Provided Information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to us. We have relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, we cannot accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of misstatements, omissions, misrepresentations, or fraudulent acts of persons providing information.

6. RISK LIMITATION

Geotechnical engineering and environmental consulting projects often have the potential to encounter pollutants or hazardous substances and the potential to cause an accidental release of those substances. In consideration of the provision of the services by us, which are for the Client's benefit, the Client agrees to hold harmless and to indemnify and defend us and our directors, officers, servants, agents, employees, workmen and contractors (hereinafter referred to as the "Company") from and against any and all claims, losses, damages, demands, disputes, liability and legal investigative costs of defence, whether for personal injury including death, or any other loss whatsoever, regardless of any action or omission on the part of the Company, that result from an accidental release of pollutants or hazardous substances occurring as a result of carrying out this Project. This indemnification shall extend to all Claims brought or threatened against the Company under any federal or provincial statute as a result of conducting work on this Project. In addition to the above indemnification, the Client further agrees not to bring any claims against the Company in connection with any of the aforementioned causes.

7. SERVICES OF SUBCONSULTANTS AND CONTRACTORS

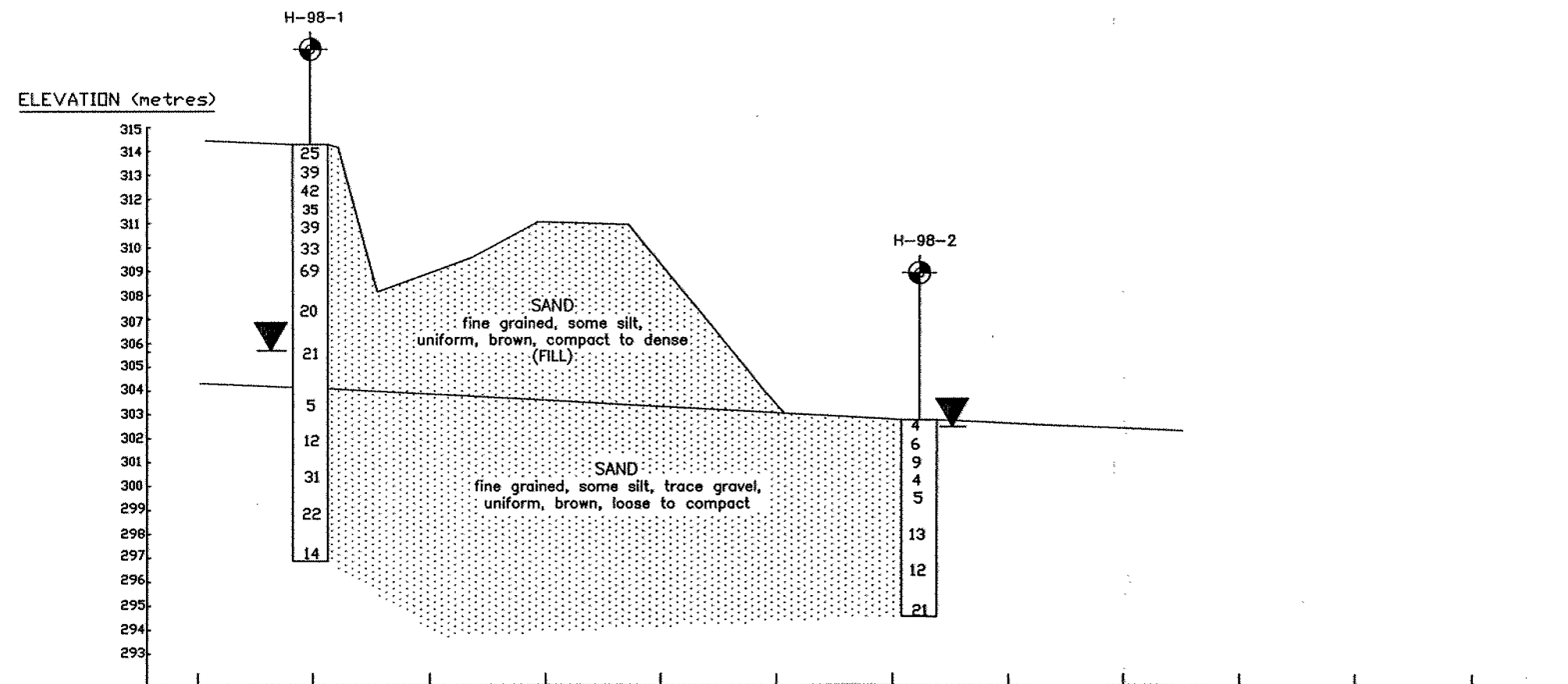
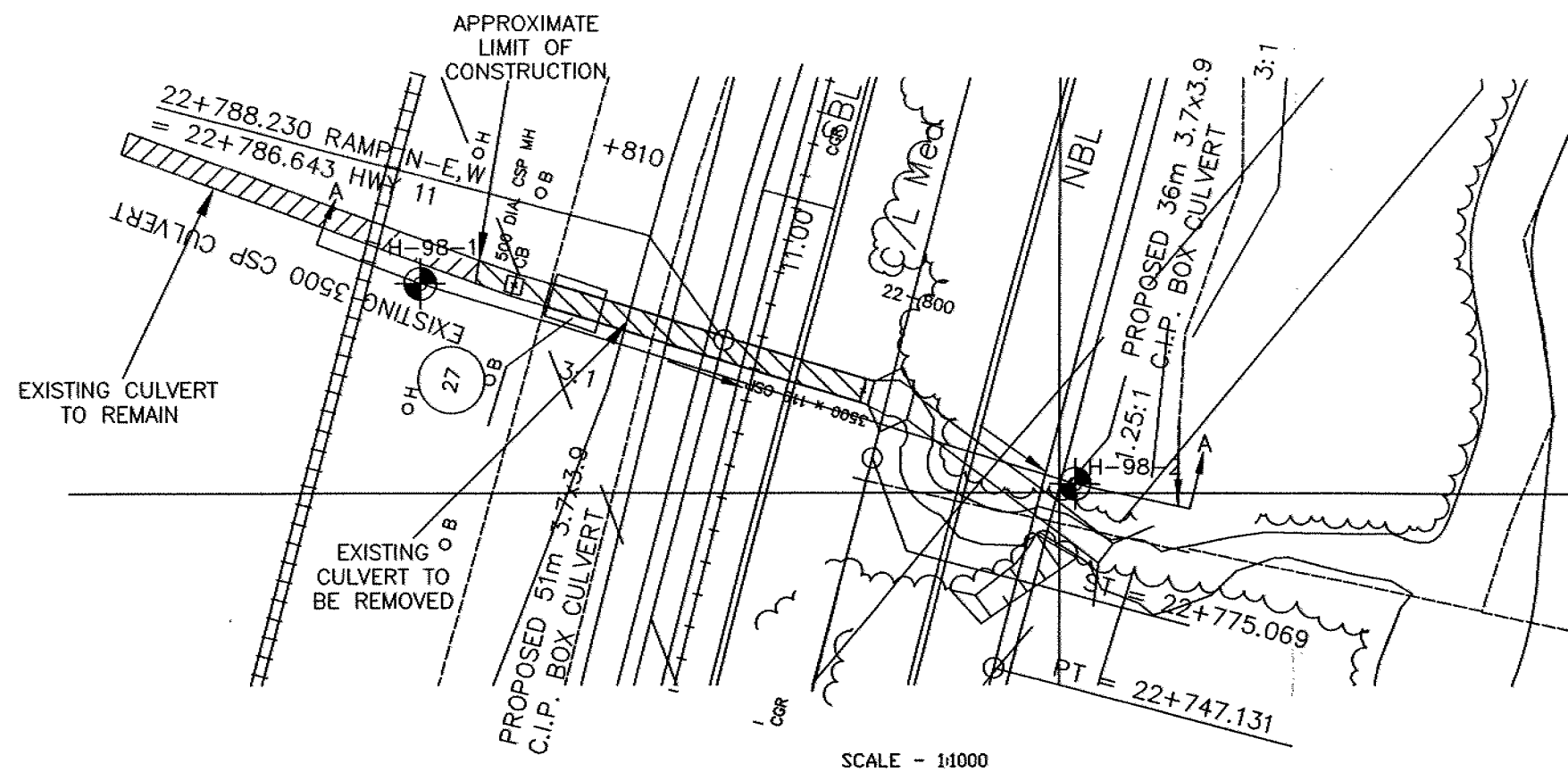
The conduct of engineering and environmental studies frequently requires hiring the services of individuals and companies with special expertise and/or services which we do not provide. We may arrange the hiring of these services as a convenience to our Clients. As these services are for the Clients' benefit, the Client agrees to hold the Company harmless and to indemnify and defend us from and against all claims arising through such hirings to the extent that the Client would incur had he hired those services directly. This includes responsibility for payment for services rendered and pursuit of damages for errors, omissions or negligence by those parties in carrying out their work. In particular, these conditions apply to the use of drilling, excavation and laboratory testing services.


8. CONTROL OF WORK AND JOBSITE SAFETY

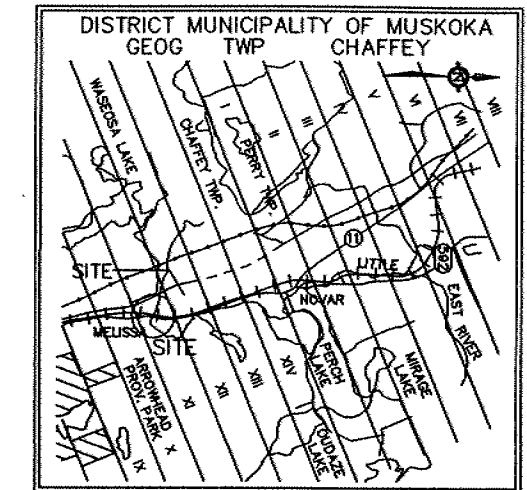
We are responsible only for the activities of our employees on the jobsite. The presence of our personnel on the site shall not be construed in any way to relieve the Client or any contractors on site from their responsibilities for site safety. The Client acknowledges that he, his representatives, contractors or others retain control of the site and that we never occupy a position of control of the site. The Client undertakes to inform us of all hazardous conditions, or other relevant conditions of which the Client is aware. The Client also recognizes that our activities may uncover previously unknown hazardous conditions or materials and that such a discovery may result in the necessity to undertake emergency procedures to protect our employees as well as the public at large and the environment in general. These procedures may well involve additional costs outside of any budgets previously agreed to. The Client agrees to pay us for any expenses incurred as the result of such discoveries and to compensate us through payment of additional fees and expenses for time spent by us to deal with the consequences of such discoveries. The Client also acknowledges that in some cases the discovery of hazardous conditions and materials will require that certain regulatory bodies be informed and the Client agrees that notification to such bodies by us will not be a cause of action or dispute.

9. INDEPENDENT JUDGEMENTS OF CLIENT

The information, interpretations and conclusions in the Report are based on our interpretation of conditions revealed through limited investigation conducted within a defined scope of services. We cannot accept responsibility for independent conclusions, interpretations, interpolations and/or decisions of the Client, or others who may come into possession of the Report, or any part thereof, which may be based on information contained in the Report. This restriction of liability includes decisions made to either purchase or sell land.



DIST 52 CONT No WP No 462-93-00	
HIGHWAY 11- FOUR LANING JESSOP'S CREEK STATION 22+788	SHEET
THURBER ENGINEERING LTD.	



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

LEGEND

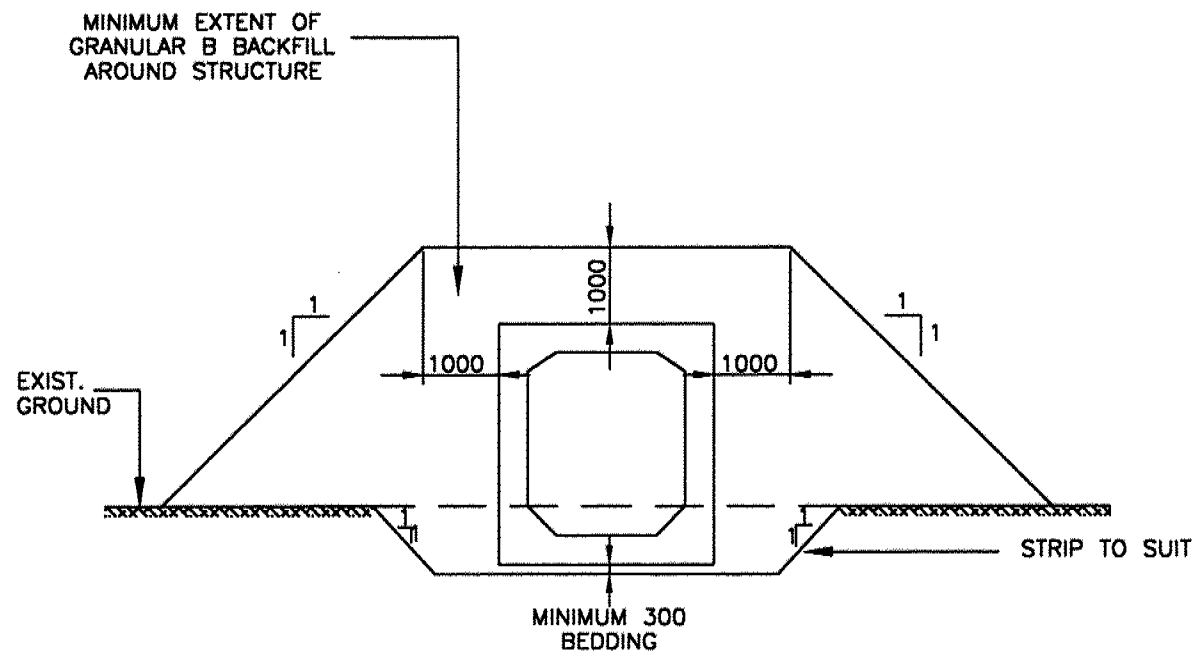

 LOCATION OF BOREHOLE
 H-98-1

WATER TABLE DEC 3/98

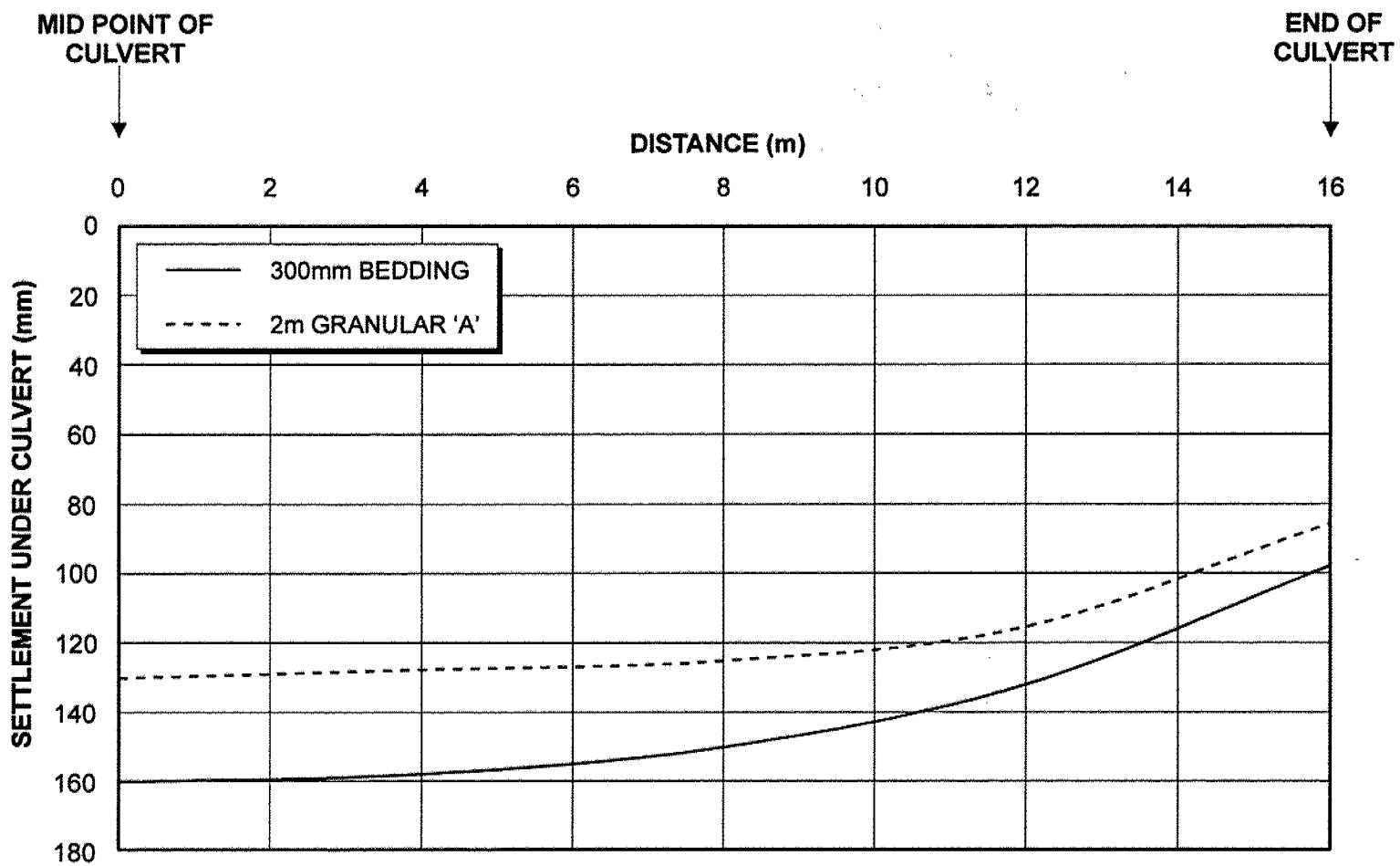
PLAN BASED ON PLAN E-625-11-
SHT 1 of 2 - SUPPLIED BY CLIENT

No	ELEV.	LOCATION	
		NORTHING	EASTING
H-98-1	314.69	5 030 831.2	325 204.7
H-98-2	303.28	5 030 802.4	325 301.9

19-1351-7h-01



BACKFILL AROUND CULVERT

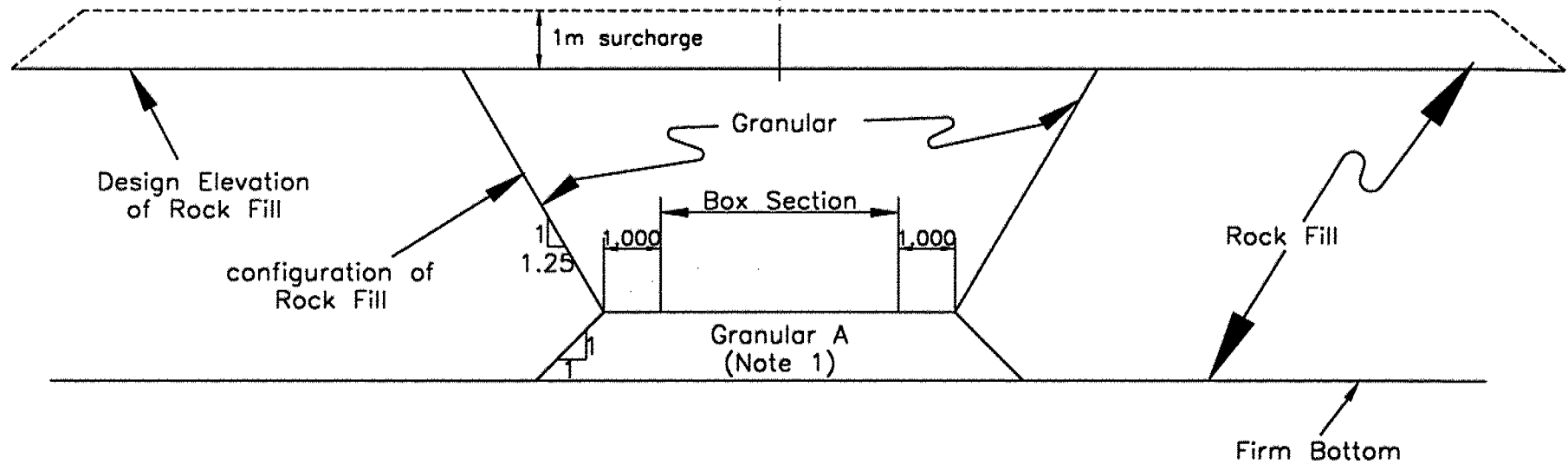


CALCULATED SETTLEMENT UNDER CENTRELINE
OF CULVERT

FIGURE 2

Note 1. Vertical limits of Granular A to be at least 300mm above and 300mm below the design underside of concrete.

C.L.
BOX SECTION



CONFIGURATION OF ROCK FILL WITH TRAPEZOIDAL
WEDGE GAP AND GRANULAR BACKFILL
(SHEMATIC ONLY)

APPENDIX A

BOREHOLE LOGS

- Symbols and Terms Used on Borehole Logs

- Unified Soil Classification

- Borehole Logs H-98-1 and H-98-2

SYMBOLS AND TERMS USED ON TEST HOLE LOGS

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	Less than 10	Less than 2
Soft	10 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 TEST HOLE LOGS

SYMBOLS FOR SAMPLE TYPE	Shelby Tube	A - Casing
	SPT	 Grab/Auger sample
	No Recovery	 Core

● MC - Moisture Content (% by Weight) as determined by sample]

▼ Water Level

C_{vane} Shear Strength Determination by Field Insitu Vane

C_{pen} Shear Strength Determination by Pocket Penetrometer

C_{lab} Shear Strength Determination using a Laboratory Vane Apparatus

C_u Undrained Shear Strength determined by Unconfined Compression Test

- (1) SPT Standard Penetration Test - refers to the number the blows from a 63.5kg hammer falling through 0.76m to advance a 60 degree truncated cone 0.3m.

UNIFIED SOILS CLASSIFICATION

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

RECORD OF BOREHOLE No H-98-1

2 OF 2

METRIC

W.P. 462-93-00 LOCATION N 5 030 831.2 E 325 204.7 ORIGINATED BY GA
 DIST 52 HWY 11 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY WM
 DATUM Geodetic DATE 98.11.12 - 98.11.12 CHECKED BY AEG

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT Y 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						
							20	40	60	80	100		
							WATER CONTENT (%)						
							w _p w w _L						
							10	20	30				
297.3			13	SS	22	299						○	0 90 10
						298							
			14	SS	14							○	
17.4	END OF BOREHOLE AT 17.4m. BOREHOLE OPEN TO 17.4m. Piezometer installation consist of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. On completion water level at 7.26m. WATER LEVEL READINGS: DATE DEPTH ELEVATION (m) (m) 13/11/98 8.52 306.17 16/11/98 8.81 305.88 17/11/98 8.78 305.91 18/11/98 8.74 305.95 03/12/98 9.06 305.63												

RECORD OF BOREHOLE No H-98-2

1 OF 1

METRIC

W.P. 462-93-00 LOCATION N 5 030 802.4 E 325 301.9 ORIGINATED BY GA
DIST 52 HWY 11 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY WM
DATUM Geodetic DATE 98.11.12 - 98.11.12 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		
303.3							SHEAR STRENGTH kPa						
303.0							○ UNCONFINED + FIELD VANE						
303.1							● QUICK TRIAXIAL × LAB VANE						
0.1	TOPSOIL		1	SS	4		20	40	60	80	100		
	SILTY SAND to SAND and SILT, fine grained, uniform, brown, loose to compact, wet trace organics at top		2	SS	6								
			3	SS	9								
			4	SS	4								
			5	SS	5								
			6	SS	13								
			7	SS	12								
			8	SS	21								
295.1													
8.2	END OF BOREHOLE AT 8.22m. BOREHOLE OPEN TO 6.10m. Piezometer installation consist of 19mm diameter Schedule 40 PVC pipe with a 1.52m slotted screen. On completion water level at 0.25m. WATER LEVEL READINGS: DATE DEPTH ELEVATION (m) (m) 12/11/98 0.19 303.09 13/11/98 0.18 303.10 16/11/98 0.21 303.07 17/11/98 0.22 303.06 18/11/98 0.20 303.08 03/12/98 0.22 303.06												

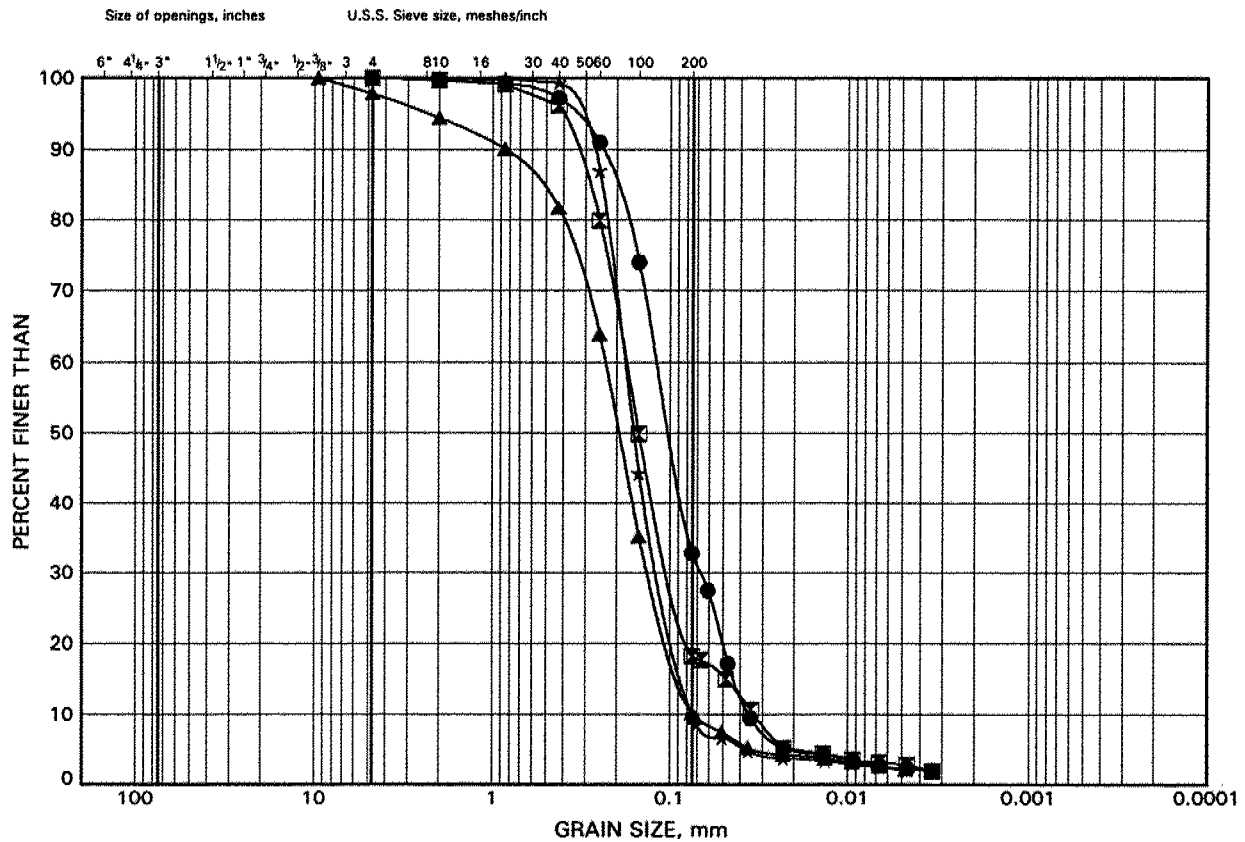
APPENDIX B

LABORATORY TEST RESULTS

- Figures B1 to B2 - Grain Size analyses
- Table B1 - pH and Sulphate Values

JESSOP'S CREEK GRAIN SIZE DISTRIBUTION

FIGURE B1

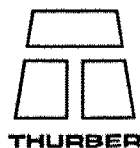


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
--------	----------	-----------	---------------

●	H-98-1	3.35	311.34
⊠	H-98-1	9.45	305.24
▲	H-98-1	10.97	303.72
★	H-98-1	15.54	299.15

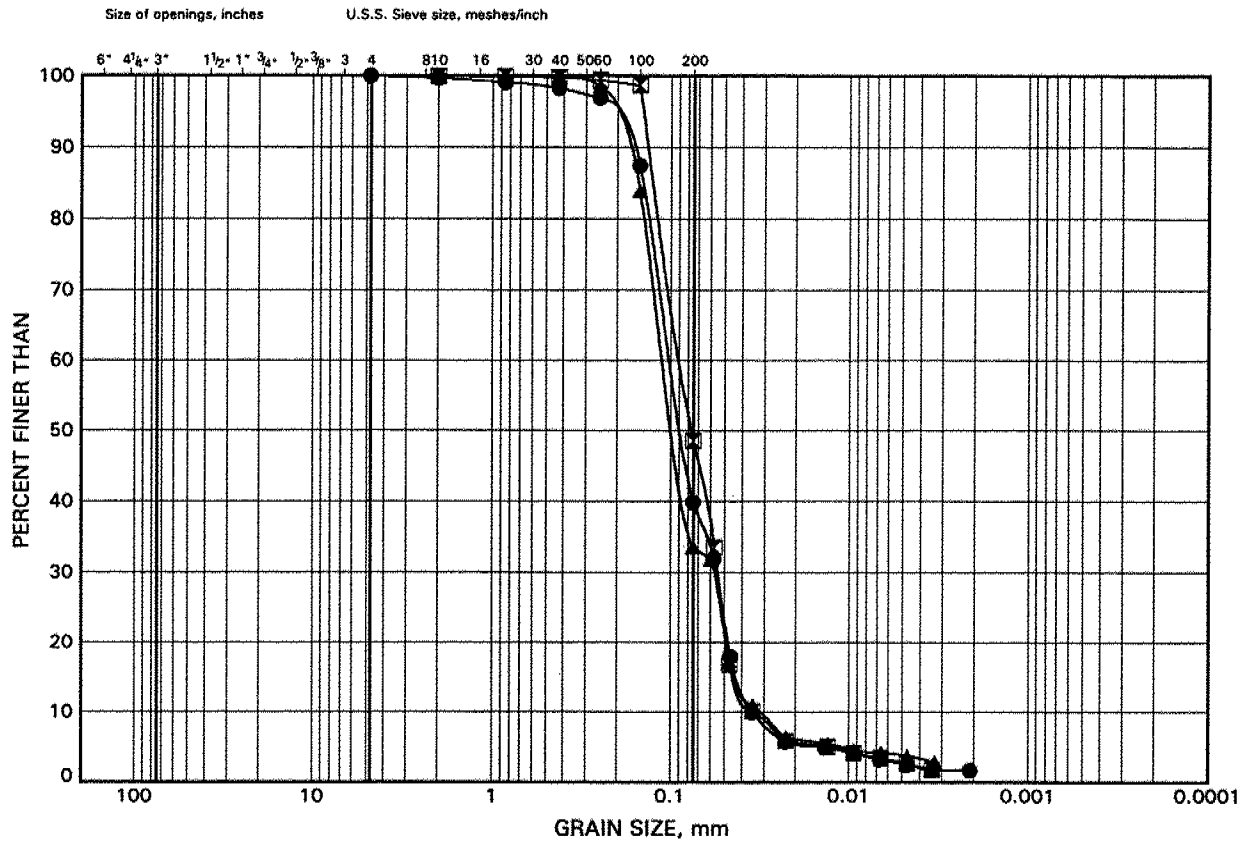
Date December 1998
Project 462-93-00



Prep'd WM
Chkd. AEG

JESSOP'S CREEK GRAIN SIZE DISTRIBUTION

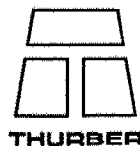
FIGURE B2



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

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	H-98-2	2.59	300.69
⊠	H-98-2	4.88	298.40
▲	H-98-2	7.92	295.36

Date December 1998
Project 462-93-00



Prep'd WM
Chkd. AEG

Table B1 - pH and SO₄

Borehole	Sample No.	Depth (m)	pH	SO₄ (ppm)
H-98-1	SS 11	12.5	7.4	103
H-98-2	SS 2	1.0	6.9	54

Re: Draft Report
Foundation Investigation and Design Report (dated November 27, 1998)
Jessop's Creek Culvert
WP 426-93-00
Hwy 11, District 52, Huntsville



P.K.

I'm sending a fax instead of the report for ease of delivery.

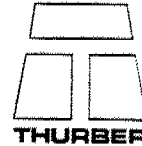
- 1) The Site # should be indicated on the title page and on the Foundation Drawing.
- 2) The Foundation Drawing does not meet our expectations. Normally north would be to the top of the page although I understand that there may be a reason for orienting this way. The section line should be shown on the plan. The existing culvert location on the plan should be clarified (bolded). ✓
- 3) Do the number of boreholes meet the requirements of the terms of reference? I do not have access to the Terms of Reference at the time of this review but it is possible that 3 boreholes with 1 borehole between existing boreholes may have been considered. Notwithstanding, the soil profile seems to be a thick deposit of sand so that an additional borehole may not be required for this project. ✓
- 4) Under Section 4.2 Groundwater, groundwater elevations should be provided (not just depths to groundwater). ✓
- 5) Are there concerns that the foundation soil is susceptible to disturbance under conditions of unbalanced hydrostatic head? If so, this concern should be noted in the factual (Foundation Investigation Report) portion of the report as this is the only information that is included in the Contract. Also, the Consultant should include explicit recommendations that suitable unwatering specifications be included in the Contract Documents. ✓
- 6) Recommendations for bearing resistance, frost protection, bedding, camber, joint details, clay seal and erosion protection at inlet, cut-off at inlet, filter blanket and erosion protection at outlet, diversion of existing water course if required for unwatering should be considered.
- 7) On page 10, the recommendation of Ka for culvert headwalls instead of Kp needs clarifications perhaps considering type of retaining wall including Retained Soil Systems (proprietary retaining walls on designated source list).
- 8) On page 11 the recommendation of Kp for shoring should be reviewed. Would Ka be acceptable? ✓

If there are any questions, please call.

Dave

THURBER ENGINEERING LTD.

Suite 101 170 Evans Avenue
ETOBICOKE, Ontario M8Z 5Y6
Phone (416) 503-3600
Fax (416) 503-3010



April 6, 1999

File: 19-1351-7h

McCormick Rankin Corporation
2655 North Sheridan Way
Mississauga, Ontario
L5K 2P8

Attention: Mr. Reno Radolli, P.Eng.

**Response to MTO Comments of March 8, 1999
Final Foundation Design Report for
Proposed Jessop's Creek Culvert
Highway 11 Four Laning
6.7 km North of Hwy 60 Northerly 13 km
W.P. 462-93-00
District 52, Huntsville**

Dear Mr. Radolli:

We have reviewed the MTO memorandum dated March 8, 1999, presenting the Foundation Section's comments on our final report for the above referenced project.

1. Foundation Design

The Foundation Section felt that we should have discussed what bearing resistance is available before recommending a scheme to overcome the lack of suitable bearing resistance. For the record and based on Borehole 98-2, we would have recommended an SLS bearing resistance of not more than 25 kPa, provided the structure could tolerate at least 50 mm settlement. Since we considered this to be a poor option, we concentrated on providing recommendations for a design that would accommodate the estimated settlements which would occur under the actual loading from the culvert or alternatively for pre-loading the site.

2. Dewatering

We understand that you have already dealt with the Dewatering NSSP.

Continued....

3. Earth Pressure

In our report, we recommended that Granular B backfill be placed around the concrete box culvert as a cushion between the concrete and the rock fill. Accordingly, we provided earth pressure parameters for Granular B. A more general treatment of the problem would allow the contractor the option of using Granular A. The parameters for both materials are shown overleaf:

Wall Type	Earth Pressure Coefficient (K)			
	OPSS Granular A $\phi' = 35^\circ$		OPSS Granular B $\phi' = 30^\circ$	
	Horizontal Ground Surface Behind Wall	Sloping Ground Surface (2H:1V)	Horizontal Ground Surface Behind Wall	Sloping Ground Surface (2H:1V)
Culvert Walls (Restrained Wall)	0.43	-	0.50	-
Wing Walls (Unrestrained Wall)	0.27	0.40	0.33	0.55

The unit weight to be used for granular A is 22.8 kN/m³

4. Culvert Inlet/Outlet Treatment

Clay seals and filters are commonly used in situations where a more pervious backfill placed within a low permeability parent material could create seepage pathways around the culvert. In this case, the embankment will be composed of rock fill and will be more pervious than the granular fill placed immediately around the culvert. In addition, the granular fill will be contained behind the culvert headwalls. We do not see an application for clay seals or filters on this site.

Continued....

McCormick Rankin Corporation

- 3 -

April 6, 1999

5. Culvert Invert

Inverts will be indicated on the final version of the drawings provided to you.

If you have any questions, do not hesitate to call our office.

Yours truly,
Thurber Engineering Ltd.
P.K. Chatterji, P.Eng.
Review Principal


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



AEG/aeg/c:C:\19\1351177H\REVLET02

APR 07 '99 10:20AM

THURBER ENGINEERING LTD.

Suite 101, 170 Evans Avenue
ETOBICOKE, Ontario M8Z 5Y6
Phone (416) 503-3600
Fax (416) 503-3010



February 4, 1999

File: 19-1351-7h

McCormick Rankin Corporation
2655 North Sheridan Way
Mississauga, Ontario
L5K 2P8

Attention: Mr. Reno Radolli, P.Eng.

**Modifications to Recommendations and
Response to MTO Comments
Preliminary Foundation Design Report for
Proposed Jessop's Creek Culvert
Highway 1 Four Laning
6.7 km North of Hwy 60 Northerly 13 km
W.P. 462-93-00
District 52, Huntsville**

Dear Mr. Radolli:

Enclosed is the final foundation report for the above referenced site. This report incorporates some modifications based on our discussions and on comments received from the Ministry.

Modifications

The main modification to the recommendations is the incorporation of a recommended construction sequence to allow construction of the culvert on the relatively loose foundation soils without the need to preload the site. This is in recognition of the fact that environmental concerns may preclude the sequence required for preloading.

Response to Ministry Comments

On December 24, 1998, we received comments on the above referenced report directly from the MTO Foundations Group. A copy of their comments is attached and our responses are set out below.

1. Site #. We requested a site number from your Mr. Gord Firth who advised that site numbers generally are not used on structures with spans less than 6 m.

Continued....

THURBER ENGINEERING

McCormick Rankin Corporation

- 2 -

February 4, 1999

2. Foundation Drawing. A profile line has been added. The orientation of the drawing is correct. The north arrow was incorrect and has been corrected. The location of the existing culvert has been clarified.
3. The investigation for the Jessop Creek culvert was not included in the original terms of reference. The number of boreholes was negotiated with the MTO Northern Region Planning and Design Section.
4. Groundwater elevations have been added to the text.
5. Foundation Soil Disturbance. If excavation is carried out below the water table without prior unwatering, disturbance will occur. A warning has been added to the factual portion of the report and the recommendations section has been augmented.
6. Various Recommendations. Recommendations have been included regarding frost, bedding and erosion protection (if needed). Due to the method of construction, we do not consider that camber will be an issue for this structure though some camber or gradient may be built in if the design requires that no water can pond in the structure. With respect to joints, this issue is covered in the report and has been discussed with the designers who will incorporate the necessary type of joint. Geotechnical bearing resistance values have not been included. It is considered that it would be misleading to give values for the existing soil conditions (which would be low) when the culvert is being designed to accommodate settlement as the embankment is constructed rather than relying on the resistance of the soil in its present state.
7. Earth Pressure. The earth pressure coefficients on page 10 of the Preliminary Report have been clarified.
8. Passive Pressure. The passive pressure coefficient on page 11 of the Preliminary Report referred to passive resistance in front of soldier piles. This has been clarified.

Continued....

APR 07 '99 10:20AM

THURBER ENGINEERING

McCormick Rankin Corporation

- 3 -

February 4, 1999

We trust these changes meet the requirements of the Ministry. If you have any questions, do not hesitate to call our office.

Yours truly,
Thurber Engineering Ltd.
P.K. Chatterji, P.Eng.,
Review Principal



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

AEG/aeg/cC:1191351/17HREVLET

MEMORANDUM



To: V. Minassian, P. Eng.
Senior Project Engineer
Planning and Design, Northern Region

March 8, 1999

From: Pavements and Foundations Section
Room 315, Central Bldg.

Tel: (416) 235-5267
Fax: (416) 235-5240

Re: Technical Review Package
WP 462-93-00
Hwy 11, From 6.7 km N of Hwy 60 Northerly 13.6 km
District 52, Huntsville

We have received the final Foundation Investigation and Design Reports for Jessop's Creek culvert and the snowmobile crossing submitted under your covering memorandum dated March 2, 1999. Review comments are contained in this memorandum.

Jessop's Creek Culvert

1. Foundation Design

The bearing resistance of the founding soil has not been included in the foundation report.

2. Dewatering

It is recommended that a Dewatering NSSP be included in the Contract that alerts the Contractor of the fact that the cohesionless soils present at the site are susceptible to conditions of unbalanced head and that the Contractor is responsible for rendering a stable excavation without inducing soil disturbance.

3. Earth Pressure

Granular backfill could be Granular "A" or Granular "B". Consequently, earth pressure design parameters should have been included in the report.

4. Culvert Inlet/Outlet Treatment

Recommendations for slope treatment (clay seal at inlet, filter material at outlet) should be included in the report.

5. Culvert Invert

The culvert invert elevation should be illustrated on the Borehole and Soil Stratigraphy Drawing.

Snowmobile Crossing

1. Comments 2, 3 and 5 above are also applicable to this structure.

We trust these comments are sufficient for your purposes. If you have any questions, please do not hesitate to contact this office.

T. Sangiuliano, P. Eng.
Foundation Engineer

for

D. Dundas, P. Eng.
Senior Foundation Engineer

cc. T. Kazmierowski

GEOCRES No. 31E-125DIST. 52 REGION W.P. No. 462-93-00CONT. No. W. O. No. STR. SITE No. HWY. No. 11LOCATION Hwy 11, 4 Laming from
 6.7 km N of Hwy 60 N'ly 13 kmNo of PAGES -

=====

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. REMARKS:

PRELIMINARY
FOUNDATION INVESTIGATION REPORT FOR
ROCK HAVEN/NORTH WABEOSA LAKE ROAD, MELISSA
HIGHWAY 41 FOUR LANING
6.7 km NORTH OF HWY 60 NORTHERLY 13 km
W.P. 462-93-00, SITE 42-322
DISTRICT 52, HUNTSVILLE

Report

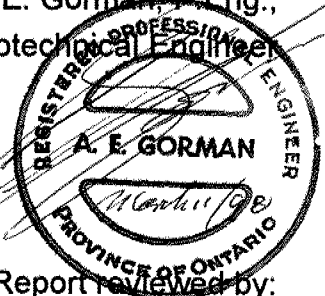
to

McCormick Rankin Corporation

Direction of fieldwork and engineering analysis by:

Thurber Engineering Ltd.
170 Evans Avenue, Suite 101
Etobicoke, Ontario
M8Z 5Y6
Phone: (416) 503 3600
Fax: (416) 503 3010

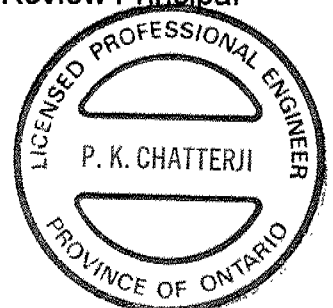
Alastair E. Gorman, P.Eng.,
Senior Geotechnical Engineer



Report reviewed by:

P.K. Chatterji, P.Eng.,

Review Principal



March 11, 1998

19-1351-7a

AEG/C:\19\1351\7a-inves.rpt

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1.	INTRODUCTION	1
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2.2	Physiography	2
2.3	Site Layout	2
3.	INVESTIGATION PROCEDURES	3
3.1	Field Investigation	3
4.	LABORATORY TESTING	3
5.	DESCRIPTION OF SUBSURFACE CONDITIONS	4
5.1	Subsurface Soil Conditions	4
5.2	Groundwater	6

DRAWINGS

19-1351-7a-01 Borehole Locations and Soil Strata

APPENDICES

Appendix A Borehole Logs
Appendix B Laboratory Test Results

**FOUNDATION INVESTIGATION REPORT FOR
ROCK HAVEN/NORTH WASEOSA LAKE ROAD, MELISSA
HIGHWAY 11 FOUR LANING
6.7 km NORTH OF HWY 60 NORTHERLY 13 km
W.P. 462-93-00, SITE 42-322
DISTRICT 52, HUNTSVILLE**

1. INTRODUCTION

This report presents the results of the foundation investigation carried out by Thurber Engineering Ltd. (Thurber) at the site of the proposed bridge and approach fills at the Rock Haven/North Waseosa Lake Road (Waseosa Road) intersection with realigned Highway 11 in the Town of Huntsville. The purpose of the investigation was to explore the subsurface soil and groundwater conditions at the site and based on that information provide the following:

1. A description of the subsurface soil and groundwater conditions at the bridge site
2. Recommendations for appropriate foundation systems.
3. The applicable values for the geotechnical parameters required for design.
4. Comments on any site specific construction procedures that might be required.

*Should not be
used in the
final report.
Criteria of the
report.*

Thurber carried out the investigation as a sub-consultant to McCormick Rankin Corporation (MRC) under Ministry of Transportation (MTO) Agreement 9750 - 7424 - 5262.

2. SITE DESCRIPTION

2.1 Site Location

The structure forms part of the four-laning of Highway 11 north of Huntsville and is located at the intersection of the proposed Waseosa road and a realigned portion of Highway 11. The site lies at Station 22+087, C/L Median Highway 11.

Locally, the site may be described as lying west of the existing Highway 11, between the highway and the CN right of way (ROW) ~~and east of the~~ *at the* junction of the existing Rock Haven Road and Highway 11. Access to the site was from existing Waseosa Road, just to the north.

2.2 Physiography

Based on The Physiography of Southern Ontario, 3rd Edition, by Chapman and Putnam, the region surrounding the site consists of bedrock ridges with shallow overburden. The bedrock is undifferentiated igneous and metamorphic rock of early Precambrian age and is generally hard and massively jointed.

The Highway 11 corridor, however, lies in a long, narrow sand plain within the region of shallow bedrock. The soils at the subject site are expected to be sand and silt, perhaps with some gravel, deposited as glacial outwash or in localized glaciolacustrine environments. There does not appear to be any commercial source of aggregate provided by this sand plain.

The nearby, meandering creek (Little East River) and several wetlands in the area suggest poor drainage and a high groundwater table.

2.3 Site Layout

At this site, Waseosa Road will be carried over the re-aligned Highway 11 on a two span bridge structure. The proposed vertical alignment of Highway 11 at this location requires a cut in the order of 7 to 8 m deep and as a result, little or no approach fill will be required to carry Waseosa Road over Highway 11. In view of the size of the structure and the anticipated soil conditions, the preferred foundation system is piles supporting an integral abutment design.

Based on the preferred foundation system, one sampled borehole is required at each foundation element and one at each approach fill. The boreholes at foundation elements are required to penetrate to a depth of 3 m below effective refusal to SPTs, defined as 100 blows for 0.3 m

penetration.

3. INVESTIGATION PROCEDURES

3.1 Field Investigation

Between January 7 and 15, and February 3 and 4, 1998, a track mounted auger rig was used on site for drilling, Standard Penetration Testing (SPT) and dynamic cone penetration testing. One hole was drilled for each abutment, one for the central pier and one at each approach fill, giving a total of five sampled boreholes. The approximate location of the boreholes are shown on Drawing 19-1351-7a-01.

The holes were advanced using hollow stem augers and SPTs were carried out at intervals of 0.75 m to a depth of 3.0 m and at intervals of 1.5 m thereafter. Fine uniform sand and silt were encountered below a relatively high water table which caused heaving of the soil into the hollow stem auger when the pilot bit was withdrawn in preparation for SPTs. In some cases, it was not possible to get the sampler down to the required depth and it became apparent that the soil immediately ahead of the auger was becoming disturbed before it could be sampled.

When it became apparent that no further, meaningful data could be obtained from the conventional drilling technique, two modifications were made to the procedures. In the first, dynamic cone penetration tests were added to provide supplemental information on the relative density of the soil ahead of the auger tip and beside the original sampled hole. In the second, the hollow stem augers were kept full of drilling mud to counterbalance the effect of the unbalanced head of groundwater.

On completion of drilling and sampling, a standpipe piezometer was installed in Borehole A-98-3 to monitor the groundwater level.

The result of the drilling and sampling are summarized on the borehole logs in Appendix A.

4. LABORATORY TESTING

Geotechnical laboratory testing consisted of natural moisture content determinations and visual classifications of all recovered samples. In addition, grain size analyses, Atterberg limits and pH and sulphate content testing were conducted on selected samples. The results of the laboratory testing are presented on the borehole logs in Appendix A, and in Figures B1 to B3 in Appendix B. The results of the pH and sulphate content are presented in Table 1.

5. DESCRIPTION OF SUBSURFACE CONDITIONS

5.1 Subsurface Soil Conditions

Detailed descriptions of the subsoil conditions encountered in the boreholes are presented on the borehole logs in appendix A. The stratigraphic profile inferred from the borehole information is shown on Drawing No. 19-1351-7a-01.

In general, the boreholes indicate topsoil over an upper sand layer, followed by a massive deposit of silt. The proposed highway cut will penetrate through the sand and will terminate in the silt layer. At greater depths, the silt was underlain by a lower sand layer.

Further, generalized description of these major soil units is provided in the following sections. The soils encountered all appeared to be lacustrine or fine outwash deposits and no evidence of boulders was found. However, the possibility of encountering boulders at random locations during construction must be recognized.

Topsoil

All boreholes encountered a 100 mm layer of topsoil at the surface. The topsoil is sandy, has a relatively high organic content (based on visual assessment) and is dark brown and moist.

Upper Sand

Below the topsoil layer, all boreholes encountered a layer of brown sand. Where fully penetrated, the sand layer was found to range in thickness from 2.2 to 6.0 m. This upper sand is classified as fine grained in Boreholes 1 through 4 and in Borehole 5 it is fine to medium grained. Some layering was apparent in the sand and occasional silt partings were noted. The SPT values recorded in the upper sand layer ranged from a low of 17 to a high of 81 blows for 0.3 m of penetration, but were generally in the range of 25 to 59. Based on the results obtained the sand has been classified as compact to very dense. Natural moisture contents measured in samples of the upper sand ranged from 5 to 21 % and it is described as moist.

Grain size distributions for selected samples are noted on the borehole logs and the plotted results are included in appendix B.

Borehole A-98-1 terminated in the upper sand layer.

Silt

Below the upper sand, Boreholes A-98-2 through A-98-5 encountered a massive deposit of silt. The thickness of the silt layer ranged from 6.2 to 12.2 m. The silt contains trace to some sand and occasional thin layers of sand about 5 to 10 mm thick. Trace clay was also observed throughout the silt, increasing to some clay in places. On the borehole logs, the silt is subdivided in layers of differing composition.

The silt is grey and the measured natural moisture contents ranged from 19 to 36 %. The silt lies below the groundwater table and is saturated. Based on the SPT values, supplemented by the results of dynamic cone penetration tests, the silt is described as compact to very dense. Several zones of lower SPT values were noted, which are likely caused by disturbance due to the unbalanced hydrostatic head of groundwater.

In Boreholes A-98-2 and A-98-3, the silt extended to depths of 17.8 and 17.6 m, respectively, where it was underlain by the lower sand. In Borehole A-98-4, a different sequence was encountered and the silt

extended to a depth of 8.6 m, below which it was underlain by the lower sand. In this hole, the lower sand layer was 7.5 m thick and was underlain again by the silt. Borehole A-98-4 terminated in this deeper silt layer.

Lower Sand

Below the silt in boreholes A-98-2 and A-98-3, and between depths of 8.6 and 16.4 m in Borehole A-98-4, the lower sand deposit was encountered. This sand is uniform, fine grained, ranging from trace silt to silty and is grey and saturated. Based on SPT values ranging from 24 to 77, and the supplemental dynamic cone test data, the sand in Boreholes A-98-2 and A-98-3 is classified as compact to very dense. In borehole A-98-4, low SPT values were obtained in the sand, but these are believed to be due the disturbance caused by unbalanced hydrostatic pressures. Measured natural moisture contents in the sand ranged from 20 to 31 % and the sand is saturated.

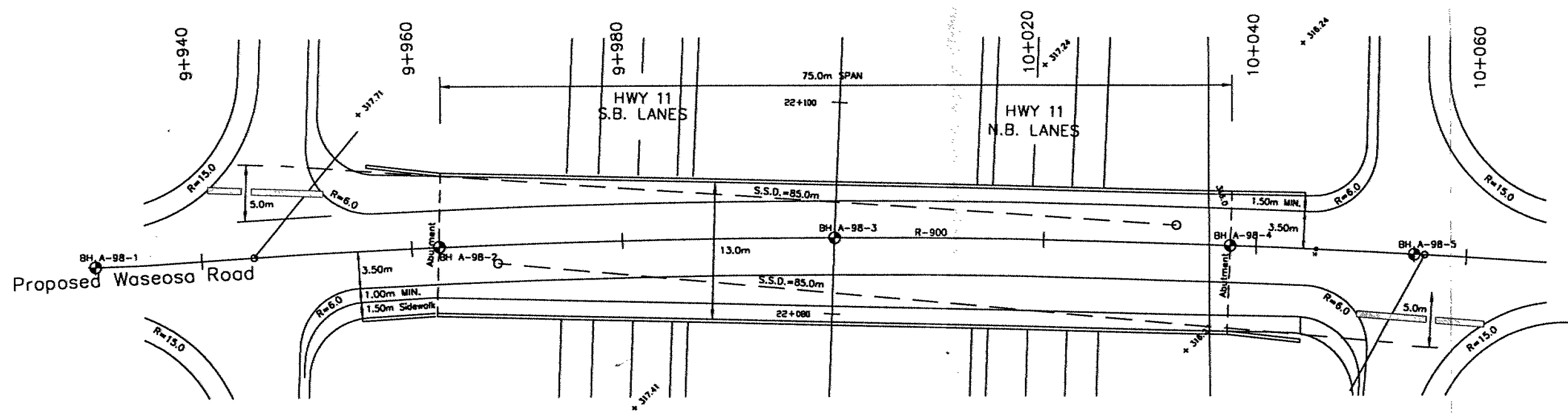
Boreholes A-98-2 and A-98-3 terminated in the lower sand deposit.

5.2 Groundwater

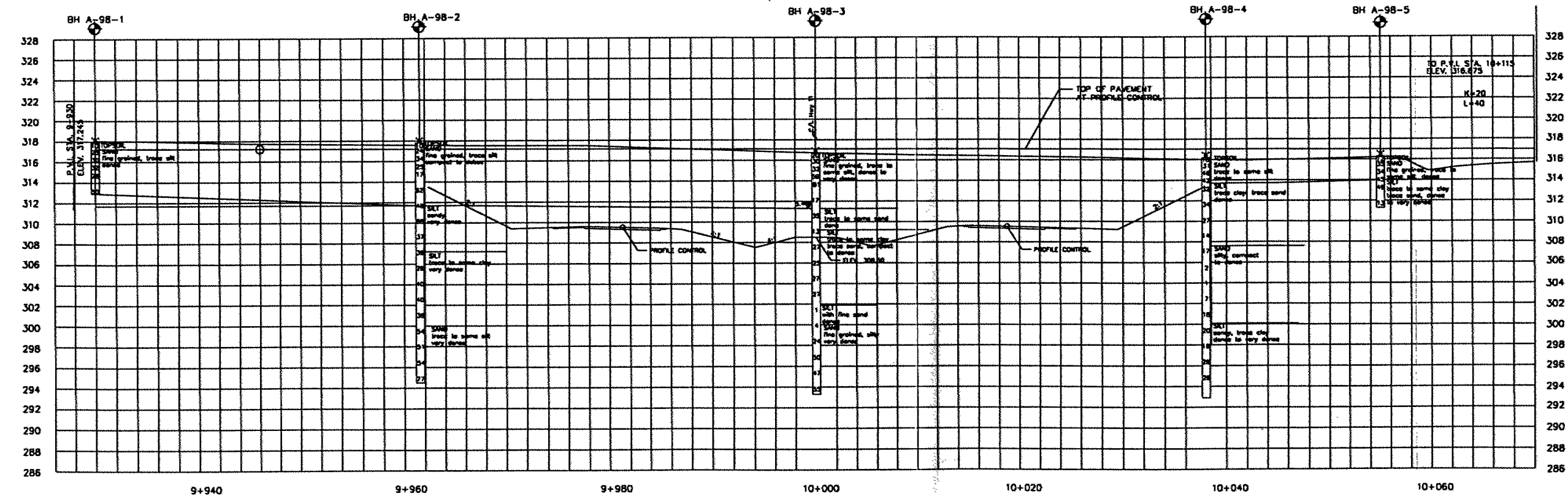
The following groundwater levels were recorded in the piezometer installed in Borehole A-98-3:

Date	Depth to Water (below existing ground surface)
Jan 12, 1998	4.6 m
Feb 4, 1998	6.7 m
Feb 23, 1998	7.0 m

Based on this data, the groundwater elevation lies at 309.9. This value is based on short term readings and may fluctuate throughout the year, in particular rising after the spring thaw.



PLAN

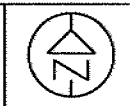


SOIL STRATA

PLAN AND PROFILE BASED ON
PLAN E-625-11-6 SUPPLIED
BY CLIENT

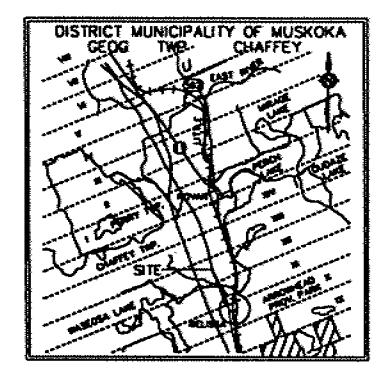
CONT No
W.P. 462-93-00

BOREHOLE LOCATIONS & SOIL STRATA



SHEET

THURBER ENGINEERING LTD.



KEY PLAN
SCALE 1:150000

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

PLATE No 625-11/168-0
DRAWING No 06250011168

LEGEND			
	Borehole		
	W. at time of investigation		
	Stream/0.5m (Std Plan Test)		
No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
A-98-1	317.927	30124.458	25084.915
A-98-2	317.987	30127.088	25126.734
A-98-3	318.635	30127.108	25186.285
A-98-4	318.188	30125.487	25201.481
A-98-5	318.372	30126.202	25219.883

APPENDIX A

BOREHOLE LOGS

- Symbols and Terms Used on Borehole Logs
- Unified Soil Classification
- Borehole Logs A-98-1 to A-98-5



BOREHOLE GRAPHIC SYMBOLS

SOILS



FILL

ORGANICS

CLAY

SILT

SAND

GRAVEL

COBBLES



SILTY CLAY

CLAYEY SILT

SILTY SAND

SAND & GRAVEL

CLAYEY SILT TILL

SILTY CLAY TILL

SANDY SILT TILL

ROCK



SHALE

LIMESTONE



SILTSTONE

GRANITE

OTHER



CEMENT GROUT

BENTONITE GROUT

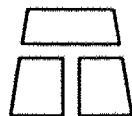


CONCRETE

WATER



BENTONITE SEAL



THURBER

UNIFIED SOILS CLASSIFICATION

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

RECORD OF BOREHOLE No A-98-1

1 OF 1

METRIC

W.P. 462-93-00 LOCATION WASEOSA RD. BRIDGE UNDERPASS / SITE 42-322 ORIGINATED BY EK
DIST 52 HWY 11 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY WM
DATUM GEODETIC DATE 98.01.08 - 98.01.08 CHECKED BY AG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT Y kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
317.9								20 40 60 80 100					
317.8	TOPSOIL, organic, dark brown		1	SS	10								
0.1	SAND trace silt, medium grained, dense, occasional silt seams to 10mm thick, brown, dry		2	SS	25		317						0 98 2
	<i>Compacted to dense</i>		3	SS	30		316						
			4	SS	32		315						
	more frequent seams below 3.5m		5	SS	26		314						
312.9			6	SS	50		313						0 97 3
5.0	END OF BOREHOLE AT 5.0m. BOREHOLE OPEN TO 4.5m. BOREHOLE DRY UPON COMPLETION. BOREHOLE BACKFILLED WITH DRILL CUTTINGS.												

RECORD OF BOREHOLE No A-98-2

2 OF 2

METRIC

W.P. 462-93-00 LOCATION WASEOSA RD. BRIDGE UNDERPASS / SITE 42-322 ORIGINATED BY EK
DIST 52 HWY 11 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY WM
DATUM GEODETIC DATE 98.01.13 - 98.01.13 CHECKED BY AG

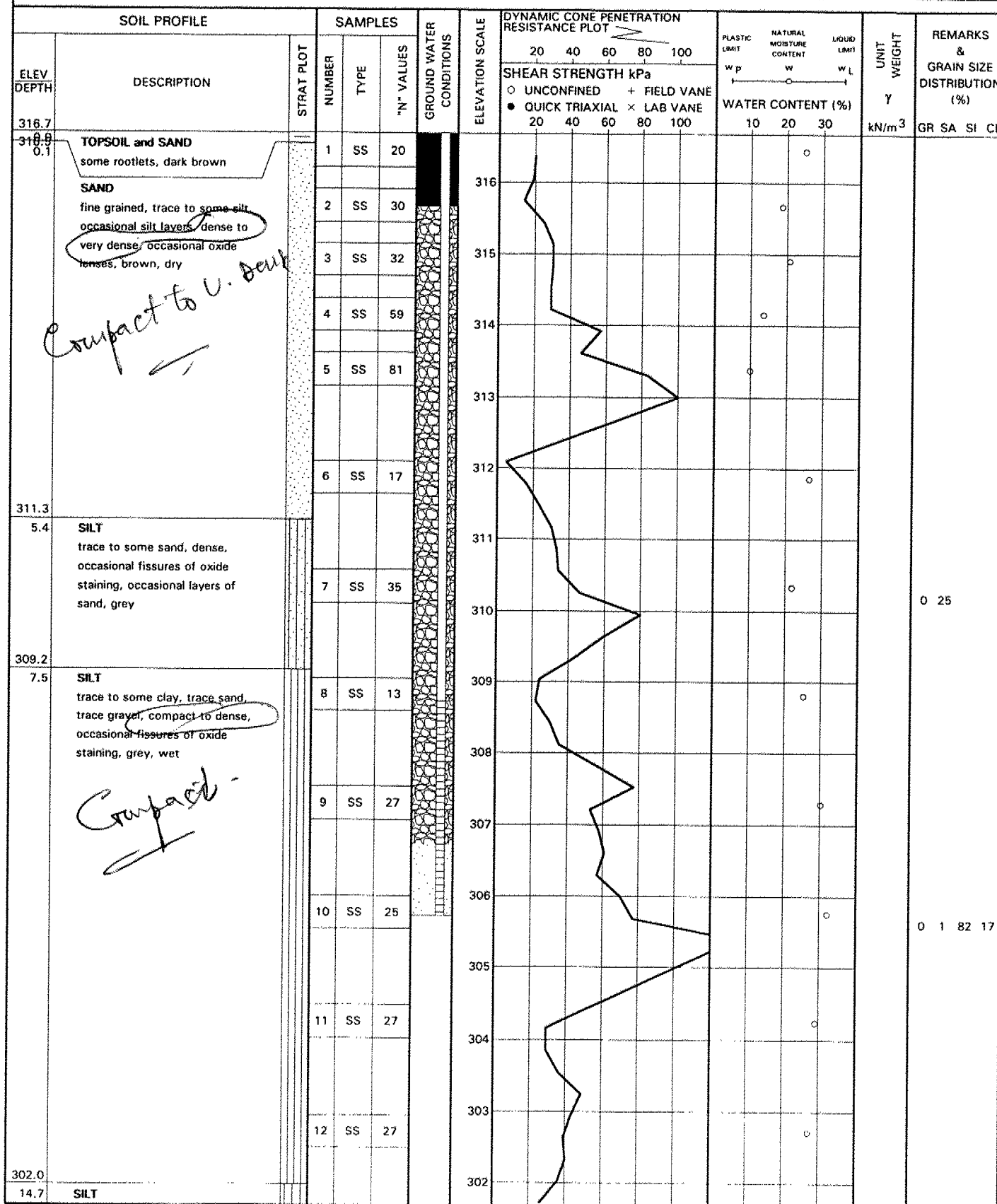
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa	WATER CONTENT (%)					
300.0	SAND trace to some silt, faint layering, very dense, grey, wet		13	SS	40									0 27
17.8			14	SS	38									
			15	SS	54									0 68
			16	SS	51									0 72
			17	SS	54									0 74
294.5			18	SS	77									
23.3	END OF BOREHOLE AT 23.32m. BOREHOLE BACKFILLED WITH DRILL CUTTINGS. Dynamic Cone Test Started at 15.8m.													

RECORD OF BOREHOLE No A-98-3

1 OF 2

METRIC

W.P. 462-93-00 LOCATION WASEOSA RD. BRIDGE UNDERPASS / SITE 42-322 ORIGINATED BY EK
DIST 52 HWY 11 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY WM
DATUM GEODETIC DATE 98.01.12 - 98.01.12 CHECKED BY AG



Continued Next Page

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

RECORD OF BOREHOLE No A-98-3

2 OF 2

METRIC

W.P. 462-93-00 LOCATION WASEOSA RD. BRIDGE UNDERPASS / SITE 42-322 ORIGINATED BY EK
DIST 52 HWY 11 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY WM
DATUM GEODETIC DATE 98.01.12 - 98.01.12 CHECKED BY AG

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			20 40 60 80 100	20 40 60 80 100					
299.1	with fine sand, wet		13	SS	1		301							
			14	SS	4		300							
17.6	SAND silty, fine grained, uniform/very dense, grey, wet		15	SS	24		299							
			16	SS	50		298							
			17	SS	47		297							
			18	SS	55		296							
293.3	END OF BOREHOLE AT 23.32m. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 3.0m slotted screen. Dynamic Cone Test Started at 18.3m.						295							
23.3							294							

Piezometer
water take?

RECORD OF BOREHOLE No A-98-4

1 OF 2

METRIC

W.P. 462-93-00 LOCATION WASEOSA RD. BRIDGE UNDERPASS / SITE 42-322 ORIGINATED BY EK
DIST 52 HWY 11 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY WM
DATUM GEODETIC DATE 98.01.15 - 98.01.15 CHECKED BY AG

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			20	40					
316.2	TOPSOIL, organic, dark brown SAND trace to some silt, occasional oxide and black staining, occasional layers of grey silt, dense, brown		1	SS	10									
316.0			2	SS	31									
0.1			3	SS	46									
313.7	SILT, trace clay, trace sand, occasional wet sand seams, occasional oxide staining, dense, brown		4	SS	42									
2.4			5	SS	32									
			6	SS	34									
			7	SS	27									
			8	SS	14									
307.6	SAND silty, very loose, grey, wet		9	SS	17									
8.6			10	SS	2									
			11	SS	1									
			12	SS	7									

Continued Next Page

3 x 3: Numbers refer to 20
Sensitivity 15-25 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No A-98-4

2 OF 2

METRIC

W.P. 462-93-00 LOCATION WASEOSA RD. BRIDGE UNDERPASS / SITE 42-322 ORIGINATED BY EK
DIST 52 HWY 11 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY WM
DATUM GEODETIC DATE 98.01.15 - 98.01.15 CHECKED BY AG

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								
300.1	SILT sandy, trace clay, layered, dense to very dense, grey, wet		13	SS	19		301						0 27 0 28 0 24
16.1							299						
			14	SS	20		298						
			15	SS	18		297						
			16	SS	28		296						
			17	SS	28		295						
292.8						294							
23.3	END OF BOREHOLE AT 23.32m. Dynamic Cone Test Started at 13.7m. Disturbed from 13.7 to 15.2m.						293						

RECORD OF BOREHOLE No A-98-5

1 OF 1

METRIC

W.P. 462-93-00 LOCATION WASEOSA RD. BRIDGE UNDERPASS / SITE 42-322 ORIGINATED BY EK
DIST 52 HWY 11 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY WM
DATUM GEODETIC DATE 98.01.08 - 98.01.08 CHECKED BY AG

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 Y kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40						60
316.3	TOPSOIL organic, dark brown SAND fine grained, trace to some silt, occasional oxide lenses, dense, brown, dry		1	SS	12										
316.0			2	SS	35										
0.1			3	SS	34										
313.9	SILT trace to some clay, trace sand, layered, occasional partings of grey sand, dense to very dense, grey		4	SS	45										
2.3			5	SS	49										
			6	SS	13										
311.3	END OF BOREHOLE AT 5.03m. BOREHOLE OPEN TO 4.9m. BOREHOLE BACKFILLED WITH DRILL CUTTINGS.														
5.0															

APPENDIX B

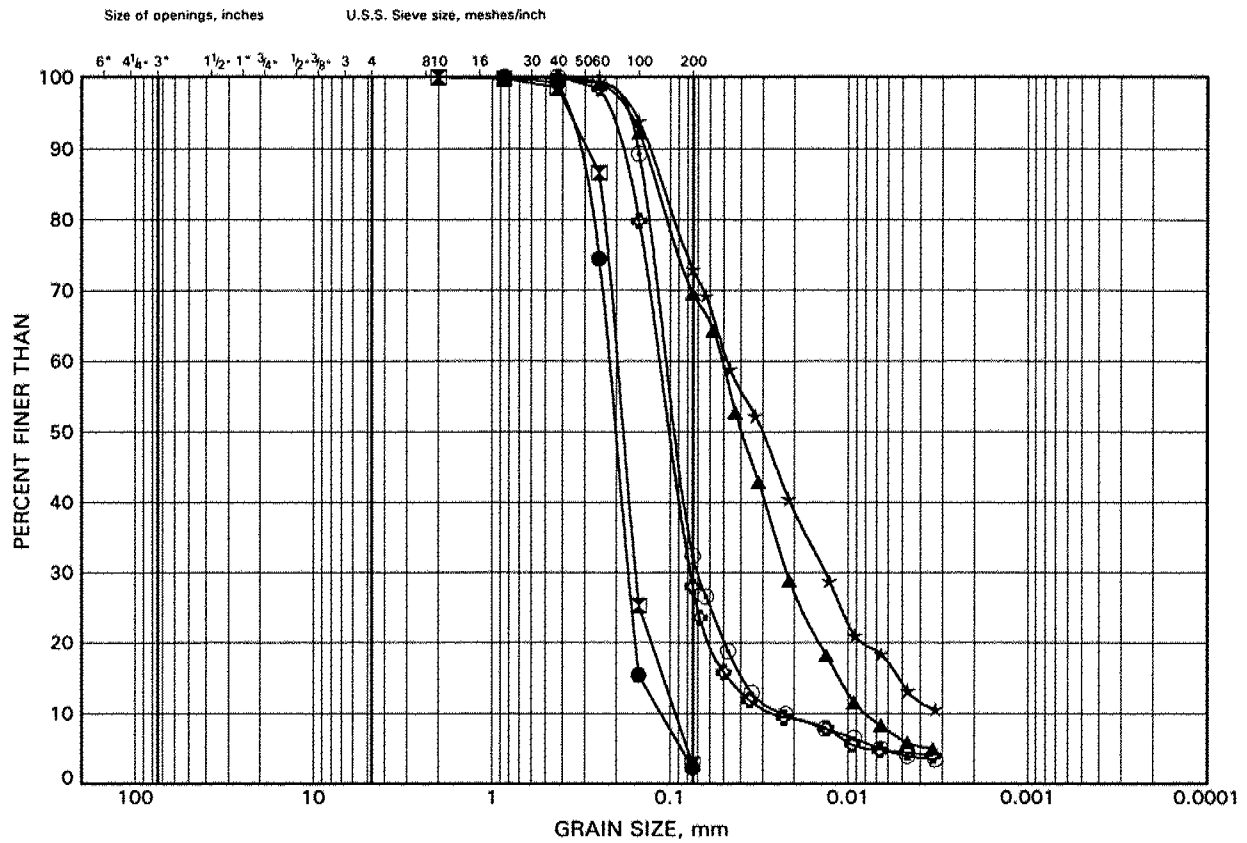
LABORATORY TEST RESULTS

- Figures B1 to B3 - Grain Size analyses

- Table 1 - pH and Sulphate

WASEOSA ROAD BRIDGE GRAIN SIZE DISTRIBUTION

FIGURE B1



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

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	A-98-1	0.99	316.94
⊠	A-98-1	4.80	313.13
▲	A-98-2	6.32	311.52
★	A-98-2	15.47	302.37
⊙	A-98-2	18.52	299.32
⊕	A-98-2	20.04	297.80

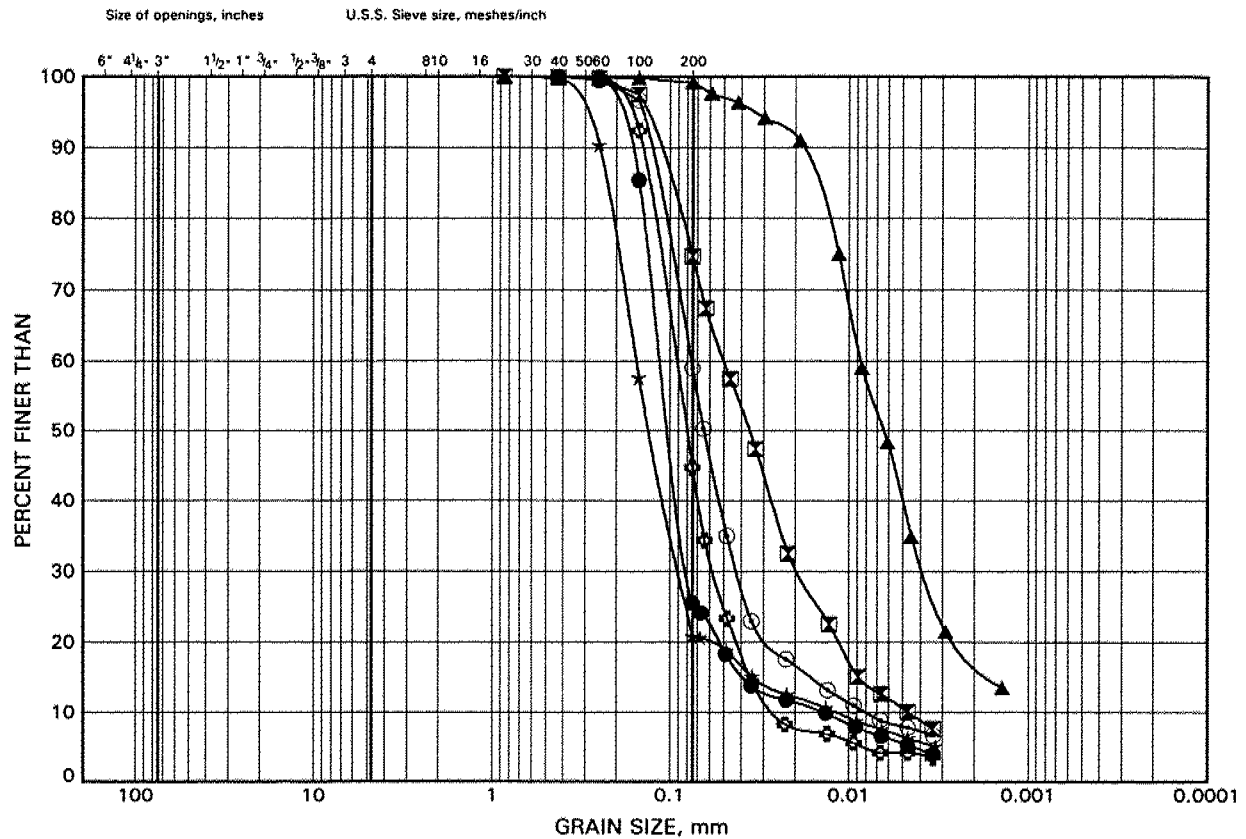
Date March 1998
Project 462-93-00



Prep'd WM
Chkd. AEG

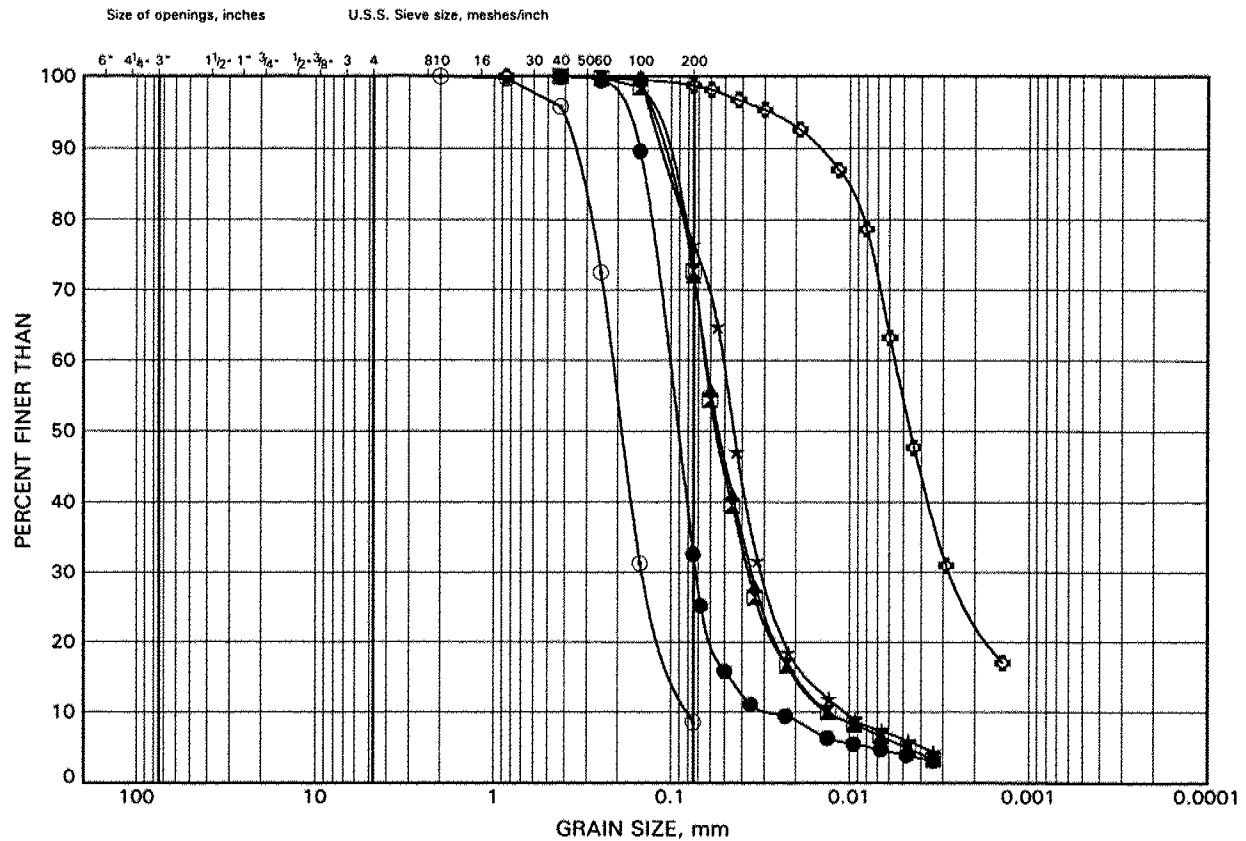
WASEOSA ROAD BRIDGE GRAIN SIZE DISTRIBUTION

FIGURE B2



WASEOSA ROAD BRIDGE GRAIN SIZE DISTRIBUTION

FIGURE B3

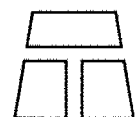


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	A-98-4	13.94	302.23
⊠	A-98-4	16.99	299.18
▲	A-98-4	20.04	296.13
★	A-98-4	21.56	294.61
⊙	A-98-5	1.75	314.52
⊛	A-98-5	3.28	312.99

Date March 1998

Project 462-93-00



THURBER

Prep'd WM

Chkd. AEG

Table 1

Results of pH and Sulphate Testing

Sample	pH	Sulphate (ppm)

PRELIMINARY
FOUNDATION RECOMMENDATION REPORT FOR
ROCK HAVEN/NORTH WASEOSA LAKE ROAD, MELISSA
HIGHWAY 11 FOUR LANE
6.7 km NORTH OF HWY 80 NORTHERLY 13 km
W.P. 462-93-00, SITE 42-322
DISTRICT 52, HUNTSVILLE

Report

to

McCormick Rankin Corporation

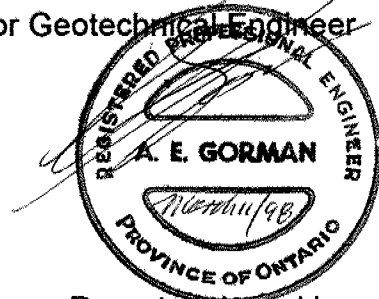
Thurber Engineering Ltd.
170 Evans Avenue, Suite 101
Etobicoke, Ontario
M8Z 5Y6

Phone: (416) 503 3600
Fax: (416) 503 3010

March 11, 1998
19-1351-7a
AEG/C:\19\1351\7a-desgn.rpt

Direction of fieldwork and engineering analysis by:

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



Report reviewed by:

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

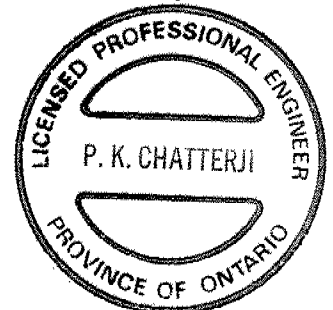


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2.2	Foundation Soil Conditions	2
2.3	Piled Foundations	2
2.3.1	Axial Capacity	3
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6.	FROST PROTECTION	7
7.	CONSTRUCTION CONCERNS	7
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Figure 1 - Fine to Medium Grained Sand Backfill for Integral Abutment Piles

**FOUNDATION DESIGN REPORT FOR
ROCK HAVEN/NORTH WASEOSA LAKE ROAD, MELISSA
HIGHWAY 11 FOUR LANING
6.7 km NORTH OF HWY 60 NORTHERLY 13 km
W.P. 462-93-00, SITE 42-322
DISTRICT 52, HUNTSVILLE**

1. INTRODUCTION

This report presents geotechnical design recommendations by Thurber Engineering Ltd. (Thurber) for the proposed bridge and approach fills at the Rock Haven/North Waseosa Lake Road (Waseosa Road) intersection with realigned Highway 11 in the Town of Huntsville. A Foundation Investigation Report has also been prepared for this site and has been issued under separate cover. This report must be read, and only used in conjunction with the corresponding Foundation Investigation Report prepared for this site by Thurber and dated March 11, 1998.

2. STRUCTURE FOUNDATIONS

2.1 Type of Structure

The proposed structure will be a two-span bridge carrying Waseosa Road over the re-aligned Highway 11. Geotechnical recommendations are required for the design of foundations at:

- the west abutment
- the central pier
- the east abutment

Each span of the proposed structure will be 38.5 m long and it is understood that an integral abutment design is preferred, if the foundation conditions are suitable.

Geotechnical recommendations are also required at the approach fills to the bridge.

2.2 Foundation Soil Conditions

The factual description of the foundation soils is presented in the Foundation Investigation Report. A discussion of the soil conditions is presented below.

At the abutments, the base of the abutment stem will be at approximate elevation 311.0. The foundation conditions encountered in the boreholes A-98-2 and A-98-4 consist of predominantly non-cohesive silt and fine sand. These soils are considered suitable for the design of an integral abutment bridge with each abutment supported each on a single row of H-piles driven to sufficient depth to achieve fixity well below the depth required to provide movement of the abutment.

For the proposed two span bridge, no movement is required at the central pier and the foundation can be considered to be effectively fixed. Either a spread footing or piled foundation would be acceptable in this location. Borehole A-98-3, however, shows that the potential founding soil is a saturated, silt lying below the existing water table. Even allowing for some long term lowering of the water table as a result of the construction of the highway cut, the water table would still be close below the underside of the footing and would rise close to, or above the footing base in wet weather. For these reasons, a spread footing is not considered suitable at this location and a piled foundation is recommended. ✓

2.3 Piled Foundations

2.3.1 Axial Capacity

The foundations of both the abutments and the central pier should be supported on HP 310X110 piles.

Axial resistance analysis has been carried out for the HP 310X110 pile using the soil parameters described in the Foundation Investigation Report and assuming both skin friction and end bearing.

For the abutments, analysis indicated that an HP 310X110 pile driven to a total depth of 28 m below the base of the abutment stem would have a factored ULS resistance of 1,500 kN and an SLS resistance of 1,000 kN. In the analysis, the upper 3.0 m was assumed not to contribute to the axial resistance since it was considered free to move with the abutment. This is expected to correspond to a pile tip elevation of approximately 283.0.

At the central pier, there is no requirement for lateral movement and the full depth of the pile below the pile cap was assumed to contribute to the axial resistance. For the pier, analysis showed that an HP 310X110 pile driven to 26 m below finished grade would have a factored ULS resistance of 1,500 kN and an SLS resistance of 1,000 kN. The pile should be driven to an approximate tip elevation of 282.5.

These geotechnical resistances should be checked against the structural capacity of the pile.

2.3.2 Lateral Resistance

The lateral resistance of the HP 310X110 pile was analyzed assuming flexure in the weak direction and the computed value is 27 kN.

2.3.3 Pile Installation

Pile driving should be carefully monitored and controlled employing the Hiley Dynamic Pile Driving Formula in accordance with MTO Standards SS 103-10 or SS 103-11 and assuming an ultimate resistance of 3,800 kN.

The piles supporting the two abutments should be installed in holes pre-augered to depths of 3.0 m which should then be backfilled with fine to medium grained, uniformly graded, loose sand. The grading requirements for the sand are shown in the attached Figure 1. In order to prevent collapse of the pre-augered hole and contamination of the sand backfill, a 600 mm diameter sleeve may be placed in the pre-augered hole.

The pile driving should be carried out using a hammer delivering in the

order of 50 kJ per blow.

During the driving process, pile which have already been driven should be monitored to determine if they are heaving due to the effects of driving adjacent piles. If this phenomenon occurs, the affected piles must be re-driven.

3. EARTH PRESSURE

The lateral earth pressures to be used in design should be computed in accordance with Section 6-7 of the OHBDC .

Backfill behind the abutment walls and wing walls within a 45 degree wedge extending upwards from the toe of the footing, should consist of free draining granular backfill meeting the minimum requirements of Ontario Provincial Standard Specifications (OPSS) for Granular A or B. These requirements are illustrated in OPSD 3501.00. The fill should be placed in maximum lift thickness of 150 mm and compacted in accordance with OPSS 501 using hand operated (walk behind) compaction equipment. A perforated subdrain should be installed behind the base of the walls as shown in OPSD 3501.00 to maintain the granular fill in a drained condition. The subdrain should be provided with a positive outlet to the highway drainage system.

For the above backfill and drainage conditions, the abutment walls and wing walls may be designed based on the following unfactored earth pressure distributions:

$$P_h = K \gamma h$$

where;

K = earth pressure coefficient, use value from table below.

γ = unit weight of soil, = 23 kN/m³ for Granular A
= 21 kN/m³ for Granular B

h = depth below top of wall, m

	Earth Pressure Coefficient (K)
Wall Type	

	OPSS Granular A $\phi' = 35^\circ$		OPSS Granular B $\phi' = 30^\circ$	
	Horizontal Ground Surface Behind Wall	Sloping Ground Surface (2H:1V)	Horizontal Ground Surface Behind Wall	Sloping Ground Surface (2H:1V)
Abutment Walls (Restrained Wall)	0.43	-	0.50	-
Wing Walls (Unrestrained Wall)	0.27	0.40	0.33	0.55

If an integral abutment design is used, the abutments will be cast integrally with the deck and therefore the abutment walls should be treated as restrained. If the wing walls will not be connected to the abutments and therefore will be able to accommodate some rotation they may be treated as unrestrained. The above also assumes a horizontal ground surface behind the abutment walls and a ground surface inclined at 2:1 behind the wing walls. If concrete approach slabs are not provided, an additional load equivalent to 600 mm of fill should be superimposed on the wall loadings to account for traffic surcharge loading.

Design lateral pressures at any depth should not be less than 16 kPa to account for compaction induced forces.

4. EMBANKMENT DESIGN

Based on the design profile, the re-aligned Highway 11 will be in a cut at the bridge site and the profile of the crossing road, Waseosa Road will be essentially at existing ground profile. Consequently, no embankments are envisaged at the west approach to the bridge and at the east approach, fills should not exceed 2.0 to 3.0 m.

For the shallow fills envisaged, the area of the base of the approach embankment should be stripped of topsoil and any other deleterious material before the embankment is constructed. Following stripping the base will consist of compact sand which will safely support the anticipated fills not exceeding 3.0 m in height. Nominal settlement of the existing sand is to be expected but this will occur as the load from the fill is applied and no long term settlements are expected.

The embankments may be constructed using inorganic earth materials placed and compacted to meet the requirements of OPSS 501. The side slopes of the embankments should not exceed 2H:1V.

If the embankments are constructed using locally derived material from the Highway 11 cut, it should be noted that this soil has an erodibility factor of 0.38, based on the Wischmeier Nomograph. It is recommended that erosion protection be applied to the slopes as soon as is practical.

5. GROUNDWATER CONTROL

The groundwater level established by piezometer readings is at Elevation 309.7, corresponding to 7.0 m below existing ground at the centre of the structure.

At the two abutments, neither short term nor long term dewatering will be required.

At the central pier, the underside of the pile cap is expected to be at Elevation 306.9. The excavation required to construct the pile cap will be carried out in saturated silt and will extend approximately 3.0 m below the stabilized groundwater level. Groundwater seepage into such an excavation would create unstable base conditions and caving of the side slopes, and it is recommended that positive groundwater control measures be implemented prior to excavation being carried out for construction of the pile cap. The groundwater control measures should be kept in place until construction of the pile cap has been completed and the excavation has been backfilled.

All excavations must be carried out in accordance with the Occupational health and Safety Act.

No permanent groundwater control measures are required for the proposed piled foundation.

For groundwater control requirements for cut slopes and pavements, reference should be made to the Geotechnical/Pavement Design Report.

6. FROST PROTECTION

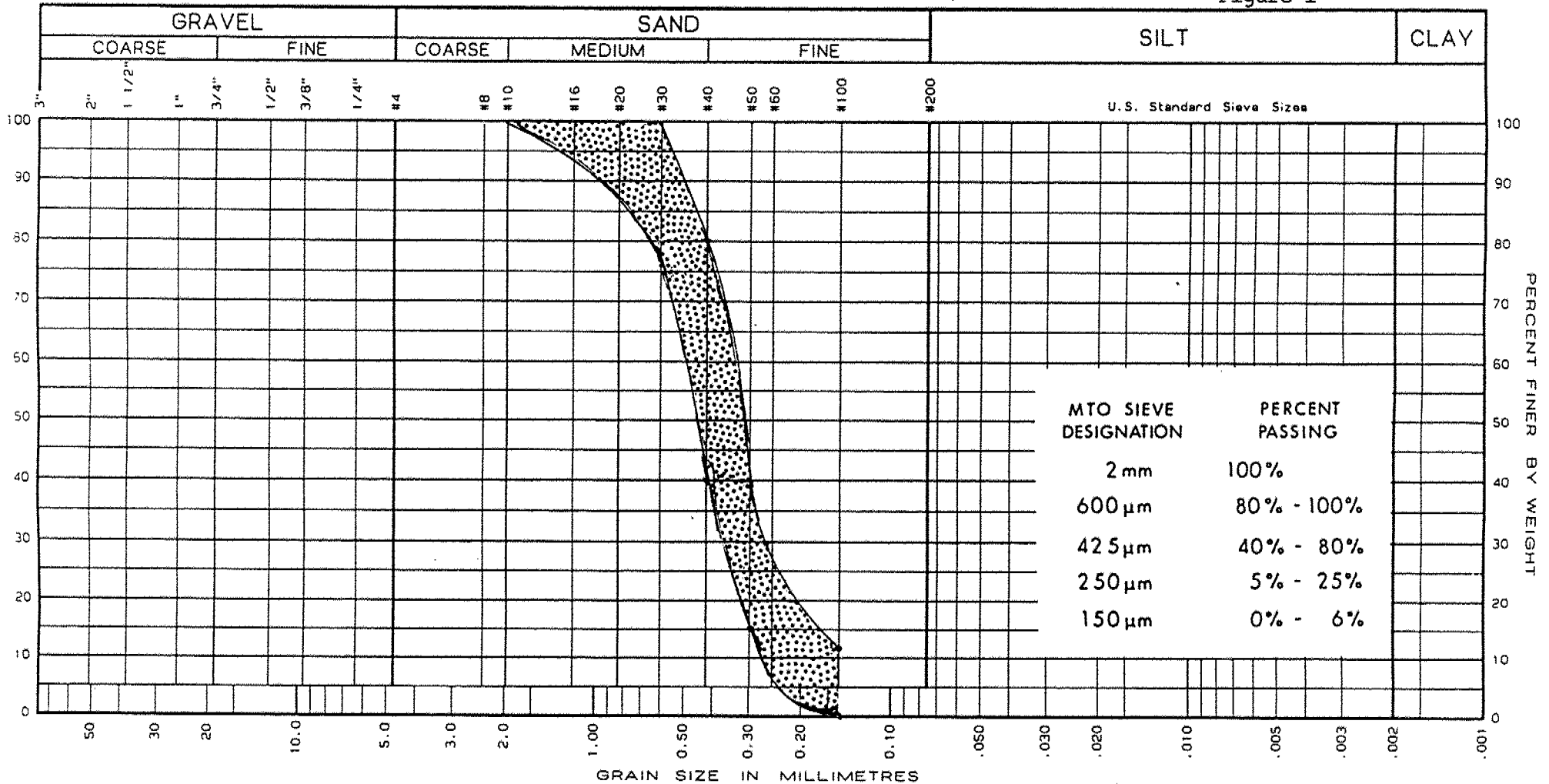
The design depth of frost penetration for this project is 1.6 m. All pile caps, grade beams and footings designed for this site must be provided with a minimum depth of soil cover of 1.6 m to protect against the penetration of frost below the foundation elements.

7. CONSTRUCTION CONCERNS

Construction of the pile cap for the central pier will be carried out from a subgrade level that will be created by excavation into a presently saturated silt stratum. It is important that the groundwater level be depressed prior to construction. In this regard, reference should be made to the sections of the geotechnical/Pavement Design Report dealing with cuts.

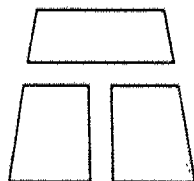
8. CONSTRUCTION INSPECTION AND MONITORING

During construction, all foundation installation, excavation and embankment construction activities should be monitored by geotechnical personnel to ensure that the foundation recommendations and design are being correctly implemented and that the soil conditions encountered do not differ materially from the interpretation used in this report.



CLASSIFICATION _____

Fine to medium grained sand
Backfill for integral abutment piles



CLAY
 SILT
 SAND
 GRAVEL

				x

CLIENT	MIO
PROJECT	19-1351-7a
LOCATION	Waseosa Road
SAMPLE	TECHNICIAN
TEST DATE	FILE N°.

Sangiuliano, Tony (MTO)

From: Sangiuliano, Tony (MTO)
Sent: Friday, March 31, 2000 10:21 AM
To: Smith, Dale P. Eng.(MTO)
Cc: Dundas, Dave (MTO); Kazmierowski, Tom (MTO)
Subject: RE: MTO W.P. 462-93-00, Supplementary Embankment Subsoil Investigation, Highway 11 NBL, Sta. 17+100 - 17+400

Dale:

We have reviewed JEGEL's proposal to conduct the foundation investigation at the existing embankment location and concur with your response that we need to investigate the area where the embankment widening is planned. It is important to emphasize that the field investigation be thoroughly conducted to determine the vertical and lateral extent of the organics AND any compressible and weak clayey soils. In addition, the Consultant should be instructed to provide a comprehensive Foundation report that presents the subsurface and groundwater conditions and contains foundation engineering options to address the stability and settlement considerations.

It is recommended that appropriate terms of reference be forwarded to the Consultant.

Tony

-----Original Message-----

From: Smith, Dale P. Eng.(MTO)
Sent: Thursday, March 30, 2000 03:31 PM
To: 'mmckay@jegel.com'
Cc: Minassian, Vatche (MTO); Sangiuliano, Tony (MTO)
Subject: MTO W.P. 462-93-00, Supplementary Embankment Subsoil Investigation, Highway 11 NBL, Sta. 17+100 - 17+400

Dear Mike

Thank you for preparing the proposed investigation plan to address the concerns regarding the proposed grade raise at the swamp crossing on W.P. 462-93-00 from Sta. 17+100 to 17+400 (your letter dated March 28, 2000).

The original investigation in my opinion did not adequately determine the conditions at this location. The swamp at this location is at approximate El. 329.0. The boreholes from the original investigation did not extend deeper than El. 330.31. Therefore, the depth of any muskeg adjacent to the existing embankment is not known, nor has the absence or presence of muskeg/soft soils beneath the existing embankment been determined.

The grade raise will necessitate embankment widening and conditions beneath the embankment are likely different from those beneath the widening. In my opinion, in order to adequately address the Ministry concerns, the proposed investigation should be augmented with boreholes at the proposed toe of the widened embankment advanced a sufficient depth below the general swamp elevation of 329.0 are necessary. This will determine the depth of muskeg to be excavated adjacent to the embankment - OPSD 203.02 or 203.03. The max. spacing of these boreholes should be 50 m. I understand your concern over the thawed conditions, however it should be possible to access the exploration location and advance boreholes using a hand operated power auger.

The Ministry does not have any construction records or relevant geotechnical information for this location (original construction was many years ago, geotechnical data obtained since original construction is limited to the pavement structure).

Borehole information from the adjacent project commences 800 m to the north of this site, and may not be of assistance. Please confirm your request for this information.

I will be in the office the rest of today and tomorrow as well (I was in the field yesterday), so I will be able to quickly respond to any concerns.

Regards,

Dale



memorandum

To: Gerry Chaput, P. Eng.
Senior Project Engineer
Planning and Design Section
Northern Region

From: Pavements and Foundation Section
Room 223, Central Building
Downsview, Ontario

Re: Interim Foundation Investigation and Design Reports
Rock Haven/North Waseosa Lake Road, Melissa
Highway 11, Four Laning, 6.7 km North of Hwy 60 Northerly 13 km
W.P. 462-93-00, Site 42-322
Highway 11, District 52, Huntsville

1998 03 27

At your request, we have conceptually reviewed the Interim Foundation Reports for the above projects, produced by Thurber Engineering Ltd to evaluate the performance of the Consultant. We have not reviewed the report in detail. The accuracy and completeness of the report remains the responsibility of the Consultant. Following are our comments:

REPORT

- We have received the report in two parts (factual and recommendation) under two covers. Normally the factual and the recommendation portions of the report are under the same cover for easier reference. However, recommendation portion starts on a new page, as the factual portion becomes part of the contract documents.
- Introduction, Page 1. In the factual portion of the report there should be no reference of recommendations or design.
- At the proposed abutment locations pile capacities are calculated based on pile tip elevation 283m. However the deepest borehole (A-98-4) only reached to elevation 292.8m. There is no soil information within 9.8m below elevation 292.8m. We are not sure how the pile capacities are calculated without having soil information between elevations 283m and

292.8m. What assumptions were made to calculate the pile capacity and what is the justification.

- **Embankment design, Page 6:** It is recommended that the side slope of the embankment should not exceed 2H:1V. Perhaps it was meant that the side slope of the embankment should not be steeper than 2H:1V. This should be reworded.
- **Frost Protection, Page 7:** As per MTO guideline frost depth for Huntsville and surrounding areas is 1.8m and not 1.6m. This should be corrected.

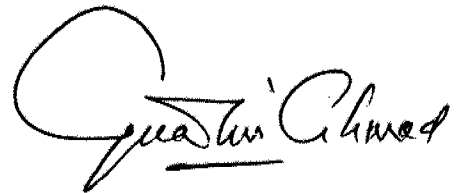
BOREHOLE LOGS

- The Location of the boreholes should be given by either Northing and Easting or Station and Offset
- The Relative Density in the Description Column of the Logs is not correct. This should be checked and corrected.
- In the grain size distribution, percentage of the fine grained soil is missing. If the samples were washed then the combined percentage of the fine grained soil (silt and clay) should be shown in brackets, e.g. 2 53 (45)
- When the boreholes continue on a new page then there should be a note in the Description column that tells that the borehole is Continued
- Information on water level is missing on the Logs. If the boreholes were dry upon completion it should also be noted on the Logs.
- Reference surface should be typed in the Description column e.g. Ground surface etc.

DRAWING

- The drawing is not to MTO standard
- The drawing is not legible. N-values, soil description and the other details cannot be read.
- No cross sections or profile showing the soil strata is provided.
- Location of the pier and the abutments should be clearly marked. Preferably with drafting tapes (shadings).

- Borehole co-ordinates plotted on the drawing do not make sense. For example, northing in that region is in five million range whereas the drawing shows northing in thirty thousand. There are no coordinates shown on the plan. The location on the drawing and the logs should be consistent. Either they should be northing-easting or station and offset. Since the stations are marked on the plan the location may be given by station or offset.
- Proposed grade is not marked on the plan or it may not be legible.



Ken Ahmad, P. Eng.
Foundation Engineer

For

T.C. Kim, P. Eng.
Senior Foundation Engineer

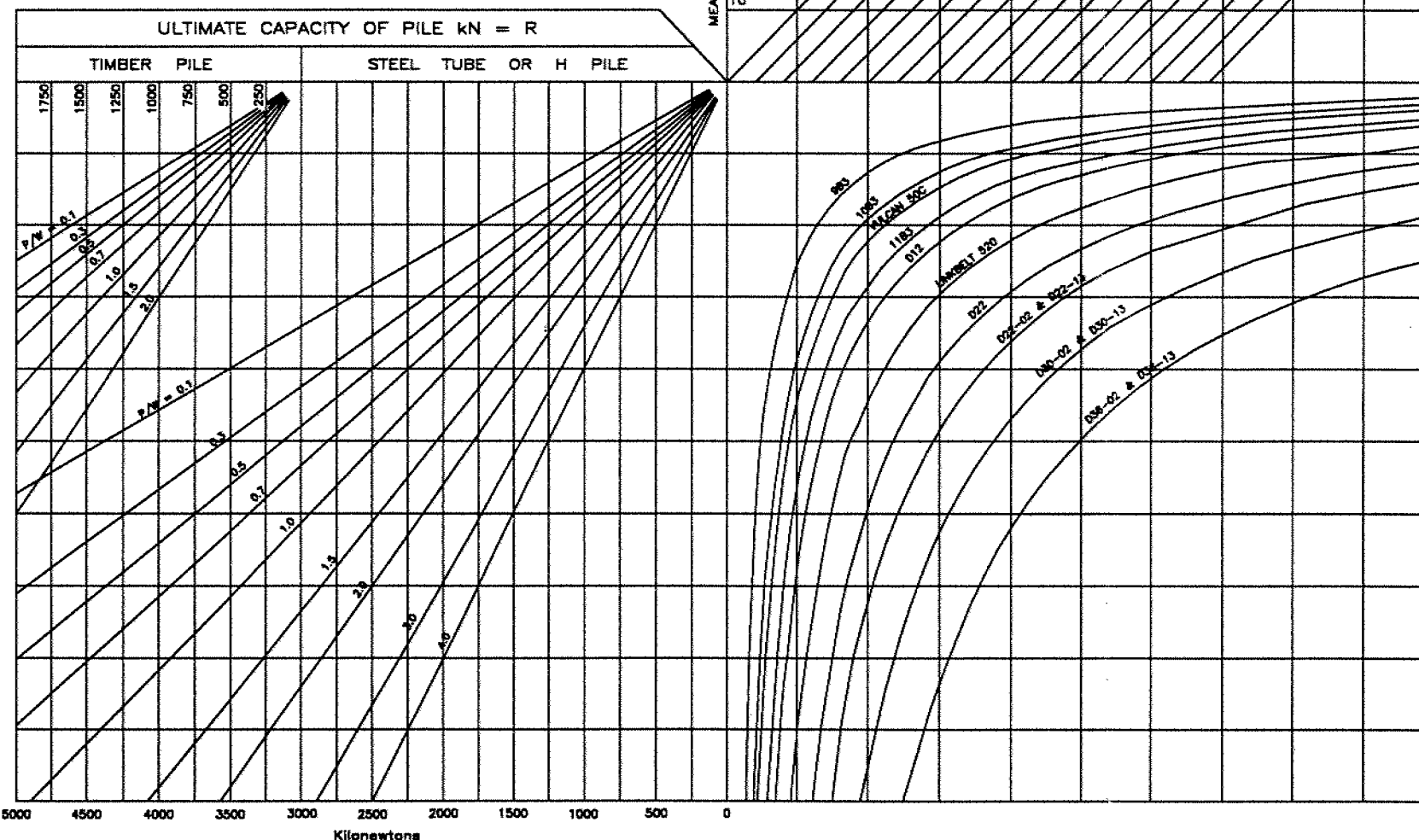
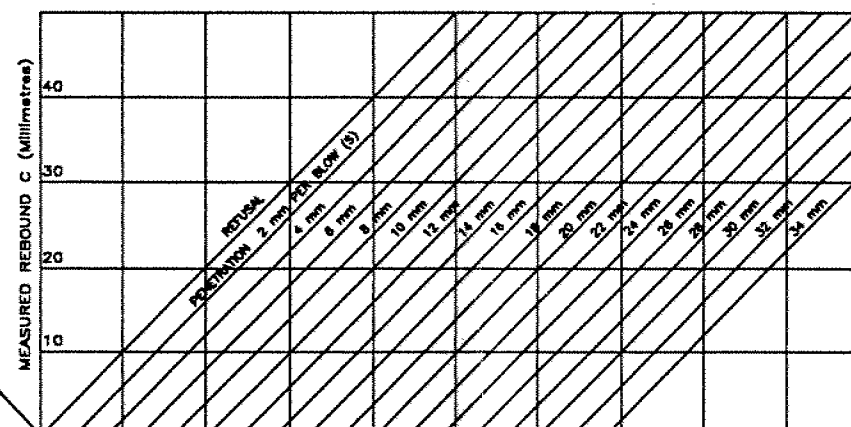
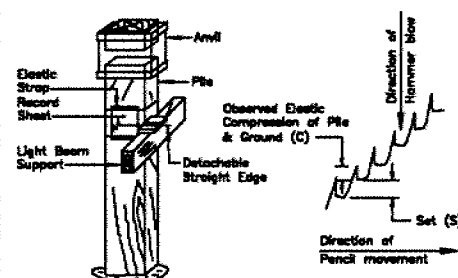
cc: T. Kazmierowski
P. Furst
J. McDougall

GEOCRES No. 31E-125DIST. 52 REGION W.P. No. 462-93-00CONT. No. W. O. No. STR. SITE No. HWY. No. 11LOCATION Hwy 11, from 6.7 Km W of
Hwy 60 Northerly 13.6 KmNo of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. REMARKS:

HAMMERS		
TYPE	MASS OF RAM W Kilograms	MAXIMUM ENERGY Joules/blow
983	728	12418
1083	1361	18948
90C	2288	20337
1183	2288	28005
D12	1250	30506
B225	1360	38300
LB520	2300	40875
B300	1700	48100
D22	2200	53828
B400	2288	62400
D22-02	2200	67000
D22-13	2200	67000
D30-02	3000	91000
D30-13	3000	91000
B500	3129	107100
D36-02	3600	115000
D36-13	3600	115000

NOTE:
Ram may also be referred to as Piston



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

CONT No
WP No

SHEET

PILE DRIVING-STEAM & DIESEL HAMMERS

METHOD OF APPLYING THE HILEY FORMULA

$$R = \frac{nWgh}{S + c/2} \quad (\text{Hiley Formula}) \quad g = 9.80665 \text{ m/s}^2$$

Where R = Ultimate pile capacity in kilonewtons
 S = Measured penetration of pile per hammer blow in millimetres
 C = Measured rebound of pile per hammer blow in millimetres
 Wgh = Energy of hammer blow in joules
 n = Efficiency of blow = $\frac{W + P_g}{W + P}$
 where $e = 0.32$ for steel (or $e = 0.55$. See Note 1 below.)
 $= 0.25$ for timber
 (These values of e have been determined by experiment)
 P = Mass of pile + anvil in kilograms
 W = Mass of ram (piston) in kilograms
 The P/W curves form the required reduction of total energy of the hammer blow according to the value of P/W

$L = R/Q$ kilonewtons
 Where L = Design capacity of pile
 Q = Factor of safety
 Use $Q = 3$ unless otherwise authorized by the Engineer

EXAMPLE 1:

Steel tube pile, O.D. = 323.80 mm, linear density = 49.73 kg/m, 20m long plus anvil of mass 600 kg, giving $P = 994.6 + 600 = 1594.6$ kg
 Deimag D12 hammer $W = 1250$ kg $P/W = \frac{1594.6}{1250} = 1.28$
 Observed measured rebound $C = 10$ mm
 Observed measured penetration $S = 5$ mm
 USING CHART: With $C = 10$ proceed horizontally to right to cut line $S = 5$ then vertically down to cut curve D12 then horizontally to left to cut $P/W = 1.28$ then vertically down to read ultimate capacity $R = 1512$ kN, $L = \frac{1512}{3} = 504$ kN

EXAMPLE 2:

HP 310x110, 50 m long plus anvil of mass 600 kg giving $P = 5500 + 600 = 6100$ kg.
 The hammer is Deimag D22-13
 $W = 2200$ kg, $n = \frac{W + P_g}{W + P} = \frac{2200 + (6100 \times 0.32 \times 0.32)}{2200 + 6100} = \frac{2824}{8300} = 0.34$
 Energy of hammer (Wgh) = 67000 J/blow
 Observed measured rebound $C = 10$ mm
 Observed measured penetration $S = 5$ mm

USING HILEY FORMULA:

$$\text{Ultimate capacity } R = \frac{nWgh}{S + c/2} \text{ kN} = \frac{0.34 \times 67000}{10} = 2278 \text{ kN}$$

$$\text{Design capacity } L = \frac{2278}{3} = 759 \text{ kN}$$

NOTE 1:

These charts are designed to cover only the piles driven with a pile cushion. Where Steel H-Piles are driven without a cushion, the ultimate pile capacity R should be calculated assuming a coefficient of Restitution $e = 0.55$.

NOTE 2:

These charts are designed to cover most cases which will be encountered on normal construction projects. Occasionally it will be found that R cannot be obtained from the charts, for instance when $C = 5$ mm and $S = 2$ mm using a Deimag D22 hammer. In such cases it will be necessary to calculate R using the original equation $R = \frac{nWgh}{S + c/2}$
 In cases where the energy of the hammer being used is slightly different from the hammer energy for which curves are drawn, the curves may still be used but the result should be reduced or increased according to the energy ratios. Example uses Linkbelt 520 curve (Energy 40 875 J) for Birminghamhammer 225 (Energy 39 300 J) but reduce result by multiplying by $\frac{39\ 300}{40\ 875}$

NOTE 3:

For Projects designed to the OHBDC, the ultimate capacity (R) is shown on the contract drawings and L and Q are not required.

STANDARD DRAWING
MARCH 1987
SS103-11
PILE DRIVING - STEAM & DIESEL HAMMERS

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

DATE	BY	DESCRIPTION
DESIGN	CHK	CODE
DRAWN	CHK	SITE
		LOAD
		DATE
		DWG

DIST 52
CONT No 99-221
WP No 465-93-01

SHEET
534

MRC **MCCORMICK RANKIN**
CORPORATION

1. ALTERNATIVE SHORING DESIGNS MAY BE CONSIDERED BY THE ENGINEER. ALTERNATIVE DESIGNS SHALL CONFORM TO THE FOLLOWING CRITERIA:

(a) RAILROAD LIVE LOAD
SURCHARGE 2.44 m OF FILL

2. THE CONTRACTOR SHALL SUBMIT SIX COPIES OF CALCULATIONS AND SHOP DRAWINGS, SHOWING FULL DETAILS OF THE PROPOSED SHORING SYSTEM TO THE CONTRACT ADMINISTRATOR FOR REVIEW PRIOR TO COMMENCING WORK. ALL CALCULATIONS AND SHOP DRAWINGS SHALL BE STAMPED AND SIGNED BY A QUALIFIED PROFESSIONAL ENGINEER LICENCED IN ONTARIO.

3. CLASS OF CONCRETE FOR SOLDIER PILE EMBEDMENT SHALL BE 20MPa.



DESCRIPTION	
DESIGN NSKD CHK KA	CODE OHBDC 1991/LOAD OHBDC DATE AUG 1999
DRAWN DGM CHK NSKD	SITE 42-10N STRUCT SCHEME DWG 3

DESIGN	NSKD	CHK	KA	CODE	OHBDC	1991	LOAD	OHBDC	DATE	AUG 1999
DRAWN	DGM	CHK	NSKD	SITE	42-10N	STRUCT	SCHEME	DWG	3	

**FINAL FOUNDATION DESIGN REPORT FOR
CNR OVERHEAD (NBL), NOVAR
HIGHWAY 11 FOUR LANING
6.7 km NORTH OF HWY 60 NORTHERLY 13 km
W.P. 462-93-00, SITE 42-10N
DISTRICT 52, HUNTSVILLE**

Report

to

McCormick Rankin Corporation

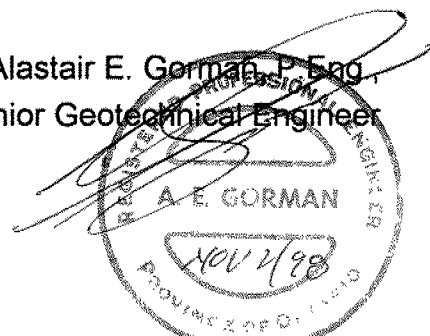
Direction of Fieldwork and Engineering Analysis by

Thurber Engineering Ltd.
170 Evans Avenue, Suite 101
Etobicoke, Ontario
M8Z 5Y6

Phone: (416) 503 3600
Fax: (416) 503 3010

October 30, 1998
19-1351-7e

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



Report Reviewed by :
P. K. Chatterji, P. Eng.,
Review Principal

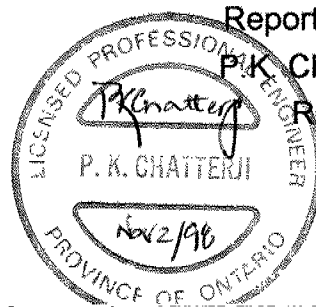


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DRAWINGS

19-1351-7E-01	Borehole Locations and Soil Strata
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Appendix B	Laboratory Test Results
Appendix C	Non-Standard Special Provision
Appendix D	Suggested Non-Standard Special Provision

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DRAWINGS

19-1351-7E-01	Borehole Locations and Soil Strata
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APPENDICES

Appendix A	Borehole Logs
Appendix B	Laboratory Test Results
Appendix C	Non-Standard Special Provision
Appendix D	Suggested Non-Standard Special Provision

**FINAL FOUNDATION DESIGN REPORT FOR
CNR OVERHEAD (NBL), NOVAR
HIGHWAY 11 FOUR LANING
6.7 km NORTH OF HWY 60 NORTHERLY 13 km
W.P. 462-93-00, SITE 42-10N
DISTRICT 52, HUNTSVILLE**

1. INTRODUCTION

This report presents the results of the foundation investigation and analysis carried out by Thurber Engineering Ltd. (Thurber) at the site of the CNR Overhead and approach fills on Highway 11 in Novar, Ontario. The purpose of the investigation was to explore the subsurface and groundwater conditions at the site and based on the data obtained provide borehole logs, soil profile and a written description of the subsurface conditions. The purpose of the analysis of the data obtained during the investigation was to produce geotechnical recommendations for the design and construction of the structure foundations and associated earth works.

Thurber carried out the investigation as a sub-consultant to McCormick Rankin Corporation (MRC) under Ministry of Transportation Ontario (MTO) Agreement 9750-7424-5262.

2. SITE DESCRIPTION

2.1 Site Location

The site forms part of the four-laning of Highway 11 north of Huntsville and is located at the crossing of the new Northbound Lanes of Highway 11 over the CNR tracks at Novar, Ontario. The site lies at Station 11+017, Centreline Northbound Lanes.

The site lies approximately 18 m east of the existing CNR Overhead in Novar and is accessible from local Secondary Highway 592 from the south

and Old Muskoka Road from the north.

2.2 Physiography

Based on The Physiography of Southern Ontario, 3rd Edition, by Chapman and Putnam, the region surrounding the site consists of bedrock ridges with shallow overburden. The bedrock is undifferentiated igneous and metamorphic rock of early Precambrian age and is generally hard and massively jointed.

The Highway 11 corridor, however, lies in a long, narrow sand plain within the region of shallow bedrock. The typical soils in the corridor consist of sand and silt, with some gravel deposited as glacial outwash or in localized glaciolacustrine environments.

From the CNR tracks southward, the area is generally flat and the ground surface is covered by topsoil and vegetal cover, including scrubby trees. To the north of the tracks, the ground rises somewhat and an existing pit reveals sand and gravel soils. The existing Highway 11 passes through a cut immediately north of the site and the soils exposed in the face of the cut are also sand and gravel.

Visual inspection of the local area suggests poor drainage and a high water table.

3. INVESTIGATION PROCEDURES

3.1 Field investigation

Between January 16 and February 17, 1998, track mounted drill rigs were used on site for drilling, Standard Penetration Testing (SPT) (following the procedure outlined in ASTM D 1586) and dynamic cone penetration testing. One sampled borehole was drilled at each of the two pier and two abutment locations, and at each approach fill, giving a total of six

boreholes. The approximate borehole locations are shown on Drawing 19-1351-7e-01.

The drilling and sampling at each borehole was started using continuous flight hollow stem auger equipment. In several holes the progress of drilling by auger was impeded by the presence of numerous cobbles and boulders in the soil and eventually effective refusal to further auger penetration was encountered. When this occurred, the drilling method was switched to mud rotary which generally obtained further penetration. In some cases, when the drilling encountered a boulder and became too arduous, the borehole was relocated a short distance and drilling recommenced from that location.

The boreholes were numbered E-98-1 through E-98-6. The depths of sampling in the six boreholes were as follows:

Borehole No.	Depth of Sampling (m)
E-98-1	7.6
E-98-2	9.3
E-98-3	11.0
E-98-4	14.6
E-98-5	17.1
E-98-6	8.1

Standard Penetration Tests (SPT) were carried out in all boreholes to assess the relative density of the soils in place and to obtain soil samples for the purposes of identification and laboratory testing. SPTs were conducted at intervals of 0.75 m in the upper 3.0 m and generally at intervals of 1.5 m thereafter. In some instances, sampling was not possible as proposed and the intervals were varied to suit the bouldery soil conditions.

Dynamic cone penetration tests, used to supplement the SPT data, were conducted at selected boreholes as follows:

Borehole No.	Depth of Dynamic Cone Test
E-98-1	Ground surface to 6.1 m, adjacent to the sampled borehole
E-98-4	At intervals between depths of 12.3 and 14.3 m in the bottom of the sampled borehole

Boreholes were not left open long enough for the groundwater to stabilize and this coupled with the constant addition of drilling fluid led to the decision not to report data on groundwater levels in open boreholes on completion of drilling. On completion of drilling and sampling and based on the relatively flat surface and permeable soil conditions across the area of interest, one standpipe piezometer was installed in Borehole E-98-4 to monitor the groundwater level.

The boreholes were backfilled with drill cuttings except in Borehole E-98-4 where sand pack was installed around the piezometer tip and a bentonite plug was installed near the top of the hole.

All recovered samples were examined, identified and logged in the field and retained in the care of the field supervisor. The samples were later transported to Thurber's Toronto laboratory by the field supervisor for further examination and laboratory analysis.

The results of the drilling, sampling and in-situ testing are summarized on the borehole logs in Appendix A.

3.2 Laboratory Analysis

The geotechnical laboratory testing included natural moisture content

determinations and visual classifications on all recovered samples. In addition, grain size analysis, Atterberg limits, pH and sulphate testing were conducted on selected samples. The results of the laboratory testing are presented on the borehole logs in Appendix A and in Figures B1 to B4 and Table 1 in Appendix B.

It should be noted that the grain size analyses were conducted on samples retrieved in the split spoon sampler and therefore exclude all material larger than approximately 35 mm. While the results are useful in assessing the composition of the soil matrix they should not be regarded as representative of the total grain size range of the soil in-situ. In particular, the conditions noted during drilling make it clear that there are numerous cobbles and boulders in the soil deposits encountered on this site.

4. DESCRIPTION OF SUBSURFACE CONDITIONS

4.1 Subsurface Soil Conditions

Detailed descriptions of the individual strata encountered in the boreholes are presented on the borehole logs in Appendix A. The stratigraphic profile inferred from the borehole information is shown on Drawing No. 19-1351-7e-01.

Examination of the profile shows that the basal stratigraphic unit is a buried mound of compact to very dense sand with cobbles and boulders. The mound peaks at or close to the ground surface (Elevation 324.5) in the area of Boreholes E-98-2 and E-98-3 (the south abutment and south pier) and slopes down to the north and to the south. The presence of similar soils was encountered in the investigation carried out for the design of the foundations of the existing Highway 11 structure immediately to the west.

Within the zone of exploration and at lower levels (below Elevation 319.3 and 317.0), the flanks of the buried mound of sand with cobbles and boulders are overlain by cohesive soils. The soil encountered on the south

flank appeared layered, suggesting a lacustrine origin, while that on the north flank was more heterogeneous. These soils were encountered in only one sample in each of two boreholes and it is difficult to determine if they represent one stratum present on both flanks of the mound or are the result of different depositional events.

The mound of coarse granular soil and the cohesive soils described above are overlain by sand and silt with a layered appearance, suggesting lacustrine origin. These soils are generally loose to compact and extend up to the base of the shallow surficial layer of peat, except in Borehole E-98-2 which shows the mound of coarse granular soils extending essentially to ground surface.

Descriptions of the general soil strata encountered are as follows.

Peat

The surficial peat deposit consisted of a highly organic, fibrous peat with numerous roots. This deposit is generally loose, ranges from dry to wet and has colours ranging from brown to black. The water content of the peat deposit ranged from 25 to 58%. The peat ranged in thickness from 0.4 to 0.8 m, but was absent at Boreholes E-98-4 and E-98-6, probably as a result of past construction or related activities.

Sand with Cobbles and Boulders (Glacial Outwash Soils)

These soils form the buried mound described above and centred about Boreholes E-98-2 and E-98-3.

The deposit consists of a sand matrix with varying proportions of silt, and gravel and containing numerous cobbles and boulders. The deposit has been classified as compact to very dense on the basis of SPT N-values ranging from 21 to values well in excess of 100, for 0.3 m penetration. Two individual SPT values of 4 and 10 were recorded at the top of the deposit. It is recognized that some of the very high SPT N-values may be due to the

presence of cobbles and boulders. The natural moisture contents measured for this soil ranged from 6 to 22%

This stratum was not positively identified in Borehole E-98-1, but effective refusal to further sampling was encountered at Elevation 317.8, possibly indicating that the top of the stratum had been reached.

The base of the deposit was not encountered in any of the boreholes, but the results of Borehole E-98-5 show that it extends to Elevation 308.7 or deeper. The top of the deposit was encountered at elevations ranging from 325.5 at Borehole E-98-2 to 315.3 at Borehole E-98-5.

Heterogeneous Mixture of Clay, Sand and Silt (Glacial Till)

A layer of cohesive soil, interpreted to be less than 2 m thick overlies the lower levels of the north flank of the coarse granular soils. The top of this deposit is noted at Elevation 317.0 in Borehole E-98-5. The soil is a clay, silty, some sand and has a heterogeneous structure suggesting glacial till. The SPT N-value measured in this layer was 17 blows for 0.3 m of penetration. The natural moisture content was 28%.

Lacustrine Sand and Silt

The main soil type overlying the flanks of the buried mound of coarse granular soil was a deposit of sand and silt. This deposit was layered, including some thin clayey seams suggesting a lacustrine origin.

The relative density of the deposit is classified as loose to compact, based on SPT N-values ranging from 4 to 27 blows for 0.3 m of penetration. Some higher values were recorded on the north half of the site at depths of approximately 2.0 m and these are attributed to the presence of scattered

cobbles and boulders. The measured natural moisture contents ranged from 13 to 31%.

This deposit extended from the ground surface, or the underside of the peat, to depths of 6.1 m (Elevation 319.3) at Borehole E-98-1 and 8.8 m (Elevation 317.0) at Borehole E-98-5.

Lacustrine Clay

A layer of lacustrine clay was encountered only in Borehole E-98-1 at Elevation 319.3. The borehole was terminated on effective refusal to sampling at Elevation 317.8, giving a thickness of clay of 1.6 m. In-situ test results indicate that the clay is stiff and of low plasticity. A water content of 30% was measured in this layer. Results of gradation and Atterberg Limit tests on a selected sample of this layer are presented in Figure B1 in Appendix B.

4.2 Groundwater

Water levels in the open boreholes ranged from 1.8 m (BH E-98-5) to 2.2 m (BH E-98-1) below ground surface.

The following groundwater levels were recorded in the piezometer installed in Borehole E-98-4:

Date	Depth to Water (below existing ground surface)
Feb 12, 1998	5.8 m
May 24, 1998	3.5 m
July 31, 1998	3.4 m
September 8, 1998	3.5 m

Based on this data, the existing groundwater level lies at Elevation 322.7. This value is based on short term readings and may fluctuate throughout the year, in particular rising after the spring thaw.

5. RECOMMENDATIONS FOR STRUCTURE FOUNDATIONS

5.1 Structure General Arrangement

Highway 11 is a north-south highway and this report deals with work proposed on the new northbound lanes. North bound Highway 11 has been adopted as construction north for this report.

The proposed structure will be a three-span bridge carrying Highway 11 over the CNR tracks at Novar, Ontario. The bridge structure will have a 19.5 m central span, two 16 m side spans and a deck width of approximately 14 m. The approximate location on the highway is Station 11+017 NBL. Geotechnical design recommendations are required for the design of foundations at:

- a. the north abutment
- b. the north pier
- c. the south pier
- d. the south abutment

Geotechnical recommendations are also required for the design of the immediate approach fills, which will be in the order of 10 m high.

5.2 Foundation Soil Conditions

The factual description of the foundation soils is presented in Section 4 of the report. A discussion of the soil conditions influencing design of the individual foundation elements is presented below.

South Abutment

The foundation soil conditions for the south abutment are represented by Borehole E-98-2. The upper 0.4 m consists of peat and it is recommended that this soil layer be stripped as part of the site preparation prior to embankment construction.

The bottom of the abutment stem will lie approximately at Elevation 331, based on the General Arrangement provided by McCormick Rankin Corporation.

Approach fill will be placed from the top of the native mineral soil at Elevation 325.5 to final grade. Thus from the underside of the abutment stem at Elevation 331 to the top of mineral soil at Elevation 325.5, the soils to be considered in foundation analysis will be the approach fill.

Below Elevation 325.5 and to the depth of exploration in Borehole E-98-2 at Elevation 316.6, the foundation soils consist of compact to very dense sand with gravel layers and numerous boulders.

South Pier

The foundation soil conditions for the south pier are represented by Borehole E-98-3.

Based on the General Arrangement drawing, the underside of foundation at this pier will lie at about Elevation 324.0.

The soils lying below the foundation level consist of dense to very dense sand with some gravel and boulders to Elevation 320.8. From that elevation to the depth of exploration at Elevation 315.6, the soils consist of dense to very dense sand and gravel.

North Pier

The foundation soil conditions for the North Pier are represented by Borehole E-98-4.

Based on the General Arrangement drawing, the underside of foundation at this pier will lie at about Elevation 324.0.

Below the foundation and to Elevation 322.2, the foundation soils consist of compact to very dense silts which may contain some cobbles and boulders. From that elevation and to the depth of exploration at Elevation 311.6, the soils consist of approximately 2.0 m of sand and gravel over sand. These soils are in a compact to very dense condition and contain cobbles and boulders.

North Abutment

The foundation soil conditions for the north abutment are represented by Borehole E-98-5. The upper 0.5 m consists of peat and it is recommended that this soil layer be stripped as part of the site preparation prior to embankment construction.

Based on the General Arrangement drawing, the underside of foundation at this abutment will lie at about Elevation 332.

Approach fill will be placed from the top of the native mineral soil at Elevation 325.3 to final grade. Thus from the underside of the abutment stem at Elevation 332 to the top of mineral soil at Elevation 325.3, the soils to be considered in foundation analysis will be the approach fill.

From the underside of the fill to Elevation 317.0, the soils consist of a

sequence of silts and sands which generally are in a loose to compact condition. Between Elevations 317.0 and 315.3, there is a layer of stiff to very stiff clay till. The clay till is underlain by compact to very dense sands which extend to the depth of exploration at Elevation 308.7.

5.3 Suitable Foundation Types

The subsoil stratigraphy and groundwater conditions established in the course of the investigation were reviewed for the purpose of making an assessment of suitable foundation systems. In the review process, an initial preference was given to a system of integral abutments.

In considering the suitability of a site for integral abutment design, a number of additional factors relating to the piles must be considered, including the following:

- the foundation must be flexible, generally meaning that the upper 3.0 m of the pile must be free to move
- the foundation performance should be the same for both abutments
- the abutment must be supported on a single row of piles, necessitating close control of the pile locations during driving.

The required flexibility can be achieved since the abutment piles will be driven through approximately 4 m of approach embankment fill before encountering native soil and if necessary, pre-drilling could also be used.

The anticipated performance of the two abutments, however, will not be the same. The soil conditions controlling the performance of the south and north abutments are represented by Boreholes E-98-2 and E-98-5, respectively. At the south abutment, the piles would be driven through the approach fill and immediately into the compact to very dense sand with cobbles and boulders. At the north abutment, the piles would be driven

through a similar thickness of fill and then through 10 m of loose to compact lacustrine soils and stiff to very stiff clay till before encountering the compact to very dense sand with cobbles and boulders. Based on these dissimilar soil conditions, the lateral resistance and deformation characteristics of the two abutments would also be dissimilar.

With respect to pile location, the piles for an integral abutment must be driven in a single row, which must be contained within the thickness of the abutment stem and achieve correct embedment within the concrete. There is, therefore, less tolerance of piles being out of location or out of plumb than there would be in the case of a conventional pile cap. The presence of numerous boulders in the foundation soil indicates that close control of the pile location and plumbness will be difficult. Pre-drilling before driving the piles might be enough to control the location but would necessitate grouting the pre-drilled hole after the pile is installed. The grouted pile would be much stiffer than a conventional driven pile. Due to the high groundwater table, grouting would have to be carried out by tremie or pressure grouting methods.

In addition to the foundation concerns related to the soils, it is noted that the bridge is proposed to have a skew of 31°. At that angle, integral abutments require a more rigorous analysis. The dissimilar soil conditions and high probability of pre-drilling being required is expected to further reduce the probability of the analysis resulting in a favourable outcome.

When all the above factors are taken into account, the recommendation from a geotechnical standpoint is to use conventional, rather than integral abutments.

The viability of spread footings was also considered. As illustrated on the soil profile, Drawing 19-1351-7e-01, the potential support for spread footings is variable across the site. Footings would be founded close to the

water table and at the north pier and abutment in particular would be underlain by a zone of loose lacustrine sediment. The use of spread footings is therefore not recommended.

In conclusion, the recommended foundation system is driven piles with conventional pile caps. Due to the nature of the anticipated support stratum, sand with cobbles and boulders, H-piles are recommended.

5.4 Piled Foundation

5.4.1 Axial Capacity

Due to the presence of numerous boulders in the soil deposits, the use of a heavier pile section driven by a hammer with a higher rated energy may be appropriate. Accordingly, computed geotechnical resistances are provided at each foundation element for the following two pile sizes:

- HP310X110 driven by a hammer delivering at least 60 kJ per blow
- HP310X132 driven by a hammer delivering at least 70 kJ per blow

All piles must be fitted with driving shoes.

In all cases, the geotechnical resistances provided must be checked against the structural capacity of the pile section.

South Abutment

In computing the capacity of piles at the abutment, it has been assumed that the fill placed for the immediate approach embankment will consist of earth fill. The underside of the abutment pile cap has been assumed at Elevation 331.0, approximately.

The following geotechnical resistances may be assumed to be available for the driven H-piles described above:

Pile Size	Driven Length (m)*	Tip Elevation (m)	Factored ULS (kN)	SLS (kN)
HP310X110	12	319.00	1,600	1,100
HP310X132	14	317.00	1,900	1,250

* Measured below pile cap.

South Pier

In computing the available geotechnical resistances, it has been assumed that the piles will be driven entirely in the mineral soils described in Borehole E-98-3, that the underside of the pile cap will lie at Elevation 324.0, approximately, and that finished grade will be 326.0 over the pile cap.

The following geotechnical resistances may be assumed to be available for the driven H-piles described above:

Pile Size	Driven Length (m)*	Tip Elevation (m)	Factored ULS (kN)	SLS (kN)
HP310X110	11	313	1,600	1,100
HP310X132	12	312	1,900	1,250

* Measured below pile cap.

North Pier

In computing the available geotechnical resistances, it has been assumed that the piles will be driven entirely in the mineral soils described in



Borehole E-98-4, that the underside of the pile cap will lie at Elevation 324.0 and that finished grade will be 326.0 over the pile cap.

The following geotechnical resistances may be assumed to be available for the driven H-piles described above:

Pile Size	Driven Length (m)*	Tip Elevation (m)	Factored ULS (kN)	SLS (kN)
HP310X110	11	313.00	1,600	1,100
HP310X132	12	312.00	1,900	1,250

* Measured below pile cap.

North Abutment

In computing the capacity of piles at the abutment, it has been assumed that the fill placed for the immediate approach embankment will consist of earth fill. The underside of the abutment has been assumed at Elevation 332.0.

The following geotechnical resistances may be assumed to be available for the driven H-piles described above:

Pile Size	Driven Length (m)*	Tip Elevation (m)	Factored ULS (kN)	SLS (kN)
HP310X110	20	312.00	1,600	1,100
HP310X132	21	311.0	1,900	1,250

* Measured below pile cap.

5.4.2 Pile Spacing

The minimum spacing of piles should be determined in accordance with OHBDC Clause 6-11.1.

5.4.3 Lateral Resistance

It is recommended that the lateral loads transferred to foundations be resisted by means of battered piles.

5.4.4 Pile Installation

Pile driving should be carefully monitored and controlled employing the Hiley Dynamic Pile Driving Formula in accordance with MTO Standards SS 103-10 or SS 103-11 and assuming ultimate resistances of:

Pile Size	Ultimate Resistance (kN)
HP310X110	3,300
HP310X132	3,750

After each pile is driven to its specified tip elevation, the elevation of the top of the pile should be recorded and checked periodically to ensure that it is not heaving due to the effects of driving adjacent piles. Piles that heave must be redriven. Allowance should be made for restriking piles in order to check for possible relaxation (decrease in bearing capacity with time). Initially, at least one third of the piles should be restruck at least one day after initial installation, with the number increased as required if pile

relaxation is noted.

5.4.5 Pile Driving Note

The pile driving note to be added to the drawings should be Note 2 in Clause 2.5.11 of the Structural Manual. The ultimate resistances to be used are 3,300 kN if HP310X110 piles are used or 3,750 kN if HP310X132 piles are used.

At this site, the stratigraphy varies along the span of the bridge and a separate Pile Driving Note is recommended for each foundation element. The elevations to be used in the notes are:

Foundation Element	Elevation for Pile Driving Note	
	HP310X110	HP310X132
South Abutment	319.0	317.0
South Pier	313.0	312.0
North Pier	313.0	312.0
North Abutment	312.0	311.0

5.4.6 Pile Installation Monitoring

During pile installation, high resistance to penetration may be encountered above the specified tip elevation, caused by obstructions such as the presence of boulders. If this occurs, and pile penetration to the specified elevation cannot be achieved without damage to the pile, driving must be suspended until the driving record is reviewed by a geotechnical engineer to determine if sufficient geotechnical resistance has been developed.

6. EARTH PRESSURE

The lateral earth pressures to be used in design of the abutment and wing walls should be computed in accordance with Section 6-7 of the OHBDC .

Backfill behind the abutment walls and wing walls within wedge extending upwards from the base of the pile cap, should consist of free draining granular backfill meeting the minimum requirements of Ontario Provincial Standard Specifications (OPSS) for Granular B, Type I. These requirements are illustrated in OPSD 3501.00. The fill should be placed in maximum lift thickness of 150 mm and compacted in accordance with OPSS 501 using hand operated (walk behind) compaction equipment close to the abutment. A perforated subdrain should be installed behind the base of the walls as shown in OPSD 3501.00 to maintain the granular fill in a drained condition. The subdrain should be provided with a positive outlet to the highway drainage system.

For the above backfill and drainage conditions, the abutment walls and wing walls may be designed based on the following unfactored earth pressure distributions:

$$P_h = K \gamma h$$

where;

K = earth pressure coefficient, use value from table below.

γ = unit weight of soil, = 21.2 kN/m³ for Granular B

h = depth below top of wall, m

Wall Type	Earth Pressure Coefficient (K)	
	OPSS Granular B $\phi' = 30^\circ$	
	Horizontal Ground Surface Behind Wall	Sloping Ground Surface (2H:1V)
Abutment Walls (Restrained Wall)	0.50	-
Wing Walls (Unrestrained Wall)	0.33	0.55

If the wing walls will not be connected to the abutments and therefore will be able to accommodate some rotation they may be treated as unrestrained. The above also assumes a horizontal ground surface behind the abutment walls and a ground surface inclined at 2:1 behind the wing walls. If concrete approach slabs are not provided, an additional load equivalent to 600 mm of fill should be superimposed on the wall loadings to account for traffic surcharge loading.

Additional lateral pressure must be added to account for compaction induced forces. The additional pressure must be computed in accordance with Section 6-7.4.3 of the OHBDC.

7. TEMPORARY SUPPORT TO CN TRACKS

If excavation is carried out within the limits prescribed by CNR, then temporary protection must be provided for the tracks. At the north pier, any shoring that might be required will be constructed in silt and fine sand soils. At the south pier, the excavation may penetrate through the silt and fine sand and into the underlying deposit of sand with cobbles and boulders. The following earth

pressure coefficients are suggested for the design of shoring:

Soil Type	K_a	K_p	Unit Weight (γ) kN/m ³
Silt and fine sand	0.39	2.56	20
Sand with cobbles and boulders	0.29	3.44	22

The above earth pressure coefficients are for a horizontal ground surface behind the shoring and should be modified if the ground surface is sloping above the shoring.

It is recommended that, due to local variations in stratigraphy, the shoring be designed conservatively using the parameters for silt and fine sand.

The design must take account of the appropriate railway loadings and any other surcharge that may act above the shoring.

For design purposes, the groundwater should be assumed to be at Elevation 322.7.

The selection and design of the temporary shoring system should be left to the contractor, subject to approval of the Engineer. However, considering the soil and groundwater conditions prevailing at this site, one suggested method of shoring is soldier piles and lagging. The system should consist of driven steel H-piles as the soldier piles together with wood lagging.

It is possible to design the shoring system support on the basis of either cantilever piles or piles supported by a waler and rakers or cross-bracing. However, in view of the proximity of the tracks and the objective of minimizing movement of the tracks, a system with a stiff bracing near the top of the piles may be preferable.

The lagging boards should be installed as excavation proceeds and all spaces behind the lagging must be immediately filled to prevent further of ground. The prevention of loss of ground is especially important where the shoring provides track protection.

8. EMBANKMENT DESIGN

Based on the design profile shown on the General Arrangement Drawing provided by McCormick Rankin Corporation and dated May 1998, the immediate approach embankments will be approximately 10 m high above existing grade.

As shown on the borehole logs and the interpreted soil profile attached in Drawing 19-1351-7e-01, a distinct surface layer of peat was noted. It is recommended that the noted organic soil be removed in the area of the immediate approach embankment to limit differential settlements between the embankment and the structure. The interpreted depths of organic soil to be removed are as follows:

Borehole No.	Approximate Depth of Material to be Removed (m)
E-98-1	0.8
E-98-2	0.4
E-98-3	0.8
E-98-4	--
E-98-5	0.5
E-98-6	--

Based on the measured groundwater elevation, stripping operations to remove these depths of organic soil will lie above the groundwater table. The base of the excavation may, however, become disturbed under the action of earth moving equipment.

Following stripping of the surficial organic soils, the exposed subgrade will be strong enough to support the proposed embankment. Most of the settlement occurring in the embankment foundation soils will be essentially immediate and will occur as the embankment is being constructed.

Earth fill may be used in embankment construction, subject to the Non Standard Special Provision (NSSP) in Appendix C.

Embankments for the CNR Overhead constructed of common earth fill should generally have side slopes not steeper than 3:1, and all peat must be stripped from below the embankment. If these embankments are specified to be constructed of Select Subgrade Material (SSM), then side slopes not steeper than 2:1 may be used, provided all peat is stripped.

The forward slope under the bridge may be constructed at 2:1 provided it is constructed with earth fill meeting the NSSP in Appendix C and the slope face is covered with concrete slope paving.

In either of the above cases, where the embankment height will exceed 8.0 m, a berm 2.0 m wide should be provided on the embankment side slopes in each 8.0 m vertical interval. Provided the forward slope does not exceed 8.0 m in height, measured from the toe to the point where it intersects the face of the abutment, the berm on the side slope need not be carried into the forward slope. In this case, the side slope berm may be transitioned from 2.0 m at the end of the wing wall to zero width at the concrete slope paving on the forward slope. The transition from a 3:1 side slope to a 2:1 forward slope may also occur in this zone.

Embankment fill should be placed in appropriate lift thicknesses and be compacted in accordance with OPSS 501.

Computer analyses has been carried out which indicate that the recommended embankment configurations are stable from both a global perspective and

surficially.

9. EXCAVATION AND GROUNDWATER CONTROL

The groundwater level established through piezometer readings at Borehole E-98-4 is Elevation 322.7, corresponding to a depth of 3.5 m below the existing ground surface.

At the two abutments, pile driving and pile cap construction will take place from a level above existing ground surface and neither short term nor long term groundwater control will be required..

At the pier locations, temporary excavation for the piles caps is anticipated to extend to Elevation 324.0. The base of the excavation, therefore, is expected to lie some 2 m above the current groundwater level as measured in the piezometer. This data indicates that groundwater control should not be required for pile cap construction at the piers unless the ground water level rises due to seasonal fluctuations. If higher groundwater levels are encountered, it is anticipated that the seepage water can be removed by means of pumping from sumps.

All excavations must be carried out in accordance with the Ontario Occupational Health and Safety Act (OHSA) and having regard to the CN requirements for excavation adjacent to their tracks. For the purpose of the OHSA, the near surface soils at the pier locations are classified as Type 3 above the groundwater table and Type 4 below.

10. FROST PROTECTION

The design depth of frost penetration for this project is 1.8 m. All pile caps, grade beams and footings designed for this site must be provided with a minimum depth of soil cover of 1.8 m to protect against the penetration of frost below the foundation elements.

11. CONSTRUCTION CONCERNS

Construction of the proposed structure will include driving steel piles for the four major foundation units. The field investigation has revealed the presence of numerous boulders, which are expected to make the control of driven piles difficult. The progress and results of the pile driving must be carefully monitored and assessed to determine if the piles are achieving the required capacity and are penetrating to the specified depths. The control of the location and plumbness of the completed piles is also of concern at this site.

12. CONSTRUCTION INSPECTION AND MONITORING

During construction, all foundation installation, excavation and embankment construction activities should be monitored by geotechnical personnel to confirm that the foundation recommendations and design are being correctly implemented and that the soil conditions encountered do not differ materially from the interpretation used in this report.

During pile installation, high resistance to penetration may be encountered above the specified tip elevation, caused by such factors as the presence of boulders. If this occurs, or if pile penetration to the specified elevation cannot be achieved without damage to the pile, driving must be suspended until the driving record is reviewed by a geotechnical engineer to determine if sufficient geotechnical resistance has been developed..

It is recommended that a Non Standard Special Provision (NSSP) be included in the contract to draw the contractors attention to the potential pile driving problems. Suggested text for the NSSP is included as Appendix D.



STATEMENT OF GENERAL CONDITIONS

1. STANDARD OF CARE

This study and Report have been prepared in accordance with generally accepted engineering or environmental consulting practices in this area. No other warranty, expressed or implied, is made.

2. COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report which is of a summary nature and is not intended to stand alone without reference to the instructions given to us by the Client, communications between us and the Client, and to any other reports, writings, proposals or documents prepared by us for the Client relative to the specific site described herein, all of which constitute the Report.

IN ORDER TO PROPERLY UNDERSTAND THE SUGGESTIONS, RECOMMENDATIONS AND OPINIONS EXPRESSED HEREIN, REFERENCE MUST BE MADE TO THE WHOLE OF THE REPORT. WE CANNOT BE RESPONSIBLE FOR USE BY ANY PARTY OF PORTIONS OF THE REPORT WITHOUT REFERENCE TO THE WHOLE REPORT.

3. BASIS OF REPORT

The Report has been prepared for the specific site, development, design objectives and purpose that were described to us by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the document are only valid to the extent that there has been no material alteration to or variation from any of the said descriptions provided to us unless we are specifically requested by the Client to review and revise the Report in light of such alteration or variation.

4. USE OF THE REPORT

The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client. NO OTHER PARTY MAY USE OR RELY UPON THE REPORT OR ANY PORTION THEREOF WITHOUT OUR WRITTEN CONSENT. WE WILL CONSENT TO ANY REASONABLE REQUEST BY THE CLIENT TO APPROVE THE USE OF THIS REPORT BY OTHER PARTIES AS "APPROVED USERS". The contents of the Report remain our copyright property and we authorize only the Client and Approved Users to make copies of the Report only in such quantities as are reasonably necessary for the use of the Report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make the Report, or any portion thereof, available to any party without our written permission. Any use which a third party makes of the Report, or any portion of the Report, are the sole responsibility of such third parties. We accept no responsibility for damages suffered by any third party resulting from unauthorized use of the Report.

5. INTERPRETATION OF THE REPORT

a) Nature and Exactness of Soil and Contaminant Description: Classification and identification of soils, rocks, geological units, contaminant materials and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgemental in nature and even comprehensive sampling and testing programs, implemented with the appropriate equipment by experienced personnel, may fail to locate some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and all persons making use of such documents or records should be aware of, and accept, this risk. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. Where special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.

(see over...)

INTERPRETATION OF THE REPORT *(continued)*

- b) Reliance on Provided Information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to us. We have relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, we cannot accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of misstatements, omissions, misrepresentations, or fraudulent acts of persons providing information.

6. RISK LIMITATION

Geotechnical engineering and environmental consulting projects often have the potential to encounter pollutants or hazardous substances and the potential to cause an accidental release of those substances. In consideration of the provision of the services by us, which are for the Client's benefit, the Client agrees to hold harmless and to indemnify and defend us and our directors, officers, servants, agents, employees, workmen and contractors (hereinafter referred to as the "Company") from and against any and all claims, losses, damages, demands, disputes, liability and legal investigative costs of defence, whether for personal injury including death, or any other loss whatsoever, regardless of any action or omission on the part of the Company, that result from an accidental release of pollutants or hazardous substances occurring as a result of carrying out this Project. This indemnification shall extend to all Claims brought or threatened against the Company under any federal or provincial statute as a result of conducting work on this Project. In addition to the above indemnification, the Client further agrees not to bring any claims against the Company in connection with any of the aforementioned causes.

7. SERVICES OF SUBCONSULTANTS AND CONTRACTORS

The conduct of engineering and environmental studies frequently requires hiring the services of individuals and companies with special expertise and/or services which we do not provide. We may arrange the hiring of these services as a convenience to our Clients. As these services are for the Clients' benefit, the Client agrees to hold the Company harmless and to indemnify and defend us from and against all claims arising through such hirings to the extent that the Client would incur had he hired those services directly. This includes responsibility for payment for services rendered and pursuit of damages for errors, omissions or negligence by those parties in carrying out their work. In particular, these conditions apply to the use of drilling, excavation and laboratory testing services.

8. CONTROL OF WORK AND JOBSITE SAFETY

We are responsible only for the activities of our employees on the jobsite. The presence of our personnel on the site shall not be construed in any way to relieve the Client or any contractors on site from their responsibilities for site safety. The Client acknowledges that he, his representatives, contractors or others retain control of the site and that we never occupy a position of control of the site. The Client undertakes to inform us of all hazardous conditions, or other relevant conditions of which the Client is aware. The Client also recognizes that our activities may uncover previously unknown hazardous conditions or materials and that such a discovery may result in the necessity to undertake emergency procedures to protect our employees as well as the public at large and the environment in general. These procedures may well involve additional costs outside of any budgets previously agreed to. The Client agrees to pay us for any expenses incurred as the result of such discoveries and to compensate us through payment of additional fees and expenses for time spent by us to deal with the consequences of such discoveries. The Client also acknowledges that in some cases the discovery of hazardous conditions and materials will require that certain regulatory bodies be informed and the Client agrees that notification to such bodies by us will not be a cause of action or dispute.

9. INDEPENDENT JUDGEMENTS OF CLIENT

The information, interpretations and conclusions in the Report are based on our interpretation of conditions revealed through limited investigation conducted within a defined scope of services. We cannot accept responsibility for independent conclusions, interpretations, interpolations and/or decisions of the Client, or others who may come into possession of the Report, or any part thereof, which may be based on information contained in the Report. This restriction of liability includes decisions made to either purchase or sell land.

APPENDIX A

BOREHOLE LOGS

- Symbols and Terms Used on Borehole Logs

- Unified Soil Classification

- Borehole Logs E-98-1 to E-98-6





BOREHOLE GRAPHIC SYMBOLS

SOILS



FILL

ORGANICS

CLAY

SILT

SAND

GRAVEL

COBBLES



SILTY CLAY

CLAYEY SILT

SILTY SAND

SAND & GRAVEL

CLAYEY SILT TILL

SILTY CLAY TILL

SANDY SILT TILL

ROCK



SHALE

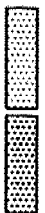
LIMESTONE



SILTSTONE

GRANITE

OTHER



CEMENT GROUT

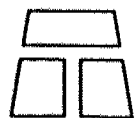
BENTONITE GROUT



CONCRETE

WATER

BENTONITE SEAL



THURBER

SYMBOLS AND TERMS USED ON TEST HOLE LOGS

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	Less than 10	Less than 2
Soft	10 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 TEST HOLE LOGS

SYMBOLS FOR SAMPLE TYPE	Shelby Tube	A - Casing
<input checked="" type="checkbox"/>	SPT	<input type="checkbox"/> Grab/Auger sample
<input checked="" type="checkbox"/>	No Recovery	<input type="checkbox"/> Core

• MC - Moisture Content (% by Weight) as determined by sample]

W Water Level

C_{vane} Shear Strength Determination by Field Insitu Vane

C_{pen} Shear Strength Determination by Pocket Penetrometer

C_{lab} Shear Strength Determination using a Laboratory Vane Apparatus

C_u Undrained Shear Strength determined by Unconfined Compression Test

(1) SPT Standard Penetration Test - refers to the number the blows from a 63.5kg hammer falling through 0.76m to advance a 60 degree truncated cone 0.3m.

UNIFIED SOILS CLASSIFICATION

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

RECORD OF BOREHOLE No E-98-1

1 OF 1

METRIC

W.P. 462-93-00 LOCATION CNR OVERHEAD (NBL), N 5 035 064.2 E 324 308.9 ORIGINATED BY EK
DIST 52 HWY 11 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY WM
DATUM GEODETIC DATE 98.01.16 - 98.01.16 CHECKED BY AG





SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT Y kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				
325.4							20 40 60 80 100					
0.0	PEAT, brown, moist to wet: (PT)		1	SS	2						55.06	
324.6												
0.8	SAND and SILT, fine grained, occasional seams of wet coarse sand, occasional seams of wet oxidized sand, compact, brown, moist to wet: (ML/SP)		2	SS	13							
323.9												
1.5	SILT, sandy, occasional layers of coarse oxidized sand, compact to very loose, grey: (ML-NONPLASTIC)		3	SS	20							
			4	SS	16							
321.7			5	SS	6							
3.7	trace to some clay											0 23 64 13
			6	SS	0							
319.3												
6.1	CLAY, silty, to SILT, clayey, trace sand, soft, slightly varved, grey: (LACUSTRINE)(CL-ML)		7	SS	2							0 6 61 33
317.8	some gravelly sand in last sample auger refusal on boulder											
7.6	END OF BOREHOLE AT 7.65m. WATER LEVEL AT 2.20m. BOREHOLE OPEN TO 2.74m. BOREHOLE BACKFILLED WITH DRILL CUTTINGS. Notes: 1) Moved ahead 1.5m and drove dynamic cone. 2) Moved ahead further 1.5m and probed by auger. Refusal at 5.5m depth.			SS	31/ .025							

RECORD OF BOREHOLE No E-98-2

1 OF 1

METRIC

W.P. 462-93-00 LOCATION CNR OVERHEAD (NBL), N 5 035 085.8 E 324 285.4 ORIGINATED BY EK
 DIST 52 HWY 11 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS/MUD ROTARY COMPILED BY WM
 DATUM GEODETIC DATE 98.01.28 - 98.01.28 CHECKED BY AG









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							10 20 30				
325.9								○ UNCONFINED + FIELD VANE											
0.0								● QUICK TRIAXIAL × LAB VANE											
325.5	PEAT, trace sand, occasional rootlets, brown		1	SS	2														
0.4	SAND, trace gravel, trace silt, loose at top then dense to very dense, light brown: (SP)		2	SS	4		325												
			3	SS	67		324							0 70 24 6					
	grinding at 2.15 to 2.29m																		
	grinding at 2.44m possible boulders		4	SS	44		323												
	compact below 3.0m		5	SS	21									2 93 5					
322.1							322												
3.8	trace to some silt, with boulders grinding at 3.81m		6	SS	30		321												
	grinding at 6.10 to 7.16m, possible boulders, cobbles gravelly, trace silt		7	SS	88/.150		320												
	augered to 7.16m, augers drifting off vertical, auger refusal						319												
318.0			8	SS	124		318							31 53 17					
7.9	coarse grained, gravelly, occasional layers of light sandy silt, very dense, brown						317												
316.6			9	SS	91														
9.3	grinding from 9.30 to 9.51m.																		
	END OF BOREHOLE AT 9.30m DUE TO REFUSAL ON BOULDER.																		
	Note: After auger refusal at 7.6m, moved BH ahead and drilled by mud rotary to 7.62m to re-start sampling. Boulders encountered from 1.5 to 6.9m.																		

RECORD OF BOREHOLE No E-98-3

1 OF 1

METRIC

W.P. 462-93-00 LOCATION CNR OVERHEAD (CNR), N 5 035 092.0 E 324 273.3 ORIGINATED BY EK
DIST 52 HWY 11 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS/MUD ROTARY COMPILED BY WM
DATUM GEODETIC DATE 98.02.05 - 98.02.05 CHECKED BY AG

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			20 40 60 80 100									
								SHEAR STRENGTH kPa									
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
							20 40 60 80 100					WATER CONTENT (%)					
												10 20 30					
326.6																	
0.0	PEAT, silty, occasional rootlets, occasional black staining, very loose, brown, dry to moist: (PT)		1	SS	3												
325.8							326										
0.8	SILT, some sand to sandy, occasional rootlets, occasional black staining, compact, brown: (ML-NONPLASTIC)		2	SS	10												
325.1							325										
1.5																	
	SAND, silty, gravelly, occasional rootlets, occasional black layers, compact to very dense, brown: (SP)		3	SS	10												
	SAND, silty, gravelly, occasional rootlets, occasional black layers, compact to very dense, brown: (SP)		4	SS	63		324										
	grinding at 2.15m, possible cobble, boulders		5	SS	71												
322.9							323										
3.7																	
	SAND, silty, trace gravel, occasional grey silt inclusions, very dense, brown: (SM)		6	SS	63		322										
	grinding at 4.14m. set up for rotary drill																
320.8							321										
5.8																	
	SAND and GRAVEL, trace to some silt, very dense, brown: (SP)		7	SS	34		320										
							319										
							318										
							317										
							316										

RECORD OF BOREHOLE No E-98-4

1 OF 2

METRIC

W.P. 462-93-00 LOCATION CNR OVERHEAD (NBL), N 5 035 105.4 E 324 256.4 ORIGINATED BY EK
 DIST 52 HWY 11 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY WM
 DATUM GEODETIC DATE 97.02.10 - 97.02.11 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										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE												
								● QUICK TRIAXIAL × LAB VANE												
326.2							20	40	60	80	100	10	20	30						
0.0	SILT, sandy, trace gravel, occasional organic partings, compact, grey to brown slight grinding from 0.45 to 0.76m, possible gravel or cobbles		1	SS	20															
			2	SS	12															
324.4	grinding at 1.22m		3	SS	54															
1.8	SILT, some fine sand, trace gravel, trace clay, layers of silt, fine sand, clay, occasional layers of oxidation, occasional oxide inclusions, laminated, compact to dense, grey to brown: (ML-NONPLASTIC)		4	SS	26															
			5	SS	45															
322.2																				
4.0	SAND and GRAVEL, trace silt, compact		6	SS	23															
	grinding at 5.03 to 5.49m, possible gravel, cobbles, boulders																			
320.4	hit boulder at 5.49m		7	SS	69/ .125															
5.8	SAND, trace to some silt, trace gravel, very dense, brown: (SP) grinding at 6.4m																			
	start tricone possible boulder or cobbles		8	SS	86															
			9	SS	36															
	substantial loss of mud at 11.28m																			
311.6																				
14.6	END OF BOREHOLE AT 14.63m.																			

Continued Next Page

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

METRIC

+ 3, \times 3: Numbers refer to Sensitivity

RECORD OF BOREHOLE No E-98-5

1 OF 2

METRIC

W.P. 462-93-00 LOCATION CNR OVERHEAD (NBL), N 5 035 118.8 E 324 241.3 ORIGINATED BY EK
 DIST 52 HWY 11 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY WM
 DATUM GEODETIC DATE 97.02.12 - 97.02.13 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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
325.8							20 40 60 80 100							
0.0	PEAT, silty, trace sand, occasional rootlets, very loose, brown		1	SS	3									
325.3														
0.5			2	SS	13		325							
	SAND, fine grained, some silt, trace gravel, occasional rootlets, occasional oxide staining, compact becoming dense, brown		3	SS	47		324							4 82 14
323.6														
2.2	SILT, trace clay, trace fine sand, occasional fissures of oxide, compact to loose, grey: (ML)		4	SS	25		323							
			5	SS	8		322							
321.2														
4.6	SAND, silty, occasional silt layers, loose to compact, grey		6	SS	5		321							
							320							
			7	SS	10		319							0 79 21
318.2														
7.6	SAND, gravelly, trace silt, compact, brown		8	SS	14		318							
317.0														
8.8	CLAY, SILT, SAND, hetogeneous mixture, stiff to very stiff: (TILL)(CL-ML)		9	SS	17		317							3 12 57 27
315.3							316							
10.5	SAND, silty, with gravel, compact to dense, grey grinding at 10.67m		10	SS	28		315							
							314							
			11	SS	49		313							
313.0														
12.8	SAND, silty, some gravel, trace clay, dense, grey grinding at 13.72m. possible cobbles and boulders becoming very dense		12	SS	39		312							8 66 20 6
							311							

Continued Next Page

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

METRIC[illegible]

+ 3, x 3: Numbers refer to Sensitivity

20
15
10
5
0

(%) STRAIN AT FAILURE

APPENDIX B

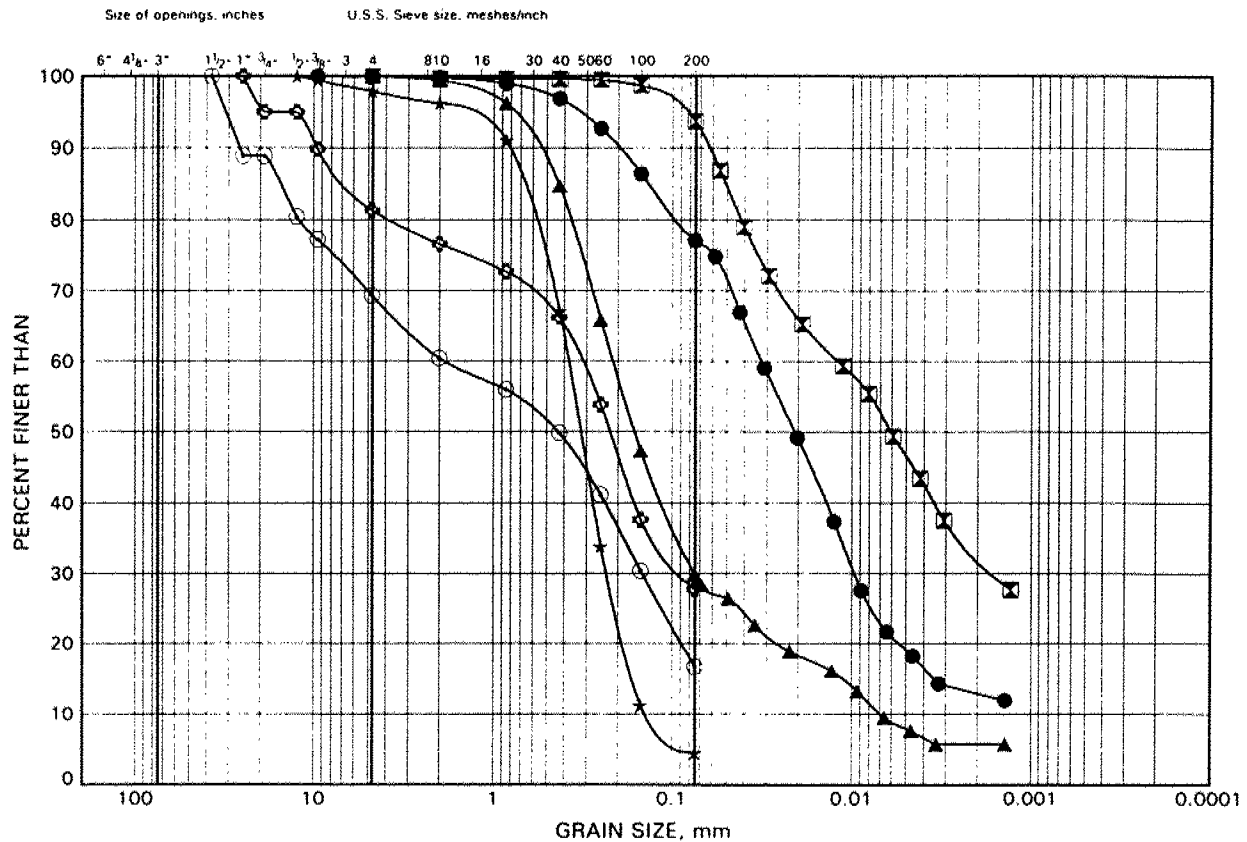
LABORATORY TEST RESULTS

- Figures B1 to B3 - Grain Size analyses
- FIGURE B4 - Plasticity Chart
- Table 1 - pH and Sulphate

CNR OVERHEAD (NBL)

GRAIN SIZE DISTRIBUTION

FIGURE B1

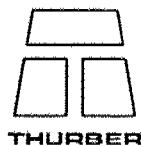


SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
--------	----------	-----------	---------------

●	E-98-1	3.28	322.12
⊠	E-98-1	6.32	319.08
▲	E-98-2	1.75	324.15
★	E-98-2	3.28	322.62
⊙	E-98-2	7.62	318.28
⊕	E-98-3	2.51	324.09

Date October 1998

Project 462-93-00

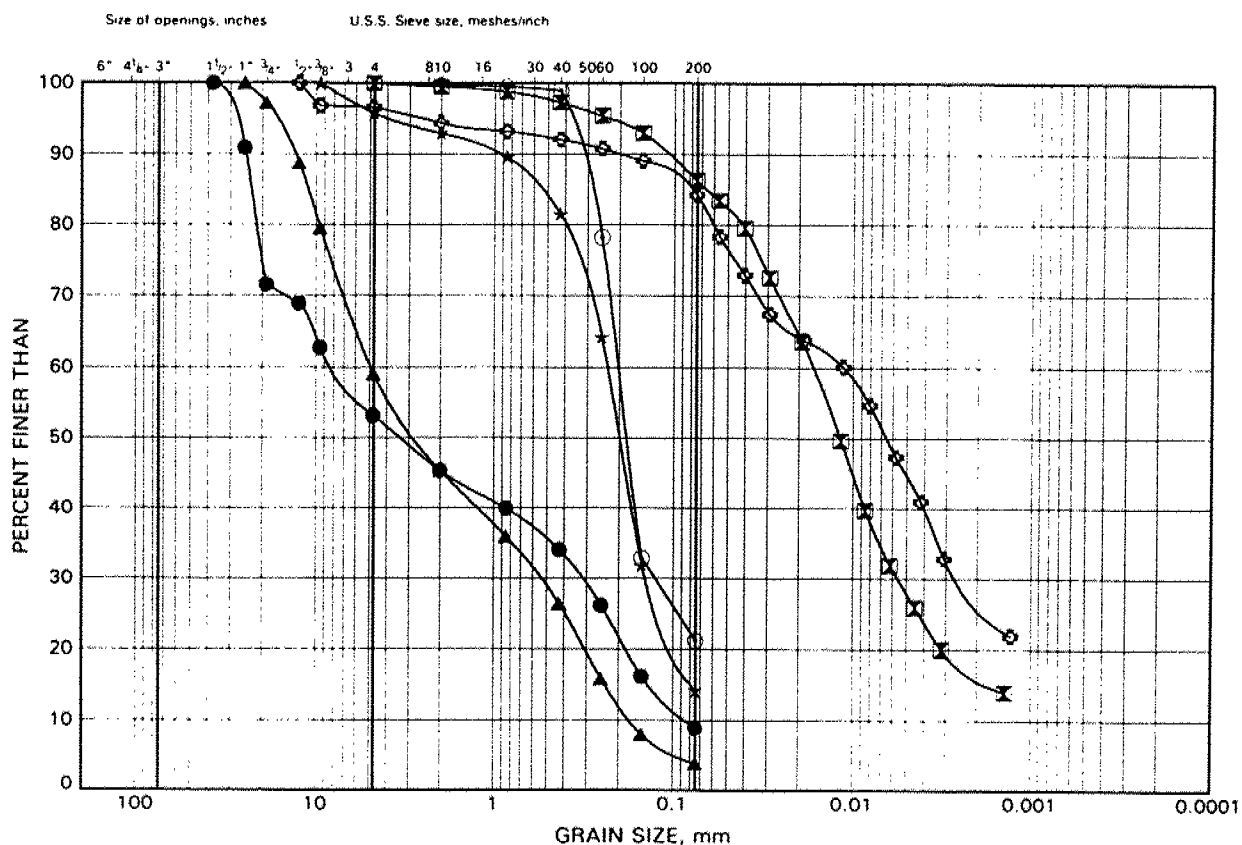


Prep'd WM

Chkd. AEG

CNR OVERHEAD (NBL) GRAIN SIZE DISTRIBUTION

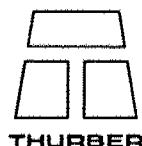
FIGURE B2



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

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	E-98-3	9.14	317.46
⊠	E-98-4	1.75	324.45
▲	E-98-4	4.80	321.40
★	E-98-5	1.75	324.05
⊙	E-98-5	6.32	319.48
◇	E-98-5	9.37	316.43

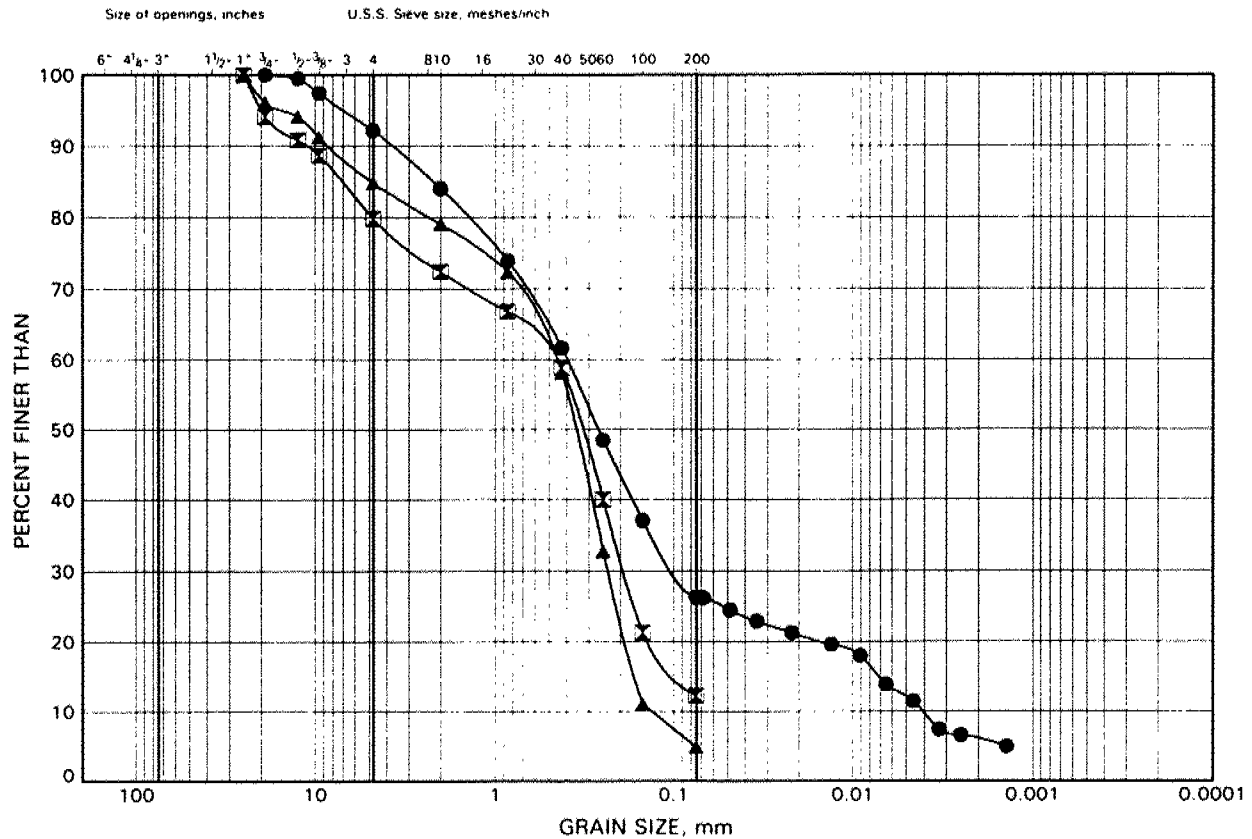
Date October 1998
Project 462-93-00



Prep'd WM
Chkd. AEG

CNR OVERHEAD (NBL) GRAIN SIZE DISTRIBUTION

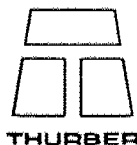
FIGURE B3



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

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	E-98-5	13.94	311.86
⊠	E-98-6	1.74	324.56
▲	E-98-6	3.28	323.02

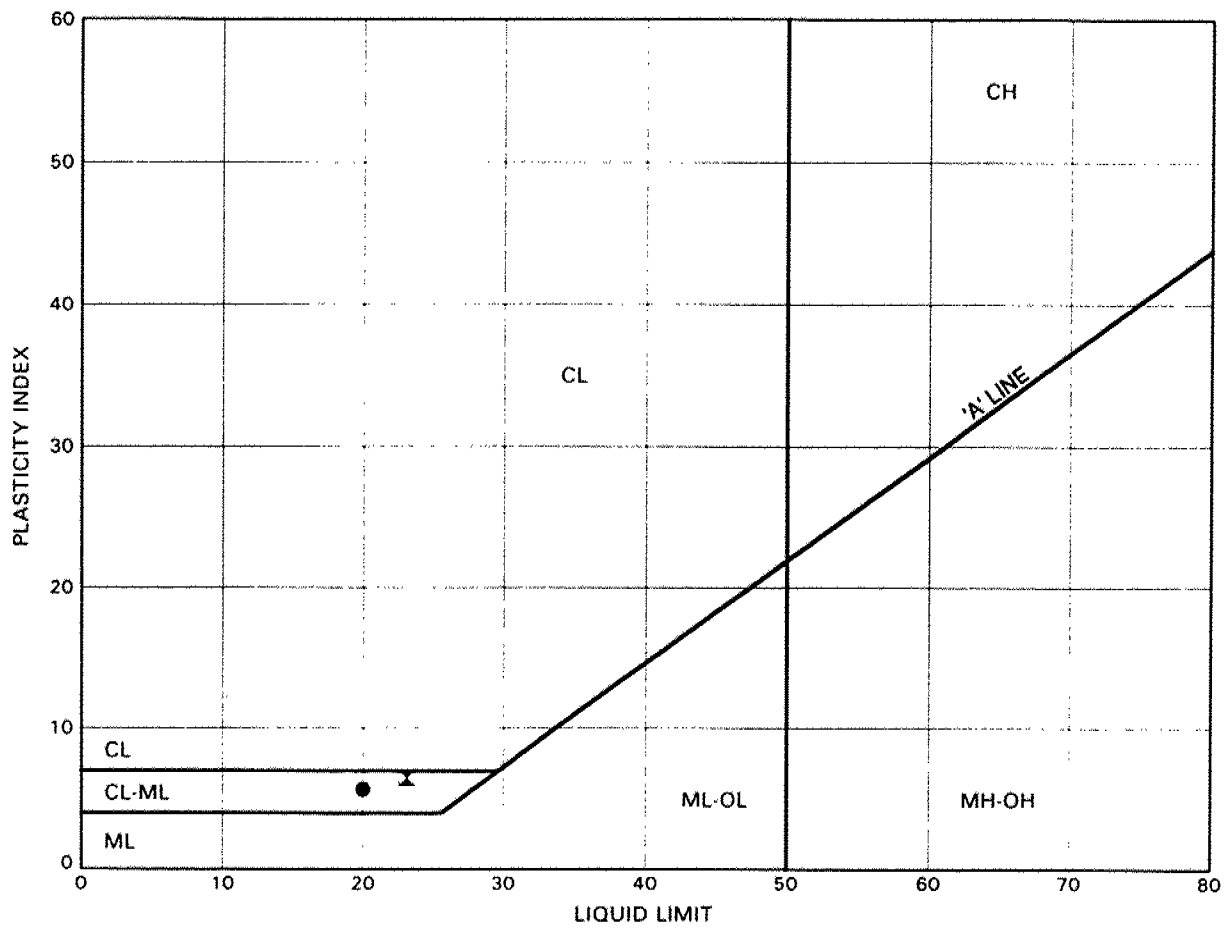
Date October 1998
Project 462-93-00



Prep'd WM
Chkd. AEG

CNR OVERHEAD (NBL)
ATTERBERG LIMITS TEST RESULTS

FIGURE B4



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	E-98-1	6.32	319.08
▲	E-98-5	9.37	316.43

Date October 1998
 Project 462-93-00



Prep'd WM
 Chkd. AEG

Table 1

Results of pH and Sulphate Testing

Sample	Depth (m)	pH	Sulphates (ppm)
E-98-2, SA2	0.8 - 1.2	6.41	47.1
E-98-3, SA3	1.5 - 2.0	5.83	32.6

APPENDIX C

NON-STANDARD SPECIAL PROVISION



NON-STANDARD SPECIAL PROVISION

Sheet 1 of 1
DATE: 1998 08 04

WP No.: 482-83-00 CONTRACT No.: DISTRICT No.: 52 HWY No.: 11
LOCATION: 8.7 km North of Highway 60, Northerly 13.6 km

1. This SP is new

This non-standard special provision outlines the requirements for on-site earth embankment fill Materials

2.

Item	Spec No.	Title or Item Description
		On-Site Earth Fill Requirements

Earth fill materials shall be free from organic material and foreign objects. Boulder content shall conform to OPSS 206.

Earth fill material which has more than 50 percent of the particles smaller than 75 μm , as determined by LS-702, shall not be used if the field moisture content is higher than:

- A. The optimum moisture content, as determined by LS-708, plus 1 percent for soils with a Plasticity index, as determined by LS-704, of 7 percent or less, or
- B. The optimum moisture content, as determined by LS-708, plus 5 percent for soils with a Plasticity index, as determined by LS-704, of more than 7 percent.

Earth material which has more than 50 percent of the particles smaller than 75 μm as determined by LS-702, shall not be used as fill in embankments having side slopes steeper than 2.5H:1V.

Earth fill material with 50 percent or more of the particles between 5 μm and 75 μm in size, as determined by LS-702, shall not be used within 1.8 m of the top of pavement elevation.

3.

Initiated By

Detailed By

Approved By

APPENDIX D

SUGGESTED NON-STANDARD SPECIAL PROVISION



The following is suggested as possible text for a Non Standard Special Provision relating to the monitoring of pile driving at the CNR Overhead, Site 42-10N:

" The Contractor's attention is drawn to the fact that the piles will be driven into a soil layer consisting of compact to very dense sand with cobbles and boulders. The presence of the cobbles and boulders may impede the progress of the driven piles.

If any pile tip does not reach the specified elevation, or if there is reason to believe the pile may be damaged, driving of that pile must be suspended and the driving record reviewed by a geotechnical engineer with experience in piled foundation design and construction. Any further driving of the pile may only be carried out in accordance with the geotechnical engineer's written directive."

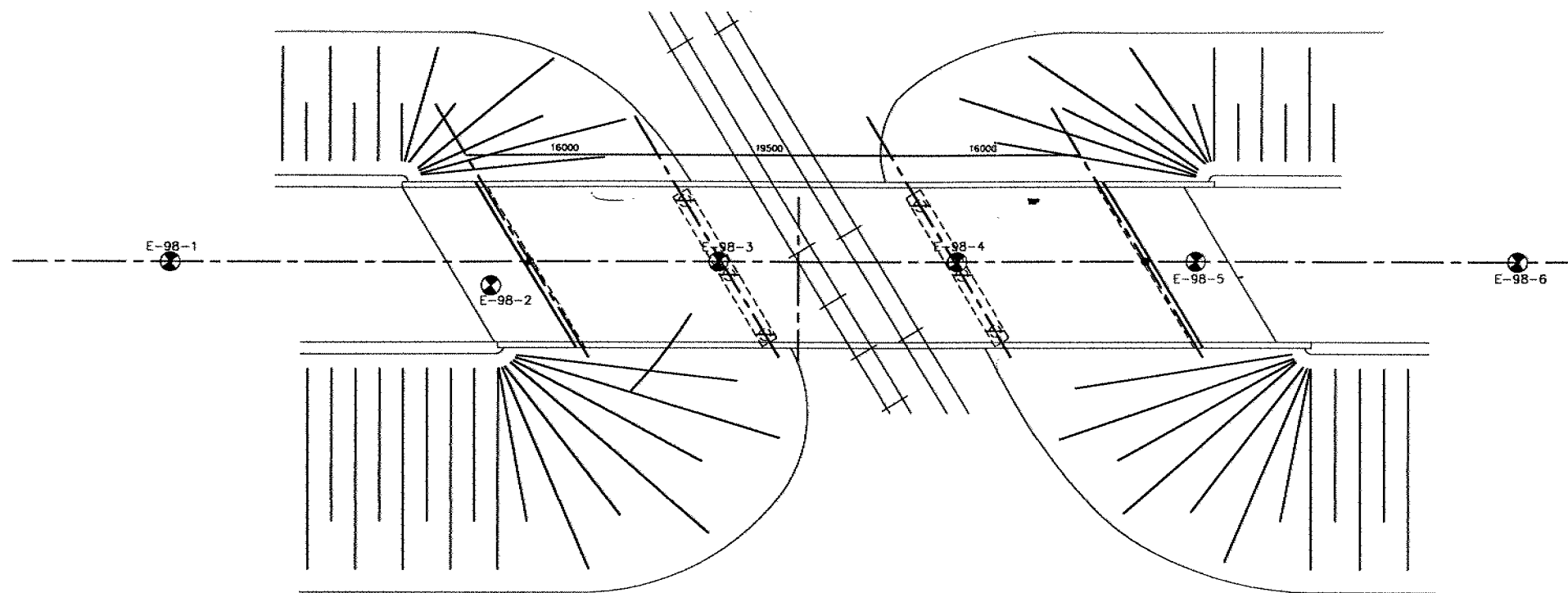


19-1351-7e-01

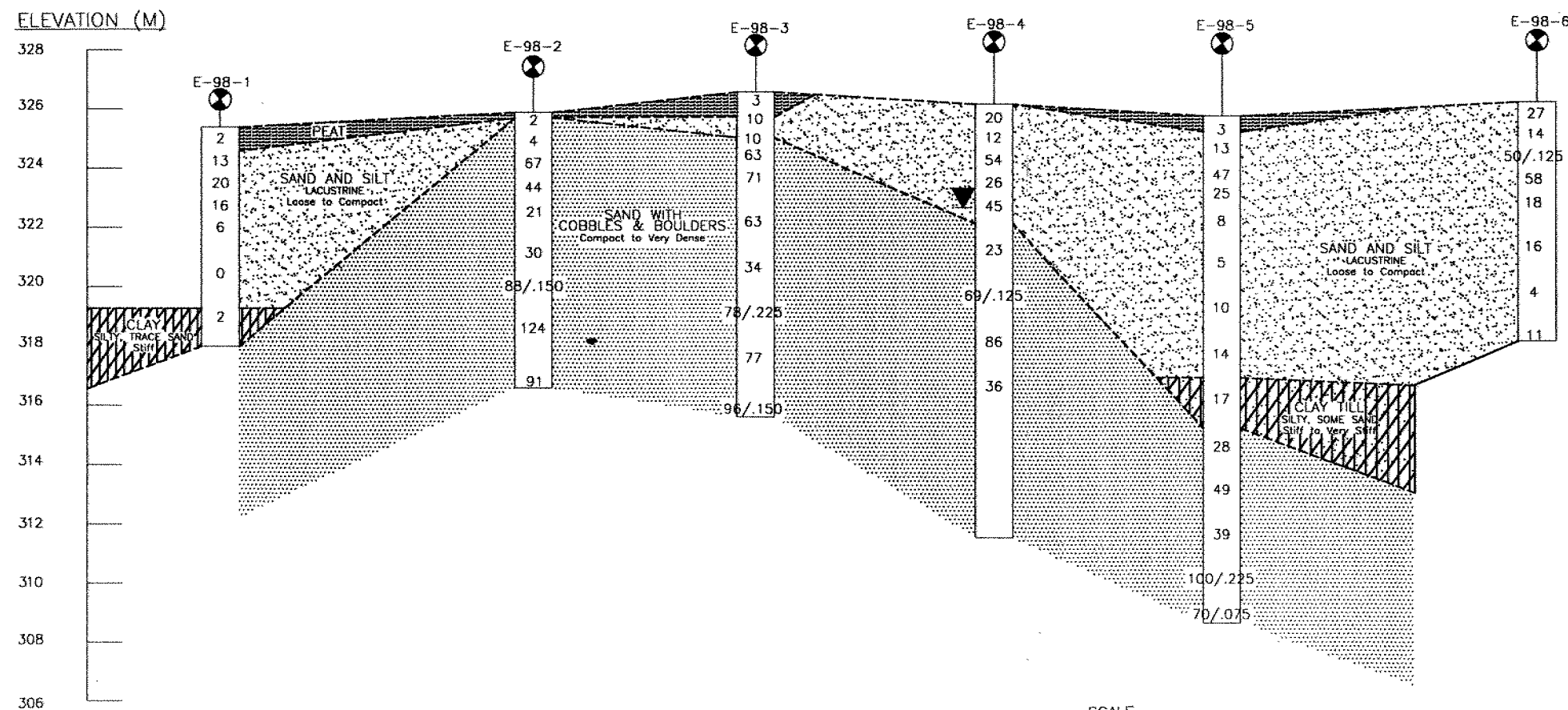
MINISTRY OF TRANSPORTATION, ONTARIO

19-1351-7e-01

19-1351-7e-01



SCALE
1:500



SCALE
V 1:200
H 1:500

DIST 52
CONT No
WP No 458-93-00

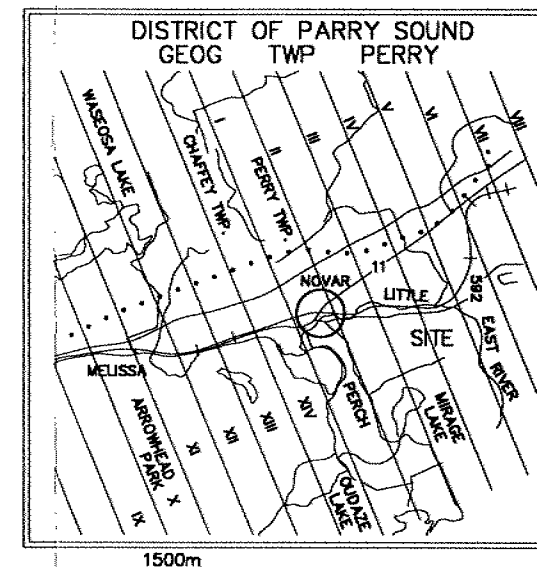
HIGHWAY 11- FOUR LANE
CNR OVERHEAD AT NOVAR



SHEET

MRC MCCORMICK RANKIN
CORPORATION

THURBER ENGINEERING LTD.



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

LEGEND			
	Borehole		
	WL September 08, 1998		
	'N' Blows/0.3m (Std Pen Test)		
No	ELEV.	LOCATION	
		NORTHING	EASTING
E-98-1	325.4	5 035 064.2	324 308.9
E-98-2	325.9	5 035 085.8	324 285.4
E-98-3	326.6	5 035 092.0	324 273.3
E-98-4	326.2	5 035 105.4	324 256.4
E-98-5	325.8	5 035 118.8	324 241.3
E-98-6	326.3	5 035 134.6	324 221.2

19-1351-7e-01

OVERSIZE DRAWING(S)

**FINAL FOUNDATION INVESTIGATION REPORT FOR
CNR OVERHEAD (NBL), NOVAR
HIGHWAY 11 FOUR LANING
6.7 km NORTH OF HWY 60 NORTHERLY 13 km
W.P. 462-93-00, SITE 42-10N
DISTRICT 52, HUNTSVILLE**

Report

to

McCormick Rankin Corporation

Direction of Fieldwork and Engineering Analysis by

Thurber Engineering Ltd.
170 Evans Avenue, Suite 101
Etobicoke, Ontario
M8Z 5Y6

Phone: (416) 503 3600
Fax: (416) 503 3010

October 30, 1998
19-1351-7e

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



Report Reviewed by :

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

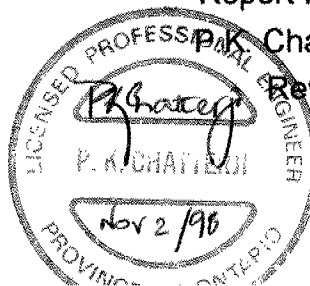


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2.1	Site Location	1
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4.	DESCRIPTION OF SUBSURFACE CONDITIONS	5
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DRAWINGS

19-1351-7E-01 Borehole Locations and Soil Strata

APPENDICES

Appendix A Borehole Logs
Appendix B Laboratory Test Results

**FINAL FOUNDATION INVESTIGATION REPORT FOR
CNR OVERHEAD (NBL), NOVAR
HIGHWAY 11 FOUR LANING
6.7 km NORTH OF HWY 60 NORTHERLY 13 km
W.P. 462-93-00, SITE 42-10N
DISTRICT 52, HUNTSVILLE**

1. INTRODUCTION

This report presents the results of the foundation investigation carried out by Thurber Engineering Ltd. (Thurber) at the site of the CNR Overhead and approach fills on Highway 11 in Novar, Ontario. The purpose of the investigation was to explore the subsurface and groundwater conditions at the site and based on the data obtained provide borehole logs, soil profile and a written description of the subsurface conditions.

Thurber carried out the investigation as a sub-consultant to McCormick Rankin Corporation (MRC) under Ministry of Transportation Ontario (MTO) Agreement 9750-7424-5262.

2. SITE DESCRIPTION

2.1 Site Location

The site forms part of the four-laning of Highway 11 north of Huntsville and is located at the crossing of the new Northbound Lanes of Highway 11 over the CNR tracks at Novar, Ontario. The site lies at Station 11+017, Centreline Northbound Lanes.

The site lies approximately 18 m east of the existing CNR Overhead in Novar and is accessible from local Secondary Highway 592 from the south and Old Muskoka Road from the north.

2.2 Physiography

Based on The Physiography of Southern Ontario, 3rd Edition, by Chapman and Putnam, the region surrounding the site consists of bedrock ridges with shallow overburden. The bedrock is undifferentiated igneous and metamorphic rock of early Precambrian age and is generally hard and massively jointed.

The Highway 11 corridor, however, lies in a long, narrow sand plain within the region of shallow bedrock. The typical soils in the corridor consist of sand and silt, with some gravel deposited as glacial outwash or in localized glaciolacustrine environments.

From the CNR tracks southward, the area is generally flat and the ground surface is covered by topsoil and vegetal cover, including scrubby trees. To the north of the tracks, the ground rises somewhat and an existing pit reveals sand and gravel soils. The existing Highway 11 passes through a cut immediately north of the site and the soils exposed in the face of the cut are also sand and gravel.

Visual inspection of the local area suggests poor drainage and a high water table.

3. INVESTIGATION PROCEDURES

3.1 Field investigation

Between January 16 and February 17, 1998, track mounted drill rigs were used on site for drilling, Standard Penetration Testing (SPT) (following the procedure outlined in ASTM D 1586) and dynamic cone penetration testing. One sampled borehole was drilled at each of the two pier and two abutment locations, and at each approach fill, giving a total of six

boreholes. The approximate borehole locations are shown on Drawing 19-1351-7e-01.

The drilling and sampling at each borehole was started using continuous flight hollow stem auger equipment. In several holes the progress of drilling by auger was impeded by the presence of numerous cobbles and boulders in the soil and eventually effective refusal to further auger penetration was encountered. When this occurred, the drilling method was switched to mud rotary which generally obtained further penetration. In some cases, when the drilling encountered a boulder and became too arduous, the borehole was relocated a short distance and drilling recommenced from that location.

The boreholes were numbered E-98-1 through E-98-6. The depths of sampling in the six boreholes were as follows:

Borehole No.	Depth of Sampling (m)
E-98-1	7.6
E-98-2	9.3
E-98-3	11.0
E-98-4	14.6
E-98-5	17.1
E-98-6	8.1

Standard Penetration Tests (SPT) were carried out in all boreholes to assess the relative density of the soils in place and to obtain soil samples for the purposes of identification and laboratory testing. SPTs were conducted at intervals of 0.75 m in the upper 3.0 m and generally at intervals of 1.5 m thereafter. In some instances, sampling was not possible as proposed and the intervals were varied to suit the bouldery soil conditions.

Dynamic cone penetration tests, used to supplement the SPT data, were conducted at selected boreholes as follows:

Borehole No.	Depth of Dynamic Cone Test
E-98-1	Ground surface to 6.1 m, adjacent to the sampled borehole
E-98-4	At intervals between depths of 12.3 and 14.3 m in the bottom of the sampled borehole

Boreholes were not left open long enough for the groundwater to stabilize and this coupled with the constant addition of drilling fluid led to the decision not to report data on groundwater levels in open boreholes on completion of drilling. On completion of drilling and sampling and based on the relatively flat surface and permeable soil conditions across the area of interest, one standpipe piezometer was installed in Borehole E-98-4 to monitor the groundwater level.

The boreholes were backfilled with drill cuttings except in Borehole E-98-4 where sand pack was installed around the piezometer tip and a bentonite plug was installed near the top of the hole.

All recovered samples were examined, identified and logged in the field and retained in the care of the field supervisor. The samples were later transported to Thurber's Toronto laboratory by the field supervisor for further examination and laboratory analysis.

The results of the drilling, sampling and in-situ testing are summarized on the borehole logs in Appendix A.

3.2 Laboratory Analysis

The geotechnical laboratory testing included natural moisture content determinations and visual classifications on all recovered samples. In addition, grain size analysis, Atterberg limits, pH and sulphate testing were conducted on selected samples. The results of the laboratory testing are presented on the borehole logs in Appendix A and in Figures B1 to B4 and Table 1 in Appendix B.

It should be noted that the grain size analyses were conducted on samples retrieved in the split spoon sampler and therefore exclude all material larger than approximately 35 mm. While the results are useful in assessing the composition of the soil matrix they should not be regarded as representative of the total grain size range of the soil in-situ. In particular, the conditions noted during drilling make it clear that there are numerous cobbles and boulders in the soil deposits encountered on this site.

4. DESCRIPTION OF SUBSURFACE CONDITIONS

4.1 Subsurface Soil Conditions

Detailed descriptions of the individual strata encountered in the boreholes are presented on the borehole logs in Appendix A. The stratigraphic profile inferred from the borehole information is shown on Drawing No. 19-1351-7e-01.

Examination of the profile shows that the basal stratigraphic unit is a buried mound of compact to very dense sand with cobbles and boulders. The mound peaks at or close to the ground surface (Elevation 324.5) in the area of Boreholes E-98-2 and E-98-3 (the south abutment and south pier) and slopes down to the north and to the south. The presence of similar soils was encountered in the investigation carried out for the design of the foundations of the existing Highway 11 structure immediately to the west.

Within the zone of exploration and at lower levels (below Elevation 319.3 and 317.0), the flanks of the buried mound of sand with cobbles and boulders are overlain by cohesive soils. The soil encountered on the south flank appeared layered, suggesting a lacustrine origin, while that on the north flank was more heterogeneous. These soils were encountered in only one sample in each of two boreholes and it is difficult to determine if they represent one stratum present on both flanks of the mound or are the result of different depositional events.

The mound of coarse granular soil and the cohesive soils described above are overlain by sand and silt with a layered appearance, suggesting lacustrine origin. These soils are generally loose to compact and extend up to the base of the shallow surficial layer of peat, except in Borehole E-98-2 which shows the mound of coarse granular soils extending essentially to ground surface.

Descriptions of the general soil strata encountered are as follows.

Peat

The surficial peat deposit consisted of a highly organic, fibrous peat with numerous roots. This deposit is generally loose, ranges from dry to wet and has colours ranging from brown to black. The water content of the peat deposit ranged from 25 to 58%. The peat ranged in thickness from 0.4 to 0.8 m, but was absent at Boreholes E-98-4 and E-98-6, probably as a result of past construction or related activities.

Sand with Cobbles and Boulders (Glacial Outwash Soils)

These soils form the buried mound described above and centred about Boreholes E-98-2 and E-98-3.

The deposit consists of a sand matrix with varying proportions of silt, and gravel and containing numerous cobbles and boulders. The deposit has been classified as compact to very dense on the basis of SPT N-values

ranging from 21 to values well in excess of 100, for 0.3 m penetration. Two individual SPT values of 4 and 10 were recorded at the top of the deposit. It is recognized that some of the very high SPT N-values may be due to the presence of cobbles and boulders. The natural moisture contents measured for this soil ranged from 6 to 22%

This stratum was not positively identified in Borehole E-98-1, but effective refusal to further sampling was encountered at Elevation 317.8, possibly indicating that the top of the stratum had been reached.

The base of the deposit was not encountered in any of the boreholes, but the results of Borehole E-98-5 show that it extends to Elevation 308.7 or deeper. The top of the deposit was encountered at elevations ranging from 325.5 at Borehole E-98-2 to 315.3 at Borehole E-98-5.

Heterogeneous Mixture of Clay, Sand and Silt (Glacial Till)

A layer of cohesive soil, interpreted to be less than 2 m thick overlies the lower levels of the north flank of the coarse granular soils. The top of this deposit is noted at Elevation 317.0 in Borehole E-98-5. The soil is a clay, silty, some sand and has a heterogeneous structure suggesting glacial till. The SPT N-value measured in this layer was 17 blows for 0.3 m of penetration. The natural moisture content was 28%.

Lacustrine Sand and Silt

The main soil type overlying the flanks of the buried mound of coarse granular soil was a deposit of sand and silt. This deposit was layered, including some thin clayey seams suggesting a lacustrine origin.

The relative density of the deposit is classified as loose to compact, based

on SPT N-values ranging from 4 to 27 blows for 0.3 m of penetration. Some higher values were recorded on the north half of the site at depths of approximately 2.0 m and these are attributed to the presence of scattered cobbles and boulders. The measured natural moisture contents ranged from 13 to 31%.

This deposit extended from the ground surface, or the underside of the peat, to depths of 6.1 m (Elevation 319.3) at Borehole E-98-1 and 8.8 m (Elevation 317.0) at Borehole E-98-5.

Lacustrine Clay

A layer of lacustrine clay was encountered only in Borehole E-98-1 at Elevation 319.3. The borehole was terminated on effective refusal to sampling at Elevation 317.8, giving a thickness of clay of 1.6 m. In-situ test results indicate that the clay is stiff and of low plasticity. A water content of 30% was measured in this layer. Results of gradation and Atterberg Limit tests on a selected sample of this layer are presented in Figure B1 in Appendix B.

4.2 Groundwater

Water levels in the open boreholes ranged from 1.8 m (BH E-98-5) to 2.2 m (BH E-98-1) below ground surface.

The following groundwater levels were recorded in the piezometer installed in Borehole E-98-4:

Date	Depth to Water (below existing ground surface)
Feb 12, 1998	5.8 m
May 24, 1998	3.5 m
July 31, 1998	3.4 m
September 8, 1998	3.5 m

Based on this data, the existing groundwater level lies at Elevation 322.7. This value is based on short term readings and may fluctuate throughout the year, in particular rising after the spring thaw.

STATEMENT OF GENERAL CONDITIONS

1. STANDARD OF CARE

This study and Report have been prepared in accordance with generally accepted engineering or environmental consulting practices in this area. No other warranty, expressed or implied, is made.

2. COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report which is of a summary nature and is not intended to stand alone without reference to the instructions given to us by the Client, communications between us and the Client, and to any other reports, writings, proposals or documents prepared by us for the Client relative to the specific site described herein, all of which constitute the Report.

IN ORDER TO PROPERLY UNDERSTAND THE SUGGESTIONS, RECOMMENDATIONS AND OPINIONS EXPRESSED HEREIN, REFERENCE MUST BE MADE TO THE WHOLE OF THE REPORT. WE CANNOT BE RESPONSIBLE FOR USE BY ANY PARTY OF PORTIONS OF THE REPORT WITHOUT REFERENCE TO THE WHOLE REPORT.

3. BASIS OF REPORT

The Report has been prepared for the specific site, development, design objectives and purpose that were described to us by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the document are only valid to the extent that there has been no material alteration to or variation from any of the said descriptions provided to us unless we are specifically requested by the Client to review and revise the Report in light of such alteration or variation.

4. USE OF THE REPORT

The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client. NO OTHER PARTY MAY USE OR RELY UPON THE REPORT OR ANY PORTION THEREOF WITHOUT OUR WRITTEN CONSENT. WE WILL CONSENT TO ANY REASONABLE REQUEST BY THE CLIENT TO APPROVE THE USE OF THIS REPORT BY OTHER PARTIES AS "APPROVED USERS". The contents of the Report remain our copyright property and we authorize only the Client and Approved Users to make copies of the Report only in such quantities as are reasonably necessary for the use of the Report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make the Report, or any portion thereof, available to any party without our written permission. Any use which a third party makes of the Report, or any portion of the Report, are the sole responsibility of such third parties. We accept no responsibility for damages suffered by any third party resulting from unauthorized use of the Report.

5. INTERPRETATION OF THE REPORT

- a) Nature and Exactness of Soil and Contaminant Description: Classification and identification of soils, rocks, geological units, contaminant materials and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgemental in nature and even comprehensive sampling and testing programs, implemented with the appropriate equipment by experienced personnel, may fail to locate some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and all persons making use of such documents or records should be aware of, and accept, this risk. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. Where special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.

(see over...)

INTERPRETATION OF THE REPORT *(continued)*

- b) Reliance on Provided Information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to us. We have relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, we cannot accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of misstatements, omissions, misrepresentations, or fraudulent acts of persons providing information.

6. RISK LIMITATION

Geotechnical engineering and environmental consulting projects often have the potential to encounter pollutants or hazardous substances and the potential to cause an accidental release of those substances. In consideration of the provision of the services by us, which are for the Client's benefit, the Client agrees to hold harmless and to indemnify and defend us and our directors, officers, servants, agents, employees, workmen and contractors (hereinafter referred to as the "Company") from and against any and all claims, losses, damages, demands, disputes, liability and legal investigative costs of defence, whether for personal injury including death, or any other loss whatsoever, regardless of any action or omission on the part of the Company, that result from an accidental release of pollutants or hazardous substances occurring as a result of carrying out this Project. This indemnification shall extend to all Claims brought or threatened against the Company under any federal or provincial statute as a result of conducting work on this Project. In addition to the above indemnification, the Client further agrees not to bring any claims against the Company in connection with any of the aforementioned causes.

7. SERVICES OF SUBCONSULTANTS AND CONTRACTORS

The conduct of engineering and environmental studies frequently requires hiring the services of individuals and companies with special expertise and/or services which we do not provide. We may arrange the hiring of these services as a convenience to our Clients. As these services are for the Clients' benefit, the Client agrees to hold the Company harmless and to indemnify and defend us from and against all claims arising through such hirings to the extent that the Client would incur had he hired those services directly. This includes responsibility for payment for services rendered and pursuit of damages for errors, omissions or negligence by those parties in carrying out their work. In particular, these conditions apply to the use of drilling, excavation and laboratory testing services.

8. CONTROL OF WORK AND JOBSITE SAFETY

We are responsible only for the activities of our employees on the jobsite. The presence of our personnel on the site shall not be construed in any way to relieve the Client or any contractors on site from their responsibilities for site safety. The Client acknowledges that he, his representatives, contractors or others retain control of the site and that we never occupy a position of control of the site. The Client undertakes to inform us of all hazardous conditions, or other relevant conditions of which the Client is aware. The Client also recognizes that our activities may uncover previously unknown hazardous conditions or materials and that such a discovery may result in the necessity to undertake emergency procedures to protect our employees as well as the public at large and the environment in general. These procedures may well involve additional costs outside of any budgets previously agreed to. The Client agrees to pay us for any expenses incurred as the result of such discoveries and to compensate us through payment of additional fees and expenses for time spent by us to deal with the consequences of such discoveries. The Client also acknowledges that in some cases the discovery of hazardous conditions and materials will require that certain regulatory bodies be informed and the Client agrees that notification to such bodies by us will not be a cause of action or dispute.

9. INDEPENDENT JUDGEMENTS OF CLIENT

The information, interpretations and conclusions in the Report are based on our interpretation of conditions revealed through limited investigation conducted within a defined scope of services. We cannot accept responsibility for independent conclusions, interpretations, interpolations and/or decisions of the Client, or others who may come into possession of the Report, or any part thereof, which may be based on information contained in the Report. This restriction of liability includes decisions made to either purchase or sell land.

APPENDIX A

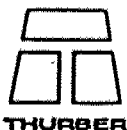
BOREHOLE LOGS

- Symbols and Terms Used on Borehole Logs

- Unified Soil Classification

- Borehole Logs E-98-1 to E-98-6





BOREHOLE GRAPHIC SYMBOLS

SOILS



FILL

ORGANICS

CLAY

SILT

SAND

GRAVEL

COBBLES



SILTY CLAY

CLAYEY SILT

SILTY SAND

SAND & GRAVEL

CLAYEY SILT TILL

SILTY CLAY TILL

SANDY SILT TILL

ROCK



SHALE

LIMESTONE



SILTSTONE

GRANITE

OTHER



CEMENT GROUT

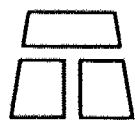
BENTONITE GROUT



CONCRETE

WATER

BENTONITE SEAL



THURBER

SYMBOLS AND TERMS USED ON TEST HOLE LOGS

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	Less than 10	Less than 2
Soft	10 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 TEST HOLE LOGS

SYMBOLS FOR SAMPLE TYPE	 Shelby Tube	A - Casing
	 SPT	 Grab/Auger sample
	 No Recovery	 Core

● MC - Moisture Content (% by Weight) as determined by sample]

⏊ Water Level

C_{vane} Shear Strength Determination by Field Insitu Vane

C_{pen} Shear Strength Determination by Pocket Penetrometer

C_{lab} Shear Strength Determination using a Laboratory Vane Apparatus

C_u Undrained Shear Strength determined by Unconfined Compression Test

- (1) SPT Standard Penetration Test - refers to the number the blows from a 63.5kg hammer falling through 0.76m to advance a 60 degree truncated cone 0.3m.

UNIFIED SOILS CLASSIFICATION

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

RECORD OF BOREHOLE No E-98-1

1 OF 1

METRIC

W.P. 462-93-00 LOCATION CNR OVERHEAD (NBL), N 5 035 064.2 E 324 308.9 ORIGINATED BY EK
DIST 52 HWY 11 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY WM
DATUM GEODETIC DATE 98.01.16 - 98.01.16 CHECKED BY AG





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	20 40 60 80 100	20 40 60 80 100		
325.4	0.0 PEAT, brown, moist to wet: (PT)		1	SS	2		325					
324.6	0.8 SAND and SILT, fine grained, occasional seams of wet coarse sand, occasional seams of wet oxidized sand, compact, brown, moist to wet: (ML/SP)		2	SS	13		324					
323.9	1.5 SILT, sandy, occasional layers of coarse oxidized sand, compact to very loose, grey: (ML-NONPLASTIC)		3	SS	20		323					
			4	SS	16		322					
321.7	3.7 trace to some clay		5	SS	6		321					0 23 64 13
			6	SS	0		320					
319.3	6.1 CLAY, silty, to SILT, clayey, trace sand, soft, slightly varved, grey: (LACUSTRINE)(CL-ML)		7	SS	2		319					0 6 61 33
317.8	7.6 some gravelly sand in last sample auger refusal on boulder END OF BOREHOLE AT 7.65m. WATER LEVEL AT 2.20m. BOREHOLE OPEN TO 2.74m. BOREHOLE BACKFILLED WITH DRILL CUTTINGS. Notes: 1) Moved ahead 1.5m and drove dynamic cone. 2) Moved ahead further 1.5m and probed by auger. Refusal at 5.5m depth.		8	SS	31/ .025		318					

RECORD OF BOREHOLE No E-98-2

1 OF 1

METRIC

W.P. 462-93-00 LOCATION CNR OVERHEAD (NBL), N 5 035 085.8 E 324 285.4 ORIGINATED BY EK
DIST 52 HWY 11 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS/MUD ROTARY COMPILED BY WM
DATUM GEODETIC DATE 98.01.28 - 98.01.28 CHECKED BY AG

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 (%)							
							PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT w _p w w _L							
325.9														
0.0														
325.5	PEAT, trace sand, occasional rootlets, brown		1	SS	2									
0.4	SAND, trace gravel, trace silt, loose at top then dense to very dense, light brown: (SP)		2	SS	4		325							
			3	SS	67		324							0 70 24 6
	grinding at 2.15 to 2.29m		4	SS	44		323							
	grinding at 2.44m possible boulders		5	SS	21									2 93 5
	compact below 3.0m													
322.1							322							
3.8	trace to some silt, with boulders grinding at 3.81m		6	SS	30		321							
			7	SS	88/ .150		320							
	grinding at 6.10 to 7.16m, possible boulders, cobbles gravelly, trace silt						319							
	augered to 7.16m, augers drifting off vertical, auger refusal													
318.0			8	SS	124		318							31 53 17
7.9	coarse grained, gravelly, occasional layers of light sandy silt, very dense, brown						317							
316.6			9	SS	91									
9.3	grinding from 9.30 to 9.51m.													
	END OF BOREHOLE AT 9.30m DUE TO REFUSAL ON BOULDER.													
	Note: After auger refusal at 7.6m, moved BH ahead and drilled by mud rotary to 7.62m to re-start sampling. Boulders encountered from 1.5 to 6.9m.													

RECORD OF BOREHOLE No E-98-3

1 OF 1

METRIC

W.P. 462-93-00 LOCATION CNR OVERHEAD (CNRI), N 5 035 092.0 E 324 273.3 ORIGINATED BY EK
DIST 52 HWY 11 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS/MUD ROTARY COMPILED BY WM
DATUM GEODETIC DATE 98.02.05 - 98.02.05 CHECKED BY AG

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			20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
326.6 0.0	PEAT, silty, occasional rootlets, occasional black staining, very loose, brown, dry to moist: (PT)		1	SS	3		326					
325.8 0.8	SILT, some sand to sandy, occasional rootlets, occasional black staining, compact, brown: (ML-NONPLASTIC)		2	SS	10		325					
325.1 1.5	SAND, silty, gravelly, occasional rootlets, occasional black layers, compact to very dense, brown: (SP) grinding at 2.15m, possible cobble, boulders grinding at 3.35m		3	SS	10		325					
322.9 3.7	SAND, silty, trace gravel, occasional grey silt inclusions, very dense, brown: (SM) grinding at 4.14m. set up for rotary drill		4	SS	63		324					
			5	SS	71		323					
320.8 5.8	SAND and GRAVEL, trace to some silt, very dense, brown: (SP)		6	SS	63		322					
			7	SS	34		321					
			8	SS	78/ 225		320					
			9	SS	77		319					
			10	SS	96/ 150		318					
315.6 11.0	END OF BOREHOLE AT 10.97m. Note: Effective auger refusal at 4.1m. Changed to mud rotary drilling.						317					
							316					

RECORD OF BOREHOLE No E-98-4

1 OF 2

METRIC

W.P. 462-93-00 LOCATION CNR OVERHEAD (NBL), N 5 035 105.4 E 324 256.4 ORIGINATED BY EK
 DIST 52 HWY 11 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY WM
 DATUM GEODETIC DATE 97.02.10 - 97.02.11 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	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
326.2 0.0	SILT, sandy, trace gravel, occasional organic partings, compact, grey to brown slight grinding from 0.45 to 0.76m, possible gravel or cobbles grinding at 1.22m		1	SS	20		326					0 14 70 17
			2	SS	12		325					
324.4			3	SS	54		324					
1.8	SILT, some fine sand, trace gravel, trace clay, layers of silt, fine sand, clay, occasional layers of oxidation, occasional oxide inclusions, laminated, compact to dense, grey to brown: (ML-NONPLASTIC)		4	SS	26		323					41 55 4
			5	SS	45		322					
322.2			6	SS	23		321					
4.0	SAND and GRAVEL, trace silt, compact grinding at 5.03 to 5.49m, possible gravel, cobbles, boulders		7	SS	69/ .125		320					
320.4			8	SS	86		319					
5.8			9	SS	36		318					
	hit boulder at 5.49m SAND, trace to some silt, trace gravel, very dense, brown: (SP) grinding at 6.4m start tricone possible boulder or cobbles substantial loss of mud at 11.28m						317					
							316					
							315					
							314					
							313					
							312					
311.6 14.6	END OF BOREHOLE AT 14.63m.											

Continued Next Page

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

RECORD OF BOREHOLE No E-98-4

2 OF 2

METRIC

W.P. 462-93-00 LOCATION CNR OVERHEAD (NBL), N 5 035 105.4 E 324 256.4 ORIGINATED BY EK
 DIST 52 HWY 11 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY WM
 DATUM GEODETIC DATE 97.02.10 - 97.02.11 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			20	40	60	80	100					
	CASING GETTING HUNG UP AT 6.10m. PULLED RODS, 3.0M ROD BENT WITH THREADS GONE. TRI-CONE BIT ALSO MISSING. WATER LEVEL READINGS: DATE DEPTH TO W.L. (m) 12/02/98 5.8 24/05/98 3.5 31/07/98 3.4 08/09/98 3.5																

RECORD OF BOREHOLE No E-98-5

1 OF 2

METRIC

W.P. 462-93-00 LOCATION CNR OVERHEAD (NBL), N 5 035 118.8 E 324 241.3 ORIGINATED BY EK
DIST 52 HWY 11 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY WM
DATUM GEODETC DATE 97.02.12 - 97.02.13 CHECKED BY AEG

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			20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
325.8 0.0	PEAT, silty, trace sand, occasional rootlets, very loose, brown		1	SS	3		325					
325.3 0.5	SAND, fine grained, some silt, trace gravel, occasional rootlets, occasional oxide staining, compact becoming dense, brown		2	SS	13		324					4 82 14
323.6 2.2	SILT, trace clay, trace fine sand, occasional fissures of oxide, compact to loose, grey: (ML)		3	SS	47		323					
			4	SS	25		322					
			5	SS	8		321					
321.2 4.6	SAND, silty, occasional silt layers, loose to compact, grey		6	SS	5		320					
			7	SS	10		319					0 79 21
318.2 7.6	SAND, gravelly, trace silt, compact, brown		8	SS	14		318					
317.0 8.8	CLAY, SILT, SAND, heterogeneous mixture, stiff to very stiff: (TILL)(CL-ML)		9	SS	17		317					3 12 57 27
315.3 10.5	SAND, silty, with gravel, compact to dense, grey grinding at 10.67m		10	SS	28		316					
			11	SS	49		315					
313.0 12.8	SAND, silty, some gravel, trace clay, dense, grey grinding at 13.72m. possible cobbles and boulders becoming very dense		12	SS	39		314					8 66 20 6
							313					
							312					
							311					

Continued Next Page

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

RECORD OF BOREHOLE No E-98-5

2 OF 2

METRIC

W.P. 462-93-00 LOCATION CNR OVERHEAD (NBL), N 5 035 118.8 E 324 241.3 ORIGINATED BY EK
DIST 52 HWY 11 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY WM
DATUM GEODETIC DATE 97.02.12 - 97.02.13 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT Y 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 (%) 10 20 30						
								PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p W W _L						
308.7	grinding/hard drilling at 15.85m, on top of boulder at 16.15m. grinding from 16.15 to 16.76m		13	SS	100/ .225		310							
309			14	SS	70/ .075		309							
17.1	END OF BOREHOLE AT 17.1m. WATER LEVEL AT 1.8m BELOW EXISTING GRADE.													

RECORD OF BOREHOLE No E-98-6

1 OF 1

METRIC

W.P. 462-93-00 LOCATION CNR OVERHEAD (NBL), N 5 035 134.6 E 324 221.2 ORIGINATED BY EK
DIST 52 HWY 11 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY WM
DATUM GEODETIC DATE 98.02.17 - 98.02.17 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	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100	20 40 60 80 100		
326.3 0.0	SAND, gravelly, trace to some silt, occasional layers of grey silt, compact to very dense, brown		1	SS	27		326							20 68 12
			2	SS	14		325							
			3	SS	59/ 125		324							
			4	SS	58		323							
			5	SS	18		322							
321.8 4.5	SAND and SILT, occasional layers of grey silt, loose to compact, slightly layered: (NP)		6	SS	16		321							15 80 5
			7	SS	4		320							
			8	SS	11		319							
318.2 8.1	END OF BOREHOLE AT 8.08m.													

APPENDIX B

LABORATORY TEST RESULTS

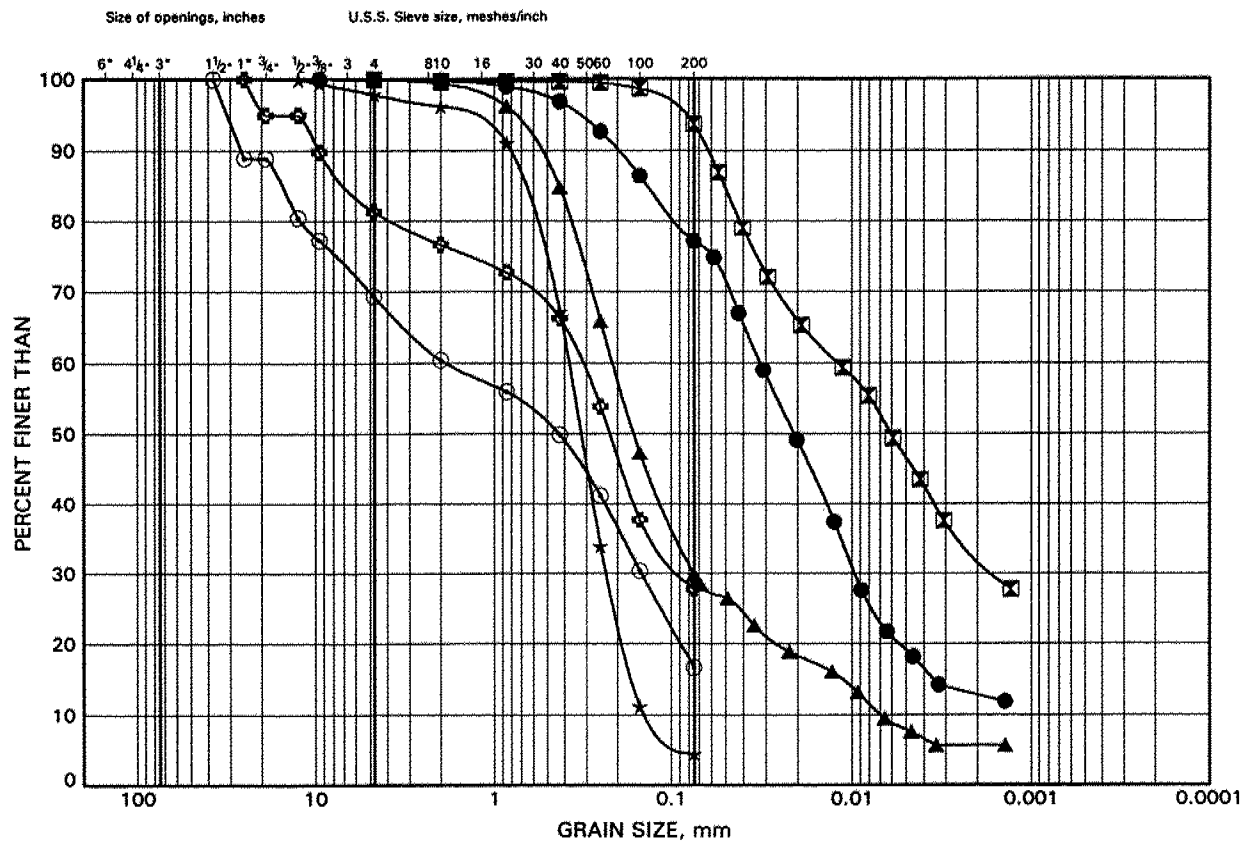
- Figures B1 to B3 - Grain Size analyses

- FIGURE B4 - Plasticity Chart

- Table 1 - pH and Sulphate

CNR OVERHEAD (NBL) GRAIN SIZE DISTRIBUTION

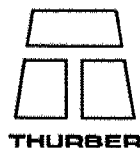
FIGURE B1



SYMBOL BOREHOLE DEPTH (m) ELEVATION (m)

●	E-98-1	3.28	322.12
⊠	E-98-1	6.32	319.08
▲	E-98-2	1.75	324.15
★	E-98-2	3.28	322.62
⊙	E-98-2	7.62	318.28
⊛	E-98-3	2.51	324.09

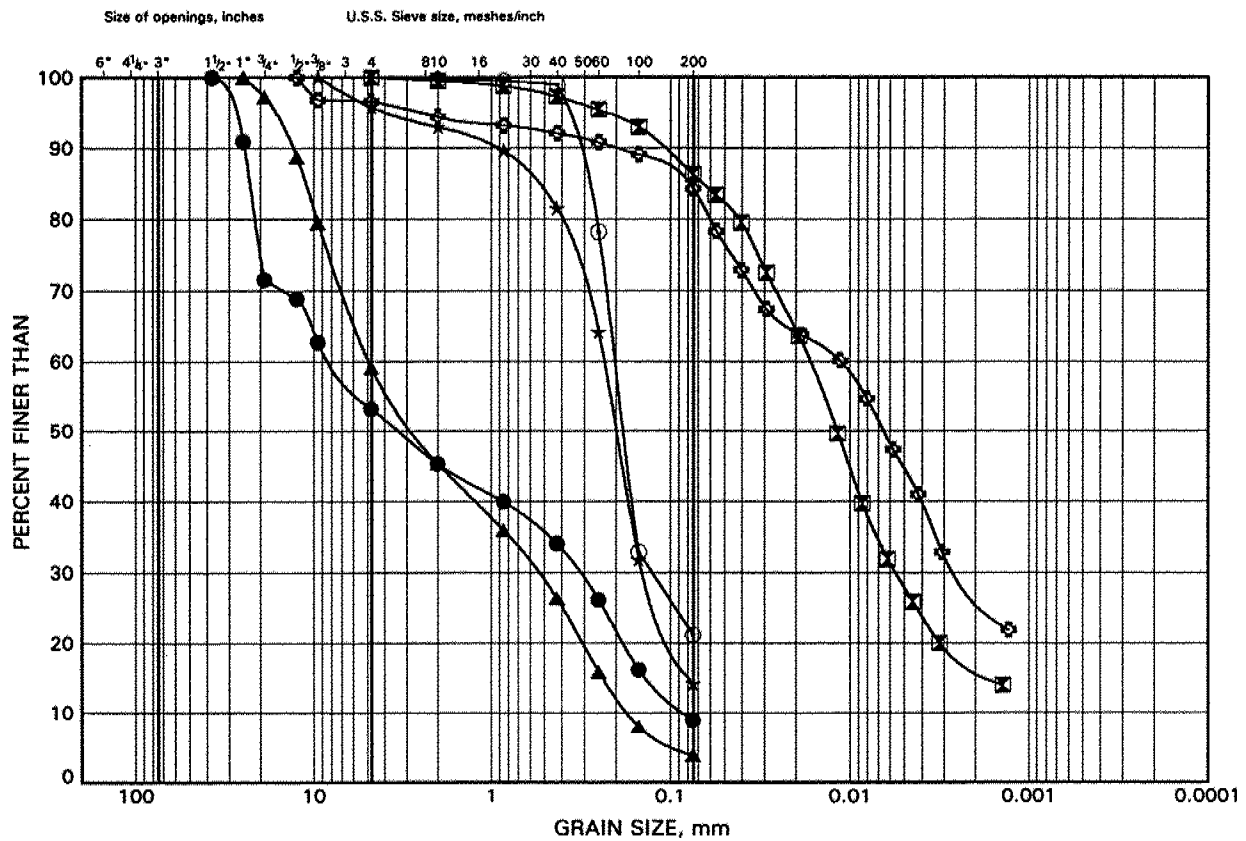
Date October 1998
Project 462-93-00



Prep'd WM
Chkd. AEG

CNR OVERHEAD (NBL) GRAIN SIZE DISTRIBUTION

FIGURE B2

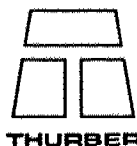


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	E-98-3	9.14	317.46
⊠	E-98-4	1.75	324.45
▲	E-98-4	4.80	321.40
★	E-98-5	1.75	324.05
⊙	E-98-5	6.32	319.48
⊕	E-98-5	9.37	316.43

Date October 1998

Project 462-93-00

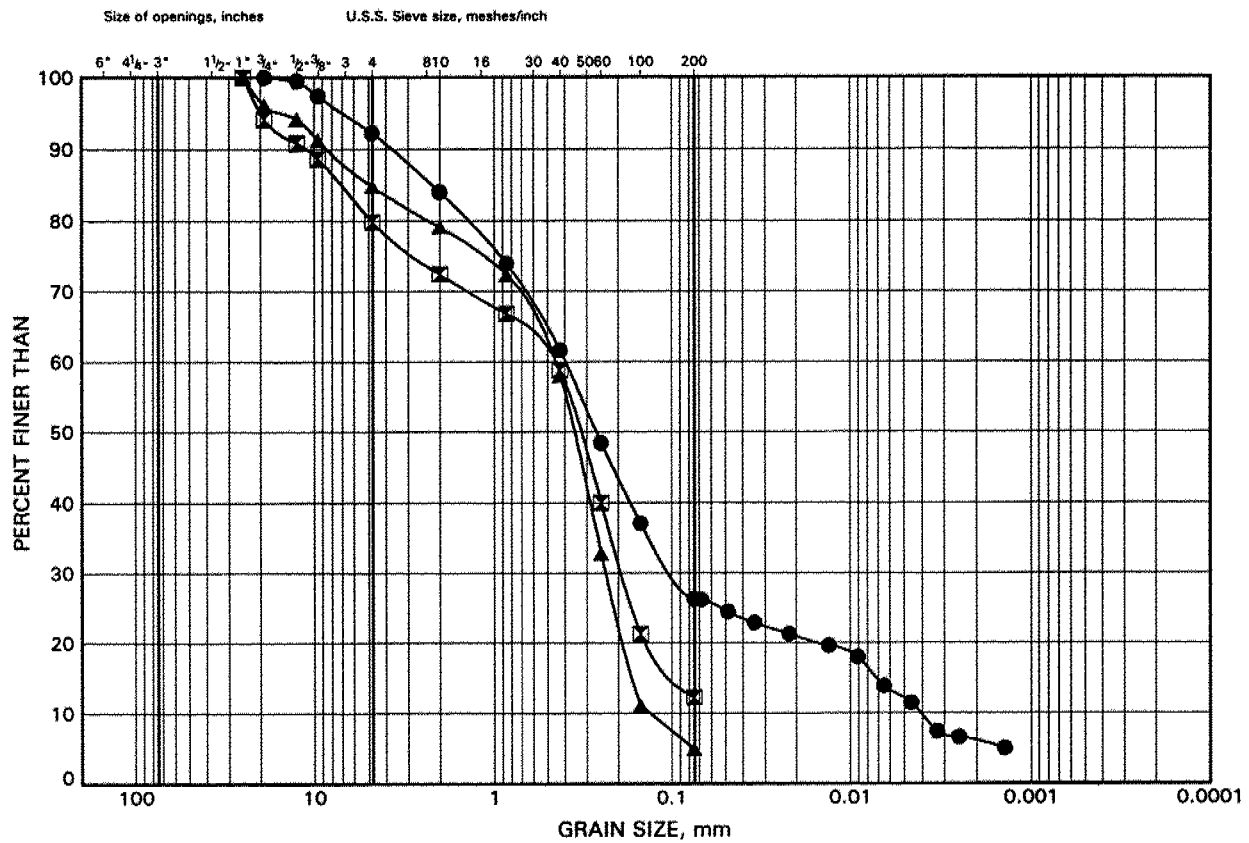


Prep'd WM

Chkd. AEG

CNR OVERHEAD (NBL) GRAIN SIZE DISTRIBUTION

FIGURE B3



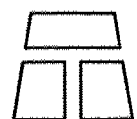
COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL BOREHOLE DEPTH (m) ELEVATION (m)

●	E-98-5	13.94	311.86
×	E-98-6	1.74	324.56
▲	E-98-6	3.28	323.02

Date October 1998

Project 462-93-00



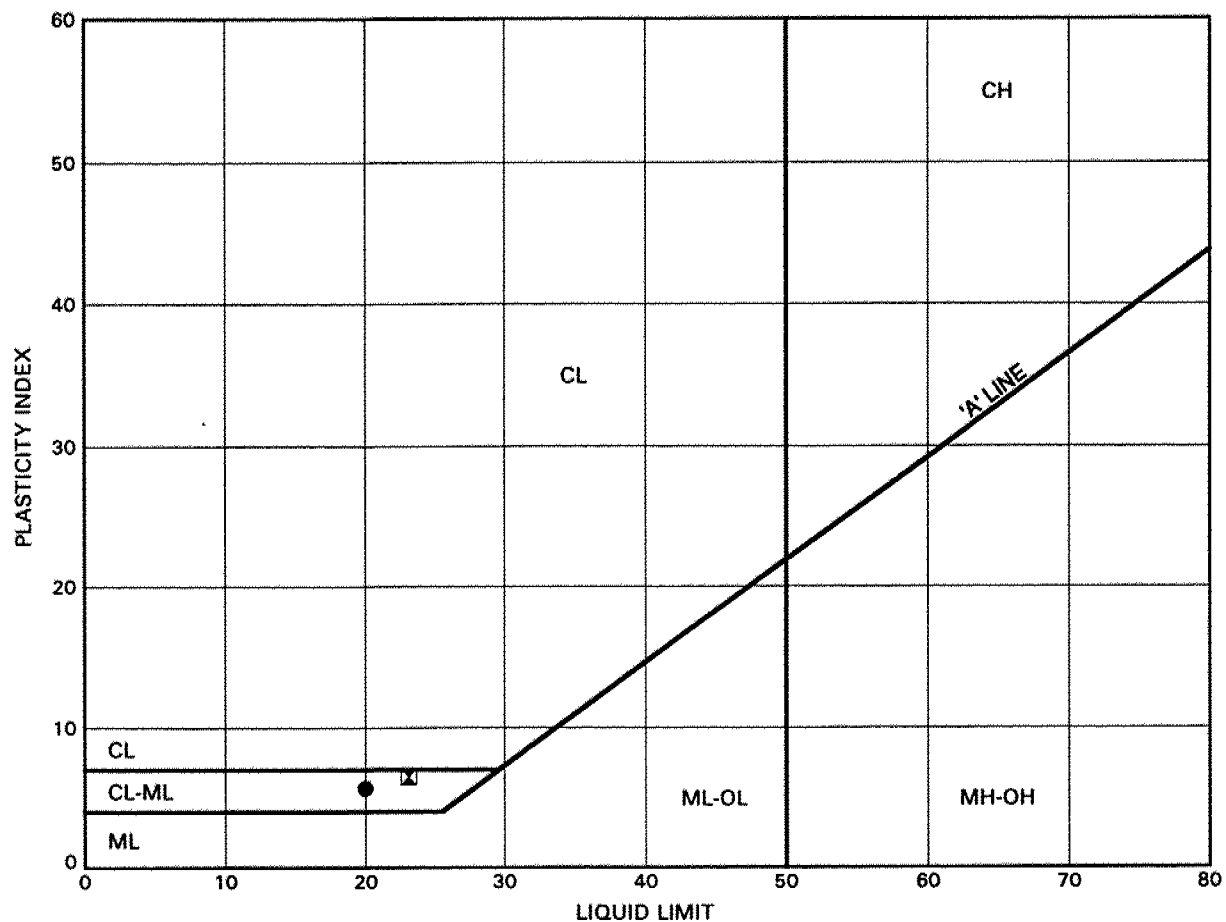
THURBER

Prep'd WM

Chkd. AEG

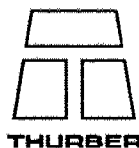
CNR OVERHEAD (NBL) ATTERBERG LIMITS TEST RESULTS

FIGURE B4



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	E-98-1	6.32	319.08
⊠	E-98-5	9.37	316.43

Date October 1998
Project 462-93-00



Prep'd WM
Chkd. AEG

Table 1**Results of pH and Sulphate Testing**

Sample	Depth (m)	pH	Sulphates (ppm)
E-98-2, SA2	0.8 - 1.2	6.41	47.1
E-98-3, SA3	1.5 - 2.0	5.83	32.6

DIST 52
CONT No
WP No 458-93-00

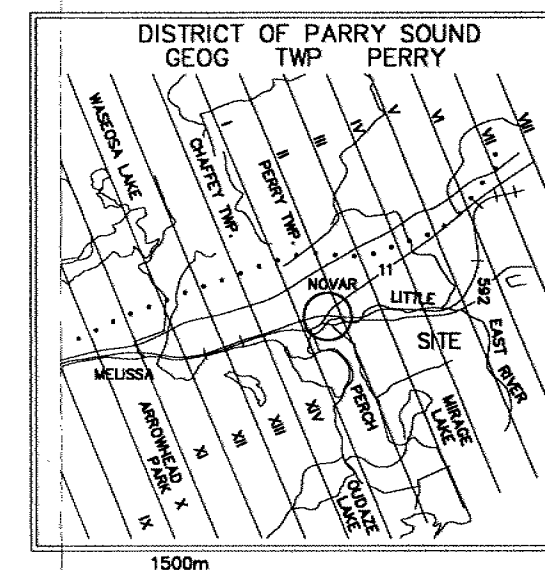
HIGHWAY 11- FOUR LANE
CNR OVERHEAD AT NOVAR



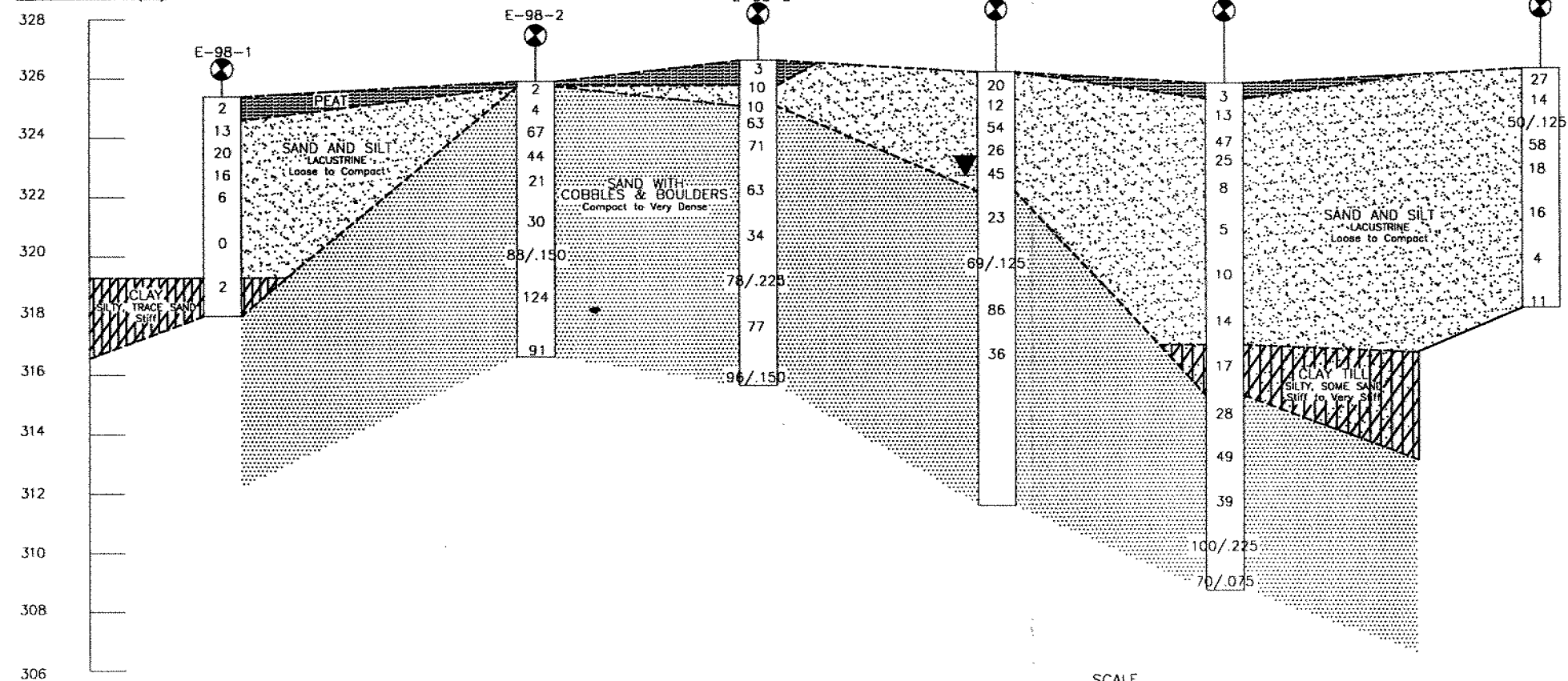
SHEET

MRC MCCORMICK RANKIN
CORPORATION

THURBER ENGINEERING LTD.



ELEVATION (M)



SCALE
V 1:200
H 1:500

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

LEGEND

- Borehole
- WL September 08, 1998
- 'N' Blows/0.3m (Std Pen Test)

No	ELEV.	LOCATION	
		NORTHING	EASTING
E-98-1	325.4	5 035 064.2	324 308.9
E-98-2	325.9	5 035 085.8	324 285.4
E-98-3	326.6	5 035 092.0	324 273.3
E-98-4	326.2	5 035 105.4	324 256.4
E-98-5	325.8	5 035 118.8	324 241.3
E-98-6	326.3	5 035 134.6	324 221.2

19-1351-7e-01

**CS COLE,
SHERMAN****FAX SHEET**TO: MINISTRY OF TRANSPORTATIONS DATE: July 21, 2000ATTENTION: Tony SANGIULIANO (FOUNDATIONS SECTION) TIME: _____FAX No.: 416-235-5240

Please deliver the following

FROM: Gene Woolings ASST. C.A._____ 5 Page(s)PROJECT No: 99-221

(which include this cover sheet)

RE: WP, 462-93-00PLING

Message: _____

Tony - As Per your Request.Gene Woolings (705) 788-6075

→ Karan Al-Bazi / Reno Radoli
McCormick Rankin
(905) 823-8500

Terry
(705) 788-7551.

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DELMAG

Special Construction Machines Limited

DELMAG D19-32 DIESEL PILE HAMMER

APPROXIMATE WEIGHTS

HAMMER
 TRIPPING DEVICE
 GUIDING FOR HAMMER & TRIP
 HELMET
 HELMET GUIDE BRACKET
 CUSHION BLOCK
 STRIKER PLATE

3,470. kg
 100. kg
 135. kg
 481. kg
 70. kg
 28.5 kg
 126. kg

ANY = 70.5 kg
 579.5

APPROX. WEIGHT OF SAFETY BRIDLE
 D19-32 RAM WEIGHT

TOTAL

4,410.5 kg
 164.0 kg

W = 1,820. kg → R

KN REG'D

(3750) 310x132 - C14R WBL

(2700) 310x110 - NOVAR OVERPASS

- LITTLE EAST NORTH
 - " " SOUTH
 - NORTH QUAD 034

PUMP ENERGY SETTINGS

4 - 57,585	Nm	=	42,440	Ft. Lbs.
3 - 48,070	Nm	=	35,420	Ft. Lbs.
2 - 38,455	Nm	=	28,320	Ft. Lbs.
1 - 28,800	Nm	=	21,220	Ft. Lbs.

ABOVE PUMP SETTINGS ARE SUBJECT TO CHANGE AND MAY VARY ON DIFFERENT SPECIFICATION SHEETS.

Steve Calow
 Steve Calow
 General Manager.

$$R = \frac{nwh}{S + \frac{1}{2}} \Rightarrow S + \frac{1}{2} = \frac{nwh}{R}$$

R: Ult. Pile Cap. in kN

As per contract drawing sheet 535

"piles to be driven in accordance with SS103-11 using ult. cap. of 3750 kN per pile." $\therefore R = 3750 \text{ kN}$

W = mass of Ram = 1820 kg (provided by contractor)
 $Wh = 57585 \text{ kJ}$ (provided by contractor) for
 D19-32 hammer

$$\eta = \frac{W + Pe^2}{W + P} \quad e = 0.32$$

Ex. For Pb NORTH Pier CNR OVERHEAD NOL

P5	P4	P3	P2	P1	P12
(P6)	P7	P8	P9	P10	P11

_____ R. W. H.

pile = 28.50m in length HP 310x132

* Anvil = 705.5 kg (provided by contractor)

* Hammer cushion 3x 13mm Aluminum with 50mm

layer of Baffle between.

$$1820 + \frac{(28.5 \times 132) + 705.5}{1820 + 4467.5} \cdot 0.32^2, S + \frac{1}{2} = \frac{0.302 (57585)}{3750} = 5.56 \text{ mm}$$

$$= 0.362 \checkmark \text{ vs } 0.368$$

PREPARED BY G. M. H. H. H.TITLE 1st ST. CA.DATE JUN 13/2000

PILE DETAILS	CONTRACT No. <u>42-10N</u>	DIST <u>10N</u>	REGION <u>10N</u>						
	STR SITE No. <u>42-10N</u>	STR WP No. <u>10N</u>	LOCATION <u>10N</u>						
	PILING CONTRACTOR <u>T. P. H. H. H.</u>								
	PILE No. AND LOCATION <u>10N</u> (ATTACH SKETCH) PILE TYPE <u>H-PILE</u> DESIGN CAPACITY <u>3750</u>								
HAMMER DETAILS	SIZE <u>310x137</u>	MASS <u>132</u> kg/m	PILE SHOE <u>YES</u>	BATTER <u>1/5</u>					
	INITIAL PILE LENGTH <u>4.10</u> m	SPliced	1	2	3	4	5	6	FINAL PILE LENGTH AFTER CUT-OFF <u>27.15</u> m
	TOTAL LENGTH OF PILE BEING DRIVEN AFTER SPlicing <u>16.2</u> <u>22.4</u> <u>28.5</u>								
	CUT-OFF ELEV <u>324.200</u> ACTUAL TIP ELEV <u>297.577</u> DESIGN TIP ELEV <u>312.000</u>								
HAMMER DETAILS	MECHANICAL HAMMER TYPE <u>DEMAR 19-37</u>		RATED ENERGY <u>57.585</u> JOULES/BLO						
	DROP HAMMER MASS (W) <u>705.5</u> kg		FALL (h) <u>1.20</u> m ENERGY (Wgh) <u>8466</u> JOULES/BLO						
	MASS OF ANVIL <u>705.5</u> kg		MASS OF MECHANICAL HAMMER RAM (W) <u>1820</u> kg FOLLOWER USED: YES <input type="checkbox"/> NO <input checked="" type="checkbox"/>						
	HAMMER CUSHION DETAILS <u>3</u>		PILE CUSHION DETAILS <u>1</u>						

GROUND ELEV AT
PILE LOCATIONS

DRIVING RECORD

DATE (G)

LENGTH IN GROUND m	PENETRATION BLOWS/20mm	LENGTH IN GROUND m	PENETRATION BLOWS/20mm	LENGTH IN GROUND m	PENETRATION BLOWS/20mm	LENGTH IN GROUND m	PENETRATION BLOWS/20mm	LENGTH IN GROUND m	PENETRATION BLOWS/20mm	LENGTH IN GROUND m	PENETRATION BLOWS/20mm	LENGTH IN GROUND m	PENETRATION BLOWS/20mm
1.2		4.2	12	12.2	21	18.2	22	24.2	19	27.2		30.2	
0.4		6.4	10	12.4	21	18.4	22	24.4	15	27.4		30.4	
0.6		6.6	6	12.6	15	18.6	21	24.6	15	27.6		30.6	
0.8		6.8	10	12.8	17	18.8	20	24.8	15	27.8		30.8	
1.0		7.0	6	13.0	15	19.0	20	25.0	18	28.0		31.0	
1.2	2	7.2	6	13.2	14	19.2	20	25.2	18	28.2		31.2	
1.4	2	7.4	10	13.4	16	19.4	20	25.4	21	28.4		31.4	
1.6	2	7.6	8	13.6	21	19.6	17	25.6	23	28.6		31.6	
1.8	2	7.8	9	13.8	19	19.8	20	25.8	25	28.8		31.8	
2.0	6	8.0	8	14.0	18	20.0	20	26.0	29	29.0		32.0	
2.2	6	8.2	10	14.2	20	20.2	20	26.2	24	29.2		32.2	
2.4	6	8.4	10	14.4	22	20.4	21	26.4	36	29.4		32.4	
2.6	5	8.6	9	14.6	21	20.6	20	26.6	55	29.6		32.6	
2.8	5	8.8	8	14.8	30	20.8	21	26.8		29.8		32.8	
3.0	5	9.0	9	15.0	20	21.0	19	27.0		30.0		33.0	
3.2	5	9.2	10	15.2	20	21.2	20						
3.4	5	9.4	16	15.4	20	21.4	14						
3.6	5	9.6	16	15.6	20	21.6	13						
3.8	2	9.8	12	15.8	21	21.8	23						
4.0	2	10.0	16	16.0	21	22.0	20						
4.2	5	10.2	21	16.2	22	22.2	17						
4.4	6	10.4	16	16.4	26	22.4	13						
4.6	12	10.6	15	16.6	26	22.6	12						
4.8	12	10.8	20	16.8	23	22.8	13						
5.0	12	11.0	16	17.0	31	23.0	14						
5.2	12	11.2	16	17.2	26	23.2	15						
5.4	12	11.4	16	17.4	30	23.4	16						
5.6	12	11.6	18	17.6	30	23.6	13						
5.8	16	11.8	22	17.8	33	23.8	13						
6.0	16	12.0	22	18.0	24	24.0	15						

RECORD OF LAST 100mm OF PENETRATION FROM GRAPH
PRODUCED ON FILE

PENETRATION	0 TO 20	20 TO 40	40 TO 60	60 TO 80	80 TO 100
BLOWS/20mm	12	17			
REMARKS (C)	5	1			

*NOTE: g = ACCELERATION DUE TO GRAVITY = 9.81 m/s²

MAIL COMPLETED FORM OR COPY TO:
MTO FOUNDATION DESIGN SECTION
ROOM 515, CENTRAL BUILDING
1201 WILSON AVENUE
DOWNSVIEW, ONTARIO M3M 1J8

P6 JULY 13/2000

N. PIER
NBL

CNR OVERHEAD

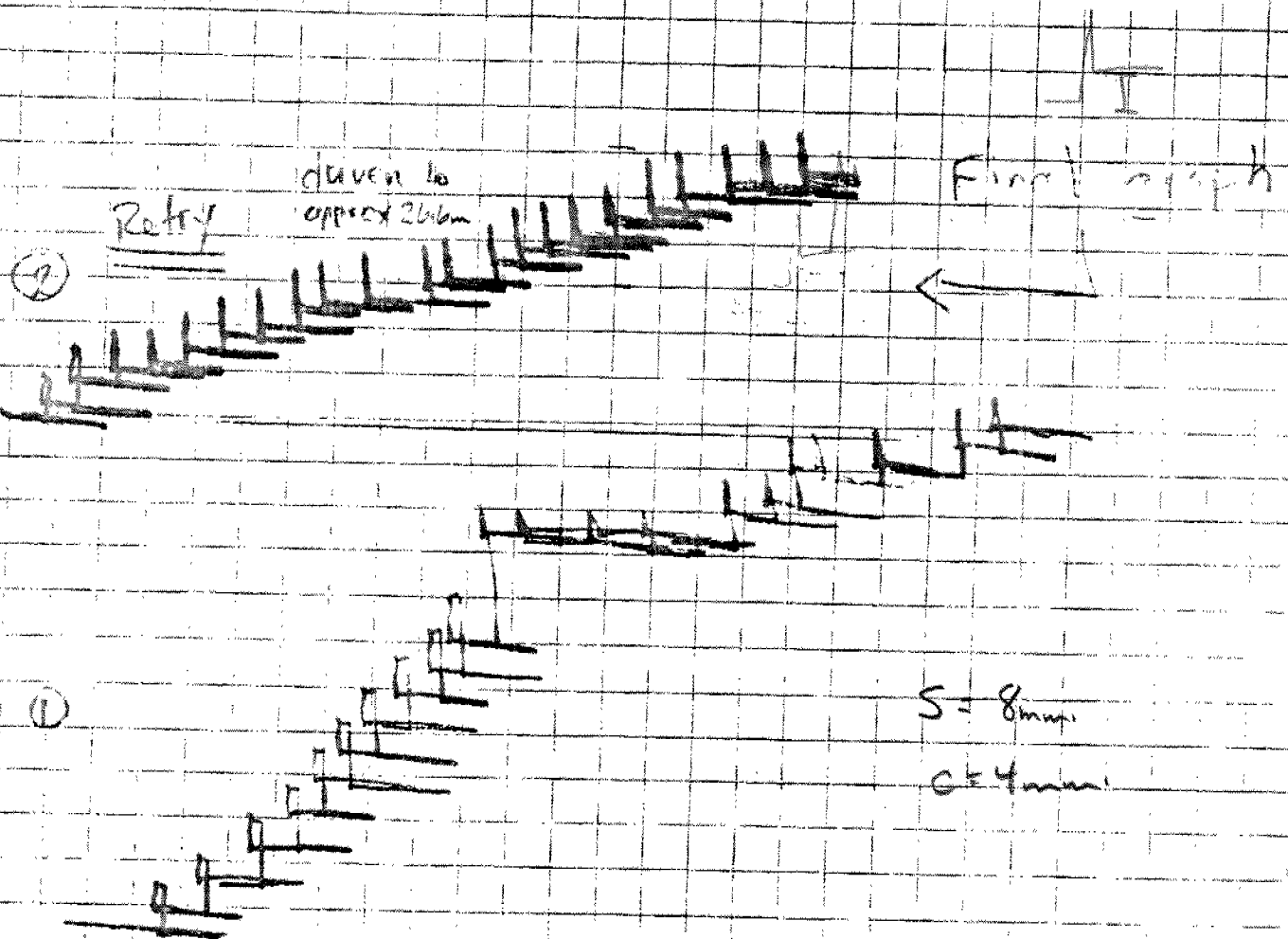
$S = 9/2$
 $2 \frac{1}{2} \times 2 \frac{1}{2}$
 $2 \frac{1}{2} \times 3$
 $3 \times 2 \frac{1}{2}$
 $3 \times 3 \times 2 \frac{1}{2}$

$S + \frac{6}{2}$
 5
 5
 $5 \frac{1}{2}$
 $5 \frac{1}{2}$

$C = \text{Avg. } 6$
 $S = 11 \times 3$

$S + \frac{6}{2}$
 $2 \frac{1}{2} \times \frac{6}{2}$
 $= 5.5$

$5 \frac{1}{2} < 5.56$ SO KILEY SATISF!



$$R = \frac{(0.362)(57585)}{8 + \frac{1}{2}}$$

185 Bathurst Avenue
Toronto, Ontario M5A 1P7
Tel: (416) 787-4259 Fax: (416) 787-4362



Special Construction Machines Limited

DELMA D19-32 DIESEL PILE HAMMER

APPROXIMATE WEIGHTS

HAMMER
TRIPPING DEVICE
GUIDING FOR HAMMER & TRIP
HELMET
HELMET GUIDE BRACKET
CUSHION BLOCK
STRIKER PLATE

3,470. kg
100. kg
135. kg
481. kg
70. kg
28.5 kg
126. kg

ANVIL
= 7-8.5 kg

TOTAL

4,410.5 kg
164.0 kg
1,820. kg → R.C.M.

APPROX. WEIGHT OF SAFETY BRIDLE
D19-32 RAM WEIGHT

FUEL PUMP ENERGY SETTINGS

Setting	Nm	Ft. Lbs.
* 4 - 57,585	42,440	
* 3 - 48,070	35,420	
* 2 - 38,455	28,320	
* 1 - 28,800	21,220	

KN READ
(3750) 310x132 - CLAR WBL
- ANVIL
(2700) 310x110 - NOVAE OVERPASS
- LITTLE EAST NOVAE
- NORTH QUASPA

ABOVE FUEL SETTINGS ARE SUBJECT TO CHANGE AND MAY VARY ON DIFFERENT SPECIFICATION SHEETS.

Steve Calow
Steve Calow
General Manager.

$$R = \frac{n W g h}{s + \frac{c}{2}}$$

$$W g h = 57,585 \text{ Joules}$$

$$n = \frac{w + P_e^2}{w + P}$$

$$e = 0.32$$

$$P = \text{pile length} \times 132 \frac{\text{KJ}}{\text{m}} + \text{anvil}$$

$$\text{anvil} = 705.5$$

~~$$W = 1820 \text{ Kg}$$~~

$$W = 1820 \text{ Kg}$$

$$\text{Tip } EI^4 = 311 \text{ m}$$

$$\text{Cut-off } EI^4 = 324 - 326$$

} Embedment Length ≥ 13 to 15 m.

MEMORANDUM



To: S. Cunningham
CCO
Northern Region

05 May 2000

From: Pavements and Foundations Section
Room 223, Central Bldg.

Tel: (416) 235-5267
Fax: (416) 235-5240

Re: Temporary Track Protection
CNR Overhead NBL
Hwy 11 Four Laning
6.7 km North of Hwy 60 northerly 13 km
WP 465-93-01, Site 42-10N
District 52, Huntsville

In response to your request dated May 2, 2000 regarding the temporary track protection design as submitted by Jim Donn Piling and as produced by RWB Engineering on their drawing dated April 13, 2000, we forward the following comments:

General

On MTO projects, it is normal policy that the Temporary Roadway or Track Protection design is the responsibility of the Contractor. An end result specification as recommended in our memorandum dated January 8, 1999 should have been included in the contract documents. The specification identifies the submission and design requirements. Typically, a performance level is defined and then the Contractor is responsible for retaining a designer for producing a design that satisfies the design requirements. Material, construction, removal and quality control requirements are also specified in the specification.

The Contractor is responsible for submitting a certificate of conformance that states that the system satisfies the performance requirements and that the monitoring requirements have been completed. Hence, the MTO does not review the design. A copy of this specification is attached.

Project Specific

On this project, a track protection design was originally included in the Contract Documents. The Contractor has however proposed an alternative design.

The proposed track protection design consists of a cantilever soldier pile – lagging system. The soldier pile size has been increased from HP 250 X 85 piles to HP 310 X 79 piles. The depth of penetration of the piles have been increased by approximately 500 mm and are to be installed to a tip elevation of 319.9. The base of excavation for the structure foundations is to be founded at Elevation 323.9. The soldier piles are to be driven in place rather than installed in preaugered holes.

A 610 mm x 25 mm plate is to be welded to the soldier pile within the embedment length of the soldier pile. It is presumed that this plate is designed to augment the passive resistance of the track protection system.

The subsurface conditions at the site consist of cohesionless sand and silts with boulders and cobbles. The groundwater table at the time of the investigation was approximately at Elevation 323 m.

The Contractor should be reminded that they are responsible and accountable for the work and the work shall be conducted to conform to the end result specification. Notwithstanding, the Contractor should be alerted of the following concerns:

1. Pile impediment could be caused by the presence of boulders and cobbles in the deposit. The Contractor's procedure should outline the method of controlling the pile installation to the design tip elevation.
2. A dewatering scheme shall be carefully designed and installed to ensure that the groundwater table is effectively drawdown as required without soil migration through the timber lagging. Any soil loss behind the lagging wall can undermine the railway tracks.

If you require additional information, please do not hesitate to contact our office.

T. Sangiuliano, P. Eng.
Foundation Engineer

for

D. Dundas, P. Eng.
Senior Foundation Engineer

End Result Specification

Roadway/Track Protection

ROADWAY PROTECTION - Item No.
TRACK PROTECTION - Item No.

Special Provision No. 539S01

April 2000

Amendment to OPSS 539

OPSS 539, December 1983, Construction Specification for Protection Schemes is deleted and replaced with the following:

539.01 SCOPE

This specification covers the requirements for the design, construction, maintenance, monitoring and removal of a protection system made necessary by excavation or other work..

539.02 REFERENCES

This specification refers to the following standards, publications or specifications:

Ontario Provincial Standard Specifications, Construction:

OPSS 903 Piling
OPSS 904 Concrete Structures
OPSS 906 Structural Steel

Ontario Provincial Standard Specifications, Material:

OPSS 1350 Concrete Materials and Production
OPSS 1601 Wood Material, Preservative Treatment and Shop Fabrication

Ontario Ministry of Labour:

Occupational Health and Safety Act

American Association of State Highways Transportation Officials:

AASHTO Guide Design Specification for Bridge Temporary Works

539.03 DEFINITIONS

For the purpose of this specification, the following definitions apply.

Anchorage System: means a system consisting of tendons installed in predrilled holes in soil or rock and encapsulated in grout or concrete that derives its load carrying capacity in bond between the grout/concrete body and the surrounding soil or rock; or tie back to deadmen.

Bracing: means the system of walers, struts, anchorages and like members that connect frames, shores or panels of a sheathing system to resist external pressures and to provide stability against lateral movement.

Cofferdam: means a water-tight enclosure.

Design Engineer: means the Engineer who produces the original design.

Design Checking Engineer: means the Engineer who checks the original design.

Dredge Line: means the exposed lower limit of the *Protection System*.

Erector: means a person that undertakes the construction of a Protection System.

Protection System: means the construction necessary to support existing or proposed work such that its function will not be affected, or, construction necessary to support work, such as open excavations, during actual building operations for safety and convenience.

Raker: means a structural member inclined to the front of the shoring wall providing lateral support.

Shoring Wall: means a structural wall consisting of wood, steel, concrete or combination of these materials that supports earth or rock and any structure, materials, utilities or other facility contained in or on the supported earth or rock mass.

Top of Shoring Wall: means the upper limit of the Protection System.

539.04 SUBMISSION AND DESIGN REQUIREMENTS

539.04.01 Submissions

539.04.01.01 Working Drawings

Three (3) copies of construction drawings shall be submitted to the Contract Administrator for

information purposes as required by:

- a) the Occupational Health and Safety Act.
- b) the Contract Drawings
- c) the Contractor's method of construction

All submissions shall bear the seal and signature of a design Engineer and design checking Engineer.

For contracts where another authority is affected such as a railway or navigable waters, the Contractor shall submit working drawings to each authority at least four(4) weeks prior to the commencement of work.

The requirements of each authority shall be satisfied before commencement of protection system installation.

539.04.01.02 Construction Drawings/Details Requirements

539.04.01.02.01 Information To Be Shown on Construction Drawings/Details

- a) Plans, Elevations & Details
 - Location of protection system and station limits
 - Plan and elevation of shoring showing the extent of the protection system.
 - Details of the shoring system including cross-sections.
- b) Design Criteria
 - Pressure diagrams including values of horizontal and vertical loads, dead load and live load surcharge.
 - Design assumptions and parameters.
 - Anchor bond stresses.
 - Pile and Anchor System stressing schedule specifying working loads.
 - Details of preload where required.
- c) Materials
 - Grade of structural steel and grade and species of structural wood.
 - Concrete strengths.
 - Grout strengths.
 - Details of protection from rain and frost action.
 - Wood lagging and size.
 - Mill certificates or test reports from an independent organization certified by the Standard Council of Canada certifying that the steel meets the requirements of the grade specified.
- d) Installation Procedure
 - Installation sequence and procedure including but not limited to the installation of piling, lagging, anchor systems and rakers.

- e) **Monitoring Method**
 - The proposed method of monitoring the performance of the Protection System during installation and use. The method of monitoring shall be consistent with the requirements specified in the Construction Section of this specification.
- f) **Removal of Protection System**
 - The details of the procedures associated with the removal of the protection system indicating: method, sequence of work, and removal limits.

539.04.01.03 Qualifications

Designer: The Design Engineer and Design Checking Engineer shall have demonstrated expertise for the work. The design checking Engineer shall have a minimum of five (5) years experience in designing protection schemes of similar nature and scope to the required work.

Erector: All work shall be performed under the direction of personnel experienced in the method of construction of protection systems. Such experience shall have been obtained within the preceding five years on projects of similar nature and scope to the required work.

Quality Verification Engineer: The Quality Verification Engineer shall have a minimum of five(5) years experience in the design of comparable protection systems, or alternatively with demonstrated expertise through providing satisfactory quality verification services for a minimum of two (2) projects in which the work was of similar scope to that in the Contract.. The Quality Verification Engineer shall be retained by the Contractor to certify that the work is in general conformance with the contract documents and to issue Certificate(s) of Conformance.

539.04.01.04 Certificates of Conformance

The Contractor shall submit, to the Contract Administrator, a Certificate of Conformance sealed and signed by the Quality Verification Engineer upon completion of each of the following operations and prior to commencement of each subsequent operation:

- a) Layout and Extent of Protection System
- b) Piling
- c) Restraining System(Anchors, Rakers, etc)
- d) Performance of Protection System (based on measurements of movement as specified in Section 539.04.02.01)
- 5) removal and disposal

The Certificates of Conformance shall state that the materials and work have been supplied and installed in general conformance with the working drawings. The Certificate of Conformance for the 'Performance of Protection System' milestone shall state that the system satisfies the performance requirements and shall identify that the monitoring requirements as specified have

been completed.

Upon completion of the operation of the protection system, the Contractor shall submit to the Contract Administrator a final Certificate of Conformance sealed and signed by the Quality Verification Engineer. The Certificate shall state that the protection system has been installed and has performed in general conformance with the stamped working drawings and contract documents.

539.04.01.06 Amendments to Protection Systems

Work shall not proceed on amendments to the protection system until the Contractor has received sealed and signed approval to proceed from the original design Engineer and design checking Engineer and has submitted a copy of the approval to the Contract Administrator.

All amendments to Protection System shall be submitted to the Quality Verification Engineer on revised Construction Drawings/Details bearing the seal and signature of the original designengineer and design checking Engineer.

539.04.01.07 Preconstruction Survey

Prior to commencing the work, the Contractor shall submit to the CA, a condition survey of property and structures that may be affected by the work. The survey shall include, but not be limited to, the locations and conditions of adjacent properties, buildings, underground structures, utility services and structures such as walls abutting the site.

539.04.02 Design

539.04.02.01 General

The protection system shall be designed for the performance level as specified in the Contract.

The Contractor shall be responsible for the complete detailed design of the protection system required to carry out the work shown on the contract drawings.

The geotechnical/foundation portion of the design shall be based on a method published in AASHTO Guide Design Specification for Bridge Temporary Works and appropriate for the specific site conditions. Design methods not meeting this design specification may be used on a particular contract when prequalified by the Owner.

A protection system shall be designed to provide protection for excavations as required by the Occupational Health and Safety Act, at the locations specified in the Contract, and at any other

location where the stability, safety or function of an existing structure and/or utility may be impaired by construction work.

The temporary slope geometry used to determine requirements of the protection system shall be in accordance with the Occupational Health and Safety Act.

Lateral movement of any portion of the protection system shall not exceed the following:

Performance Level

Level 1	Angular Distortion 1:1000 but shall not be more than 10 mm
Level 2	Angular Distortion 1:200 but shall not be more than 25 mm
Level 3	Angular Distortion 1:100 but shall not be more than 50 mm

Angular Distortion = $\pm D / H$

D = Horizontal displacement (mm) at height H

H = Height (mm) above dredge line to point of measurement or height above the nearest system retraining support.

When performance Level 1 is specified, the bracing system shall be preloaded.

Where the bracing systems are preloaded the effects of the preload shall not cause damage to adjacent facilities.

When any part of a structure foundation is within a horizontal distance of $\frac{1}{3} H$ from the face of the protection system, movement of the protection system shall not exceed 5 mm.

539.04.02.02 Design Assumptions

The design assumptions shall accurately represent the subsurface conditions prevalent at the site, and shall be specific to the type of protection system used. Soil, rock and groundwater conditions are described in the Foundation Investigation Report for the Contract.

539.04.02.03 Vertical and Horizontal Loadings

Vertical and horizontal loadings used shall represent existing conditions and accepted design practice. Future loadings that are known and may affect the protection system during its useful life shall be included

539.05 MATERIALS

539.05.01 Wood

Wood shall be according to OPSS 1601, shall be of the size, grade and species shown on the working drawings and shall be in sound condition, free from defects which will impair its strength. Wood lagging does not have to be grade-stamped.

539.05.02 Metal

Structural steel shall be according to OPSS 906. Piling shall be according to OPSS 903. The material shall be of the grade shown on the working drawings.

539.05.03 Proprietary Shoring and Patented Accesories

Where proprietary shoring or patented accessories, are to be used, the Contractor shall follow the Manufacturers' recommendations as to load carrying capacity. The recommended load carrying capacities shall be supported by test results from a qualified and recognized testing laboratories.

539.05.04 Concrete

Concrete shall be according to OPSS 1350.

539.05.05 Other Materials

The designer may consider other suitable materials when sufficient information is available to quantify the allowable design loads or when the manufacturer's recommendations as to load carrying capacities are supported by test results from an independent organization accredited by the Standards Council of Canada.

539.07 CONSTRUCTION

539.07.01 General

Protection systems shall be built according to the specifications and the stamped working drawings.

Concrete construction shall be according to OPSS 904.

Structural steel shall be according to OPSS 906.

Piling shall be according to OPSS 903.

The protection system shall be protected from the detrimental effects of rain and frost action.

Material used in the protection system shall remain the property of the Contractor unless otherwise specified.

Loss of soil from behind the shoring shall be prevented during and following the installation of the lagging.

Concrete shall be placed in the dry unless otherwise specified in the contract. Where cofferdams are used they shall be sealed sufficiently to permit concrete to be placed in the dry. When concrete cannot be placed in the dry, tremie techniques shall be employed according to OPSS 904.

539.07.02 Removal of Protection Systems

Protection systems, shall be removed from the right-of-way unless otherwise specified in the contract.

The Contractor shall obtain approval from the Ministry of the Environment and other approving Authorities when all or any portion of the protection system is to be left in place.

Where piles are left in place the top shall be removed to at least 1.2 m below the finished grade or ground level or at least 0.6 m below the stream bed.

The method and sequence of removal shall be such that there will be no damage to new work, existing work and the facility being protected.

Unless otherwise specified the area remaining disturbed after removal of the protection system shall be restored to as close to its original condition as possible.

539.08 Quality Assurance/Quality Control

The Contractor shall undertake a preconstruction site condition survey and monitor the installation as specified herein.

A preconstruction site condition survey shall be done on adjacent structures and facilities within a horizontal distance of 2 H from the face of the protection system. Monitoring shall be conducted by a Registered Ontario Land Surveyor or an Engineer according to the program submitted with the construction drawings/details.

The protection systems shall be monitored during construction. Readings shall be taken during installation of the protection system at the top, at each restraint point, at the dredge line and

halfway between the restraint points at each construction stage. After installation the above readings shall be taken weekly.

All test results, observations and records including the preconstruction damage survey taken during construction and operation of the protection system shall be available on the site for review by the Contract Administrator.

If movement of the protection system is more rapid than is expected, or if movement approaches the allowable limit, the Contract Administrator shall be notified and suitable measures shall be taken to ensure stability of the protection system and to ensure movement does not exceed the performance level specified.

539.10 BASIS OF PAYMENT

Payment at the contract price for the above tender items shall be full compensation for all labour, equipment and materials to do the work.

Where the contract shows that a protection system is required but a tender item is not included and where a protection system is made necessary because of the Contractor's operations the cost shall be included in the item or items directly associated with the protection system and shall be full compensation for all labour, equipment and materials required to carry out the work including subsequently removing the temporary protection system and performing any necessary restoration work.

WARRANT: Always with these tender items.

☐ URGENT!

☒ AS SOON AS POSSIBLE

SUBJECT

RECEIVED

MAY 4 2000

PAVEMENT AND
FOUNDATIONS
SECTION

TO

DAVE DUNDAS

FROM

STEVE CUNNINGHAM

DEPT

MUNTSVILLE

CONT 99-221

DATE

MAY 2, 2000

MESSAGE

DAVE

COULD YOU PLEASE REVIEW AS DISCUSSED

THANKS

Steve

REPLY

REPLY FROM

DATE

☐ URGENT!

☒ AS SOON AS POSSIBLE

SUBJECT

TO *DAVE DUNDAS*

FROM *STEVE CUNNINGHAM*

DEPT. *KUNTSVILLE*
CONT 99-221

DATE *May 2, 2000*

MESSAGE

DAVE

COULD YOU PLEASE REVIEW AS DISCUSSED

THANKS

Steve

REPLY

REPLY FROM

DATE

☐ URGENT!

☒ AS SOON AS
POSSIBLE

SUBJECT

REPLY COPY SEND TO

DAVE DUNDAS

STEVE CUNNINGHAM

MUNTSVILLE

DEPT.

CONT 99 221

DATE

11/24/2, 2000

MESSAGE

DAVE

COULD YOU PLEASE REVIEW AS DISCUSSED

THANKS

Steve

REPLY

REPLY FROM

DATE

TRACK PROTECTION - REMOVAL PROCEDURE

MTO 99-221 CNR Overhead @ Novar

Removal of track protection as follows:

- Steel soldier piles and timber lagging to be cut off and removed to 300 mm below proposed final ground elevation.
- Removed materials to be disposed of off site by Looby

TRACK PROTECTION - MONITORING SYSTEM

MTO 99-221 CNR Overhead @ Novar

The offset distance from a determined control line and the top of pile elevation and will be measured and recorded at least once per week for each pile to ensure that movement does not exceed the performance levels specified.

Readings will be recorded on the attached monitoring sheets and relevant observations will be noted on the comments sheets. These reports will be on file in the Looby site office.

TRACK PROTECTION - MONITORING SYSTEM

MT0 99-221 CNR Overhead @ Novar - Track Protection

South Track Protection

Completed By: _____

I
S1

I
S2

I
S3

I
S4

I
35I
56

I
\$7

58

59

[illegible]

**NTO 99-221 CNR Overhead @ Novar - Track Protection**

North Track Protection

Completed By: _____

I
N1

I
N2

I
N3

I
N4

I
N5

IN

I
N7

NE

1
N9

[illegible]

**MTO 99-221 CNR Overhead @ Novar - Track Protection**

South Track Protection

Completed By: _____

I
S1

I
52

I
S3

I
54

I
35

I
S6

57

I
SBI
SE[illegible]

**MTD 99-221 CNR Overhead @ Novar - Track Protection**

North Track Protection

Completed By: _____

1

N9

[illegible]

MEMORANDUM



To: V. Minassian, P. Eng.
Senior Project Engineer
Planning and Design, Northern Region

January 9, 1999

From: Pavements and Foundations Section
Room 315, Central Bldg.

Tel: (416) 235-5267
Fax: (416) 235-5240

Re: Pile Design Reassessments
Novar Rd Underpass
Waseosa Bridge
WP 462-93-00
Hwy 11, From 6.7 km N of Hwy 60 Northerly 13.6 km
District 52, Huntsville

We have received your memoranda dated December 23, 1998 and January 4, 1998 that describes changes to the pile capacities and lengths at the Novar Rd Underpass and Waseosa Bridge structure. The changes in the recommendations are summarized in Table 1 attached to this memorandum. Our review comments are contained in this memorandum.

1. The increase in embedment lengths are as follows:

Waseosa

3 metres @ W. Abutment
12 metres @ Pier
16 metres @ E. Abutment

Novar Rd

10 metres @ W. Abutment
12 metres @ Pier
10 metres @ E. Abutment

2. In our review of the original recommendations, we commented that the axial capacities were larger than MTO would conventionally recommend at sites with similar subsoil conditions. This was communicated to the Consultant and the Consultant agreed to reduce the axial capacities. The axial capacities were to be reduced to a magnitude of approximately 750 kN at SLS and 1100 Kn at ULS
3. The revised capacities are approximately 15% higher at the Waseosa Bridge and 10% at the Novar Bridge with an increase in embedment length ranging from 10% to 57% at Waseosa

and approximately 30% at Novar Rd. The recommended revised capacities are based on static analysis which is the least accurate method of predicting pile capacities.

4. The Consultant has justified the pile increase by increasing the embedment lengths. However, we have two concerns regarding the increased embedment lengths:

- Proposed tip elevations will be at a depth significantly below the depth explored during the foundation investigation. (Up to 25 metres at the Waseosa Bridge). Consequently, we will be penetrating depths within which we have no subsurface information. This is not conventional MTO practice.

- Although the piles are designed as friction piles that develop their supporting capacity from the shaft of the pile, there is no certainty that the actual capacity will increase linearly with increased embedment. In fact, critical embedment depths can actually limit the capacities.

In summary, the pile capacities recommended are based on static analysis. The results of the static analysis suggest pile capacities larger than actual pile load test data. Despite the fact that the embedment lengths are larger, in our opinion, there is a larger risk than normal associated with the larger capacities recommended.

Consequently, we reiterate our recommendation that consideration be given to conducting a pile load test that would yield site specific accurate results. The results could be extrapolated to the other sites on the project and would minimize the risk. Details of the pile load test including development of the testing specifications and costs can be provided by our office.

If you have any questions or require additional information, please do not hesitate to contact this office.

T. Sangiuliano, P. Eng.
Foundation Engineer

for

D. Dundas, P. Eng.
Senior Foundation Engineer

cc. T. Kazmierowski

Table 1 - Revised Pile Capacities and Lengths



MEMORANDUM



To: V. Minassian, P. Eng.
Senior Project Engineer
Planning and Design, Northern Region

January 8, 1999

From: Pavements and Foundations Section
Room 315, Central Bldg.

Tel: (416) 235-5267
Fax: (416) 235-5240

Re: Technical Review Package
WP 462-93-00
Hwy 11, From 6.7 km N of Hwy 60 Northerly 13.6 km
District 52, Huntsville

We have received the contract drawings and documents for the above mentioned project. Our review comments are contained in this memorandum.

Little East River Bridges

South Bridge/North Bridge

1. Interlocking Steel Sheet Piling

The Construction sequence includes the installation of sheet piling. The Construction sequence shall also include the removal of the sheet piling.

2. Dewatering

It is recommended that a Dewatering NSSP be included in the Contract that alerts the Contractor of the fact that the cohesionless soils present at the site are susceptible to conditions of unbalanced head and that the Contractor is responsible for rendering a stable excavation without inducing soil disturbance.

3. CSP Installation

Clarification is required regarding the groundwater level drawdown. Presently, it is stated on the contract drawings to "lower water table within sheet piling to 1.0 m below projected base of excavation" It is assumed that this implies that the groundwater level be lowered to an elevation that corresponds to 1.0m below the peat subexcavation level (approximately Elevation 315.5). The base of the augered hole within which the 600 mm diameter CSP is to be installed is at Elevation 312.5. Under these conditions, the Contractor will need to auger the hole beneath the groundwater table. It is recommended that an NSSP included in the Contract Documents to alert the Contractor of the cohesionless soil conditions submerged below the groundwater table and these soils are susceptible to conditions of unbalanced head and that the Contractor is responsible for rendering a stable augered

hole without any soil sloughing or soil cave in.

4. Thickness of Granular Bentonite

On Section 1, Sheet 519, there is a note that states: "clay seal inside and outside CSP to **El. 314.0m**"

The base of the augered hole is at Elevation 312.5m. Two 500 m lift thickness of granular bentonite have been described in the Construction. Consequently, $312.5\text{m} + 0.5\text{m} + 0.5\text{m} = \mathbf{313.5\text{m}}$.

There is a difference of 0.5 m in the two elevations.

5. Filter Material

It is suggested that the Piling Note 8(filter material specifications) be removed as a piling note and relocated to Step 7 of the construction sequence that describes the filter sand placement.

6. Piles

a. Piling Lengths

South Bridge

The piling lengths tabulated in the Pile Data table correspond to tip elevations of 280.9 m. The design tip elevation is 273.5m

North Bridge

The piling lengths tabulated in the Pile Data table correspond to tip elevations of 288.9 m. The design tip elevation is 275m

b. Piling Installation

It is recommended that the piles be driven to the design tip elevation and the capacity verified using the Hiley Dynamic Formula at the design tip elevation. Consequently the Piling Note 5 and the note on Section 1 on Sheet 519 should be revised accordingly.

c. Retapping Piles

It is recommended that selective retapping of piles be undertaken firstly rather than retapping all the piles. In the revised OPSS 903, 10 % of the piles but no fewer than two are to be retapped no sooner than 24h after installation to confirm the axial compressive resistance is sustained.

7 Approach Embankments

a. The Foundation Investigation and Design Report recommends the subexcavation of peat and all deleterious material within the approach embankment fill area. This requirement

has not been illustrated or stated on the contract drawings.

- b. Transverse embankment slopes are designed at a geometry of 1.25H:1V. Forward embankment slopes, on the other hand, are designed at 2H:1V. An explanation of the different slope geometries is requested.

Novar Rd U'Pass

1. Piling

Installation

It is recommended that the piles be driven to the design tip elevation and the capacity verified using the Hiley Dynamic Formula at the design tip elevation. Consequently, it is suggested that the Piling Note 5 be revised to identify that the pile capacity be verified at the design tip elevation.

Retapping

It is recommended that selective retapping of piles be undertaken firstly rather than retapping all the piles. In the revised OPSS 903, 10 % of the piles but no fewer than two are to be retapped no sooner than 24h after installation to confirm the axial compressive resistance is sustained.

1. Dewatering - Pier

It is recommended that a Dewatering NSSP be included in the Contract that alerts the Contractor of the fact that the cohesionless soils present at the site are susceptible to conditions of unbalanced head and that the Contractor is responsible for rendering a stable excavation without inducing soil disturbance. The pier pile cap shall be constructed in the dry.

3. Approach Embankment

Subexcavation

On sheet 546, it is identified and illustrated that unsuitable material is to be removed. However, this removal appears to be only applicable to the construction of earth pads at the abutment structure foundation location. The subexcavation has to be extended to cover the entire approach embankment area.

Slope Geometry

The approach embankments have been designed at a 3H:1V slope geometry and a 2 metre midheight berm. Historically, for similar surface and subsoil conditions, we have designed such embankments at a 2H:1V slope geometry.

CNR Overhead

1. Piles

Pile Size

On projects with similar subsurface conditions, HP 310 x 110 piles have also been used. It should be confirmed that HP 310 x 132 piles are more economically viable.

Pile Installation

In view of the presence of boulders and cobbles at the site, it is essential that the pile installation be carefully controlled using the Hiley Dynamic Formula to ensure that the piles are not damaged during the installation.

Retapping Piles

It is recommended that selective retapping of piles be undertaken firstly rather than retapping all the piles. In the revised OPSS 903, 10 % of the piles but no fewer than two are to be retapped no sooner than 24h after installation to confirm the axial compressive resistance is sustained.

2. Roadway Protection

It is recommended that an end result specification that covers the design, construction, monitoring and removal of protection schemes be included in the contract documents. This specification can be obtained from our office.

3. Dewatering - Piers

It is recommended that a Dewatering NSSP be included in the Contract that alerts the Contractor of the fact that the cohesionless soils present at the site are susceptible to conditions of unbalanced head and that the Contractor is responsible for rendering a stable excavation without inducing soil disturbance. The pier pile caps shall be constructed in the dry.

4. Approach Embankments

Subexcavation

The requirement to subexcavate the peat and other organic and deleterious material shall be identified on the contract drawings.

Slope Geometry

The approach embankments have been designed at a 3H:1V slope geometry and a 2 metre midheight

berm. Historically, for similar surface and subsoil conditions, we have designed such embankments at a 2H:1V slope geometry.

Jessop Creek Culvert

1. Bearing Capacity at SLS

The foundation report predicts settlement magnitudes in the order of 250 mm under the proposed new NBL embankment loading at the culvert location. Please confirm that the culvert design has considered this magnitude of settlement. It appears that the culvert has been articulated to perhaps accommodate some of the movement.

2. Erosion Control

Erosion control measures at the culvert outlet are not illustrated.

3. Dewatering

It is recommended that an NSSP be included in the Contract Documents that specifies that the culvert be constructed in the dry. The Contractor shall be alerted that the cohesionless soils present at the site are susceptible to conditions of unbalanced head and that the Contractor is responsible for rendering a stable excavation without inducing soil disturbance.

It should also be specified that the Contractor submit a proposal that addresses any temporary creek diversion.

4. Roadway Protection

It is recommended that an end result specification that covers the design, construction, monitoring and removal of protection schemes be included in the contract documents. This specification can be obtained from our office.

North Waseosa Lake Rd Underpass

1. Borehole Plan and Soil Stratigraphy Drawing

This drawing should be included in the contract documents.

2. Footing Layout and Pile Details

This drawing is not included in the contract documents.

3. Retained Soil System

RSS design criteria (Geometry, Appearance and Performance) should be identified on the contract drawing

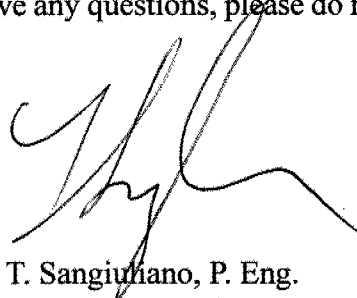
The design approach regarding specifically the application of the CSP's is inconsistent with the design of the CSP's at the Little East River Bridges where two culverts(600mm diameter and 800mm diameter CSP's are used).

Our contract drawing review comments were submitted in a memorandum dated September 9, 1998.

4. Dewatering - Pier

It is recommended that a Dewatering NSSP be included in the Contract that alerts the Contractor of the fact that the cohesionless soils present at the site are susceptible to conditions of unbalanced head and that the Contractor is responsible for rendering a stable excavation without inducing soil disturbance. The pier pile cap shall be constructed in the dry.

We trust these comments are sufficient for your purposes. If you have any questions, please do not hesitate to contact this office.



T. Sangiuliano, P. Eng.
Foundation Engineer

for

D. Dundas, P. Eng.
Senior Foundation Engineer

cc. T. Kazmierowski

MEMORANDUM



December

~~September 9, 1998~~

To: V. Minassian, P. Eng.
Senior Project Engineer
Planning and Design, Northern Region

From: Pavements and Foundations Section
Room 315, Central Bldg.

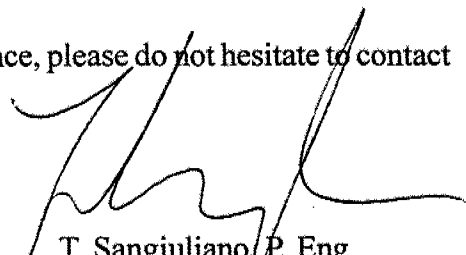
Tel: (416) 235-5267
Fax: (416) 235-5240

Re: Foundation Investigation Report Reviews
Little East River North & South
CNR Overhead (NBL)
WP 462-93-00
Hwy 11, From 6.7 km N of Hwy 60 Northerly 13.6 km
District 52, Huntsville

As requested in your memorandum dated December 3, 1998, we have reviewed the Final Foundation Investigation Reports for the Little East River Northbound and Southbound structures and the CNR Overhead structure. Comments derived from our review of the draft reports were submitted in our memoranda dated July 28, 1998 and September 9, 1998 for the CNR Overhead and the Little East River structures respectively.

In general, our previous comments have been addressed in the Final Foundation Reports. Only the figures illustrating the grain size distribution curves have not been changed. As stated previously, conventionally figures are presented based on soil type rather than borehole and we have no further comments.

We have no further comments. If you require additional assistance, please do not hesitate to contact our office.

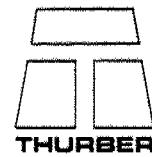


T. Sangiuliano, P. Eng.
Foundation Engineer

for

D. Dundas, P. Eng.
Senior Foundation Engineer

cc. T. Kazmierowski



December 14, 1998

File: 19-1351-7

McCormick Rankin
2655 North Sheridan Way
Mississauga, Ontario
L5K 2P8

Attention: Mr. Reno Radolli, P. Eng.

**Re: Novar Road Underpass, Highway 11
Pile Capacity and Length**

Dear Sir:

Further to our recent meeting with MTO where they expressed a concern regarding the recommended pile capacity and lengths for the Novar Road Underpass, we have reassessed the pile design. The results of this reassessment are summarized in Table 1 attached. Table 1 indicates that the piles may have to be driven deeper than indicated in our original design.

We understand that the drawings for the Novar Road underpass have not been issued for Tender yet and the pile driving note for Novar Bridge should be modified as follows :

Piles to be driven in accordance with Standards SS 103-10 or SS 103-11 using an ultimate capacity of 2700 kN but must be driven below the following elevations for the various foundation elements:

Foundation Element	Elevation of Base of Abutment Stem or Pile Cap	Piles Must be Driven Below Elevation
West Abutment	324.8	281.8
Central Pier	318.5	275.5
East Abutment	326.3	283.3

The revised ultimate resistance corresponding to Hiley's Formula is based on the stated MTO policy (refer to MTO letter dated September 9, 1998 on Little East River) in which ultimate resistance is taken as two (2) times the Factored ULS Capacity.

Continued....

McCormick Rankin

- 2 -

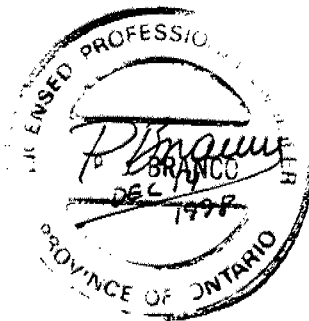
December 14, 1998

We trust the contents of this fax clarify the results of our reassessment which addresses the concerns of the MTO foundation section. Please do not hesitate to call if you require any additional clarification.

Yours very truly,
Thurber Engineering Ltd.



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



Paulo Branco, P. Eng.
Principal

ea\c:\19\1351\7.letter1

Continued....

Table 1
Pile Design Reassessment : Novar Road Underpass, Hwy 11

Bridge	Foundation Element	No. of Piles	SLS Capacity per pile (kN)	Factured ULS Capacity per pile (kN)	Ultimate Pile Capacity for control of driving based on Hiley's Formula (kN)	Most likely pile depth based on our reassessment (m below abutment stem or pile cap)
Novar	West Abutment	8	900	1350	2700	43
	Central Pier	8	900	1350	2700	43
	East Abutment	8	900	1350	2700	43

Previous Recs

1000 1600 33m

1000 1600 31m

1000 1600 3m

GEOCRES No. 31E-125DIST. 52 REGION W.P. No. 462-93-00CONT. No. W. O. No. STR. SITE No. HWY. No. 11LOCATION Snowmobile Crossing6.7 km N of Hwy 60 N'ly 13 kmNo of PAGES - 1

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. REMARKS:

Re: Draft Report
Foundation Investigation and Design Report (dated November 27, 1998)
Snowmobile Crossing Station 17+650 between 6.7km N of Hwy 60 and 19.7 km
N of Hwy 60
WP 462-93-00
Hwy 11, District 52, Huntsville



P.K.

I'm sending a fax instead of the report for ease of delivery.

- 1) The Site # should be indicated on the title page and on the Foundation Drawing.
- 2) The Foundation Drawing does not meet our expectations. Normally north would be to the top of the page although I understand that there may be a reason for orienting this way. The section line should be shown on the plan.
- 3) Do the number of boreholes meet the requirements of the terms of reference? I do not have access to the Terms of Reference at the time of this review but it is possible that 3 boreholes with 1 borehole between existing boreholes may have been considered. Notwithstanding, the soil profile seems to be a thick deposit of consistent overburden so that an additional borehole may not be required for this project. ✓
- 4) Are there concerns that the foundation soil is susceptible to disturbance under conditions of unbalanced hydrostatic head? If so, this concern should be noted in the factual (Foundation Investigation Report) portion of the report as this is the only information that is included in the Contract. Also, the Consultant should include explicit recommendations that suitable unwatering specifications be included in the Contract Documents if necessary. It is recognized that the culvert invert may be above the groundwater. ✓
- 5) Recommendations for bearing resistance, frost protection, bedding, camber, joint details erosion protection at inlet, erosion protection at outlet should be considered. It is recognized that the water control provisions will not be required. ✓
- 6) On page 11, the recommendation of K_a for culvert headwalls instead of K_o needs clarifications perhaps considering type of retaining wall including Retained Soil Systems (proprietary retaining walls on designated source list). That is, to describe how to achieve unrestrained condition.
- 8) On page 12, proof rolling could be a mechanism to confirm that peat has been removed. ✓
- 9) On page 6, thickness of native sand should be noted as well as top and bottom elevation. ✓
- 10) Same comment as #9 for Silt. ✓
- 11) On page 8 explicit recommendations are required for preparing base on rock fill. Also would ✓

it be possible to avoid placing rock fill below the base of the culvert (if it would be of benefit and if it was practical)?

If there are any questions, please call.

Dave

**FINAL FOUNDATION INVESTGATION AND DESIGN REPORT FOR
PROPOSED SNOWMOBILE CROSSING
HIGHWAY 11 FOUR LANING
6.7 km NORTH OF HWY 60 NORTHERLY 13 km
W.P. 462-93-00,
DISTRICT 52, HUNTSVILLE**

Report

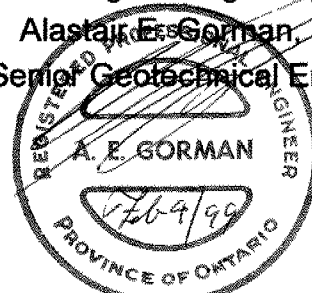
to

McCormick Rankin Corporation

Thurber Engineering Ltd.
170 Evans Avenue, Suite 101
Etobicoke, Ontario
M8Z 5Y6
Phone: (416) 503 3600
Fax: (416) 503 3010

Direction of fieldwork and engineering analysis by:

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



Report reviewed by:

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

February 3, 1999
File: 19-1351-7F

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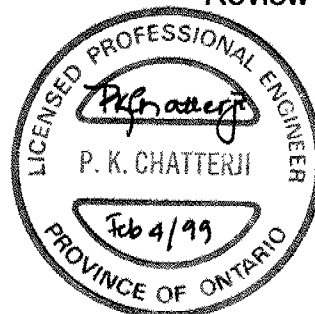


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FIGURES

Figure 1 Configuration of Rock Fill with Trapezoidal Wedge Gap and Granular Backfill

DRAWINGS

19-1351-7f-01 Borehole Location Plan and Soil Profile

APPENDICES

Appendix A Borehole Logs
Appendix B Laboratory Test Results

**FINAL FOUNDATION INVESTIGATION AND DESIGN REPORT FOR
PROPOSED SNOWMOBILE CROSSING
HIGHWAY 11 FOUR LANING
6.7 km NORTH OF HWY 60 NORTHERLY 13 km
W.P. 462-93-00,
DISTRICT 52, HUNTSVILLE**

1. INTRODUCTION

This report presents the results of the foundation investigation and analysis carried out by Thurber Engineering Ltd. (Thurber) at the site of the proposed snowmobile crossing under Highway 11 in the Geog. Twp. of Perry. The purpose of the investigation was to explore the subsurface soil and groundwater conditions at the site and based on the data obtained provide borehole logs, soil profile and a written description of the subsurface conditions. The purpose of the analysis of the data obtained during the investigation was to provide recommendations for the design and construction of the structure foundations and associated earth works.

Thurber carried out the investigation as a sub-consultant to McCormick Rankin Corporation (MRC) under Ministry of Transportation (MTO) Agreement 9750 - 7424 - 5262.

2. SITE DESCRIPTION

2.1 Site Location

The subject site lies in the Right-of-Way (ROW) of the four-laning of Highway 11 north of Huntsville and is located where the ROW crosses an abandoned railway track bed now used in winter as a snowmobile trail. The site lies at Station 17+650, approximately, C/L Median Highway 11 or approximately 6 km north of Novar.

At the site, the existing highway is carried on an embankment approximately 5.0 m high over an area of muskeg.

2.2 Physiography

Based on The Physiography of Southern Ontario, 3rd Edition, by Chapman and Putnam, the region surrounding the site consists of bedrock ridges with shallow overburden. The bedrock is undifferentiated igneous and metamorphic rock of early Precambrian age and is generally hard and massively jointed.

Along this part of the Highway 11 corridor, the troughs between the exposed bedrock ridges are generally filled with fine sand deposits and muskeg.

On the west side of the ROW, the snowmobile route lies at the toe of a low hill rising to the north. For approximately 700 to 800 m to the south, the exposed surface soils consist of muskeg. To the east of the ROW, the muskeg soils stretch both to the south and to the north. At present, a small creek is carried under the existing Highway 11 embankment in a culvert. Free water is visible in much of the muskeg area.

3. INVESTIGATION PROCEDURES

3.1 Field Investigation

Between July 16 and 20, 1998, a Nodwell track mounted auger rig was used on site for drilling, Standard Penetration Testing (SPT, following the procedures of ASTM D 1586) and dynamic cone penetration testing. One hole (F-98-2) was drilled from the west shoulder of the existing highway for the purposes of exploring both the existing embankment fill and the underlying native soil. A second sampled borehole (F-98-1) was drilled in the alignment of the proposed new southbound lanes, giving a total of two sampled boreholes. One dynamic cone penetration test was carried out close to the second borehole (F-98-1). The approximate locations of the boreholes and dynamic cone penetration test are shown on Drawing 19-1351-7f-01.

The holes were advanced using hollow stem augers which were kept full of drilling mud at all times to counteract the effect of an unbalanced head of

groundwater in the fine sand and silt soils encountered at the site.

The boreholes were numbered F-98-1 and F-98-2. The depths of sampling in the two boreholes were as follows:

Borehole No.	Depth of Sampling (m)
F-98-1	9.8
F-98-2	15.7

Samples were recovered at selected intervals throughout the depths of the boreholes in conjunction with Standard Penetration Tests (SPT) (following the test procedure outlined in ASTM D 1586). Samples were generally recovered at intervals of 0.75 m in the upper 3.0 m and thereafter at intervals of 1.5 m.

A dynamic cone penetration test was conducted adjacent to one hole as follows

Borehole No.	Depth of Dynamic Cone Test
F-98-1	From ground surface to 15.2 m, adjacent to the hole

The site lies in an area of muskeg with open areas of free water at the surface. For design purposes, the groundwater level is assumed to lie at the ground surface.

All recovered samples were examined, identified and logged in the field and were transported to Thurber's Toronto laboratory by the field supervisor for further examination and laboratory analysis.

The result of the drilling, sampling and in-situ testing are summarized on the borehole logs in Appendix A.

3.2 Laboratory Analysis

Geotechnical laboratory testing consisted of natural moisture content determinations and visual classifications of all recovered samples. In addition, grain size analyses were conducted on selected samples. The results of the laboratory testing are presented on the borehole logs in Appendix A, and in Figures B1 and B2 in Appendix B.

4. DESCRIPTION OF SUBSURFACE CONDITIONS

4.1 Subsurface Soil Conditions

Detailed descriptions of the subsoil conditions encountered in the boreholes are presented on the borehole logs in Appendix A. The stratigraphic profile inferred from the borehole information is shown on Drawing No. 19-1351-7f-01.

The underlying native soil at the site consists of silt, trace to some sand, trace gravel. Prior to highway construction, the silt was overlain by some coarser sand alluvium and by peat. As a result of the construction of Highway 11, the native soils are overlain at Borehole F-98-2 by a sand, gravelly, trace to some silt (fill). The different soil types are described below.

Main Embankment Fill

The main embankment fill is the material in BH F-98-2 between the ground surface at Elevation 334.6 and a depth of 6.7 m, Elevation 327.9.

Borehole F-98-2, drilled on the shoulder of the existing highway, first penetrated the fill placed for the construction of the highway embankment. The composition of this fill is described as sand, gravelly, trace to some silt. It was found to be in a compact to very dense state based on SPT values ranging from 15 to 94 blows for 0.3 m penetration and one case of 50 blows to advance 0.13 m. The fill is brown in colour and was found to have natural moisture contents ranging from 3 to 18%.

Lower Sand

Below Elevation 327.9, the borehole penetrated further sand deposits down to Elevation 325.6, approximately. This 2.3 m thick layer has been labelled the "lower sand".

This layer consists of sand, trace gravel, trace to some silt, peat and organic layers. It is apparent that this layer represents a zone in which replacement of the original peat occurred. There has been some mixing of embankment fill and some of the original peat. Though sample recovery was poor, a layer of peat up to 50 mm thick was visible in the recovered samples. It is apparent that the original peat was not completely removed. The lower limit of this layer was indistinct but has been interpreted from sample examination to lie at about Elevation 325.6. It is possible that this layer represents fill originally place to construct the rail bed.

The lower sand layer was found to be in a very loose to compact state, based on SPT values ranging from 1 to 12 blows for 0.3 m penetration. The colour of the deposit ranged from dark brown to black and the measured natural moisture contents ranged from 18 to 21%.

Peat

Borehole F-98-1, drilled beyond the west toe of the existing embankment, encountered a 0.9 m layer of peat at the surface. The peat is described as very soft, sandy, trace gravel, trace wood, trace roots. The measured SPT values were 2 for 0.3 m penetration and the measured natural moisture contents were 128 and 408%. The peat is dark brown to black.

In the borehole, the peat extended down to Elevation 328.6, a thickness of 0.9 m. This borehole is believed to be positioned within the old rail bed and thus may represent an area where peat has already been stripped and replaced by fill. It is possible that the depths of peat to the north and south of the borehole will be appreciably greater.

Native Sand

Below the peat, Borehole F-98-1 encountered a layer of sand which is considered to correspond to the lower sand described at the base of the embankment fill. The sand extended from Elevation 328.6 to 326.3, a thickness of 2.3 m.

The sand is described as gravelly, trace silt and the SPT values ranging from 2 to 21 blows for 0.3 m penetration indicate very loose to compact conditions. On the basis of visual identification and the location of the borehole, this sand may be fill originally placed to construct the rail bed. The measured natural moisture contents ranged from 15 to 21%.

Silt

Below the sand in Borehole F-98-1 and the lower sand in Borehole F-98-2, both boreholes encountered a deposit of silt. The silt was encountered at Elevation 326.3 in Borehole F-98-1 and 325.6 in Borehole F-98-2. The silt layer had not been penetrated by Elevation 318.9, establishing that it is at least 7.4 m thick.

This silt is described as containing trace to some sand, trace gravel. Based on SPT values ranging from 3 to 11, the deposit is in a very loose to compact state. This state of relative density is generally confirmed by the results of a dynamic cone penetration test carried out adjacent to Borehole F-98-1. The silt is grey and wet.

Both boreholes drilled at this site terminated in the silt deposit at Elevations 319.7 and 318.9.

4.2 Groundwater

The groundwater level is essentially equal to the adjacent free water level at the toe of the existing embankment and for design purposes may be assumed to be at Elevation 329.5.

4.3 Disturbance in Excavation

The soils encountered at this site will be susceptible to base boiling and disturbance in any excavations which may be carried down below the groundwater table prevailing at the time of construction. This condition will be due to the unbalanced hydrostatic head as the excavation is taken below the water table. If any such excavation is required, consideration should be given to prior unwatering to lower the water table at least 1.0 m below the base of the excavation.

5. RECOMMENDATIONS FOR STRUCTURE FOUNDATIONS

5.1 Type of Structure

The proposed structure will be a concrete box section with inside measurements of 3.5 m high by 4.0 m wide. The invert will lie close to the elevation of the existing abandoned rail bed, which is essentially the same as the ground Elevation 329.5 at borehole F-98-1. It is understood that separate structures will be constructed in the new embankment and the existing embankment with a gap between them in the median. The length of the structure will be approximately 23 m under the SBL and 29 m under the NBL (existing embankment).

Geotechnical recommendations are required for the design and construction of the structures within the new and the existing embankments.

5.2 Construction Within the New Embankment

It is understood that the Geotechnical/Pavement Design Report recommends that the crossing of the wetland be accomplished by stripping the peat and constructing a rock fill embankment. It is recommended that the proposed snowmobile crossing structure be constructed within the new embankment in the same fashion as a concrete box culvert.

Settlement of the loose to compact foundation soils under the weight of the embankment is one of the principal issues related to the performance of a concrete box section within a new embankment. A settlement analysis was conducted for the box section in the new embankment assuming the following:

- native mineral soil at Elevation 328.6±, based on Borehole F-98-1, with all overlying peat stripped
- Granular A fill constructed up to the underside of the box section, assumed Elevation 329.0
- box section constructed on the base described above
- rock fill and road section completed to final elevation, assumed to be

at Elevation 334.5 for the purposes of this analysis

The results of the analysis have been reported below as settlements at the centreline of the concrete box section and at various points across the width of the highway embankment. The following settlements were computed:

Centreline of Highway	¼ Point	Edge of Embankment
265 mm	249 mm	167 mm

Due to the nature of the soils below the proposed embankment, the settlements are expected to be "immediate" in nature and to be essentially complete shortly after embankment construction.

If the new embankment is built in an advance contract at least one year prior to construction of the box section, the following sequence is recommended to facilitate completion of the majority of the settlement prior to placing the structure and to facilitate construction of the box section within the embankment:

1. Strip all peat and muck from the area of construction. Based on Borehole F-98-1, firm bottom should be encountered by Elevation 328.6, or a depth of 0.9 m below existing grade. This borehole is believed to have been drilled through the old rail bed and the depth of stripping to the north and south may be greater. Reference should be made to the borehole results reported in the Geotechnical report for an indication of possible depths.
2. Place Granular A fill along the alignment of the box section, extending at least 1.0 m beyond the culvert in all directions and to at least 300 mm above the design underside of the concrete base slab of the box section. It is anticipated that the top of the granular A will be about water level and that no effective compaction can be carried out on the granular A as it is placed. See Figure 1 for a schematic illustration of the arrangement.

3. Construct the rockfill embankment to its full height leaving a trapezoidal wedge shaped gap at the location of the box section, as illustrated in Figure 1.
4. Chink the surface of the rock fill in accordance with normal practice. The chinking should cover the top of the embankment plus the sides and base of the wedge. Chinking must effectively prevent future loss of road sub-base material into the interstices of the rock fill.
5. Backfill the volume of the wedge with granular material as specified for road sub-base material compacting to at least 95% SPMDD.
6. At this stage, it is assumed that the embankment has been in place for at least one year.
7. Excavate the granular fill in the wedge to accommodate construction of the box section. Stockpile the excavated granular material for use as backfill around and above the box section. Excavation should be carried down to the required level for construction or to the top of the previously placed Granular A (see Step 2 above), whichever is lower. Make up any difference with Granular A compacted to 98% SPMDD.
8. Prepare a well compacted, uniform foundation for the structure.
9. Construct the box section. No specific standards exist, but reference should be made to OPSS 422 and OPSD 803.021 for guidance.
10. Backfill around and above the box section using the stockpiled granular material up to subgrade level. For this situation, compaction to 98% SPMDD is recommended.

If the concrete box section must be constructed soon after the embankment is completed, i.e. without the anticipated one year delay, the construction sequence should be as above but with the following additional steps substituted for Step 6:

- 6.a Overbuild the granular fill to at least 1.0 m above final subgrade level and for 10.0 m to either side of the centreline of the box section to provide surcharge.
- 6.b After the granular fill has been placed the settlement of the embankment should be monitored by a Geotechnical Engineer. Construction of the box section may begin after the Engineer has

decided that the settlement has reached at least 90% of the estimated total value. Settlement may be monitored by means of a settlement plate on the centreline of the box section and centreline of the embankment. Level surveying to ± 5 mm should be sufficiently accurate.

Some small rebound and recompression of the foundation soil will occur during the excavation and backfilling to construct the box section, but this is expected to be too small to affect the structure.

Following the construction sequence outlined above will remove the need to construct the culvert with a camber to compensate for settlement.

The geotechnical bearing resistance available at the underside of the concrete box section based on the construction procedure outlined above may be conservatively taken as:

- factored ULS 500 kPa
- SLS 200 kPa.

The method of construction outlined above will result in the concrete box culvert being surrounded by granular fill and being underlain by granular fill and rock fill. The result of this construction will be a zone of non-frost susceptible fill surrounding the concrete box section and no further frost protection is considered necessary.

If the ultimate design results in a situation where water flow may be allowed through the box section, consideration should be given to erosion protection at the inlet and outlet.

All excavations must be carried out in conformance with the Occupational Health and Safety Act (OHSA).

5.3 Construction Within the Existing Embankment

The results of the investigation, as represented by the log for Borehole F-98-2, indicate that the embankment has been constructed mainly of sand, gravelly with trace silt. With an assumed structure invert at approximate Elevation 329.0, the underside of the structure will bear on compact, gravelly sand fill underlain by the very loose to compact lower sand.

The effect of constructing the 3.5 m high box section will be to reduce the vertical stress on the lower sand due to the existing embankment. As a result, there should be no new settlement of the underlying soils due to the construction of the box section. Some elastic rebound and recompression may occur due to the process of excavation and backfilling. Since the unloading and reloading will occur over a confined area, the reconsolidation as the backfill is placed will likely be minor and within tolerable limits for the structure.

It is recommended that the structure be placed in an open excavation within the existing embankment following the procedure set out below:

1. Excavate across the embankment width to accommodate the box section. It is recommended that the base of the excavation be at least 1.0 m wider than the box section on each side and be carried down to a level 200 mm below the underside of the base of the box section.
2. Prepare a well compacted, uniform foundation for the structure. Proof rolling may be specified as a method of detecting soft areas in the base of the excavation which will require special attention.
3. Place Granular A up to the underside of the box as a bedding layer.
4. Construct the box section. No specific standards exist, but reference should be made to OPSS 422 and OPSD 803.021 for guidance.
5. Backfill around and above the box section using Granular B, Type I up to subgrade level. For this situation, compaction to 98% SPMDD is recommended.

Material excavated from the existing embankment may be used as backfill provided it meets the requirements of OPSS 1010, or is expressly approved

by the Engineer.

The geotechnical bearing resistance available at the underside of the concrete box section based on the construction procedure outlined above may be conservatively taken as:

- factored ULS 500 kPa
- SLS 200 kPa.

The method of construction outlined above will result in the concrete box culvert being surrounded by granular fill and being underlain by granular fill and rock fill. The result of this construction will be a zone of non-frost susceptible fill surrounding the concrete box section and no further frost protection is considered necessary.

If the ultimate design results in a situation where water flow may be allowed through the box section, consideration should be given to erosion protection at the inlet and outlet.

5.4 Settlements Between Structures

Since the structures within the two embankments will be separated by a median gap, any small differential movement which may occur between the two structures is not an issue from a geotechnical perspective.

6. EARTH PRESSURE

The lateral earth pressures to be used in design of the box section and associated wing walls should be computed in accordance with Section 6-7 of the OHBDC .

The granular backfill should conform to Ontario Provincial Standard Specifications (OPSS) 1010 for Granular B, Type 1 and the walls of the box section and any wing walls may be designed based on the following unfactored earth pressure distribution:

$$pH = K \gamma h$$

where;

K = earth pressure coefficient, use value from table below.

γ = unit weight of soil, = 21.2 kN/m³ for Granular B

h = depth below top of wall, m

Wall Type	Earth Pressure Coefficient (K)	
	OPSS Granular B $\phi' = 30^\circ$	
	Horizontal Ground Surface Behind Wall	Sloping Ground Surface (2H:1V)
Box Section Walls (And Restrained Walls)	0.50	-
Unrestrained Wing Walls	0.33	0.55

If the wing walls are cast integrally with the concrete box section, it is recommended that they be treated as restrained walls. Alternatively, it may be possible to construct the wing walls as Restrained Soil Systems (RSS) or other system which can tolerate some forward movement, in which case it is appropriate to treat the wing walls as unrestrained (active case).

Additional lateral pressure must be added to account for compaction induced forces. The additional pressure must be computed in accordance with Section 6-7.4.3 of the OHBDC.

7. CONSTRUCTION CONCERNS

Construction within the existing embankment may encounter soil conditions differing from those described in this report and upon which the design recommendations are based. In particular, work elsewhere in the embankment crossing the Scotia Wetlands has revealed 1.2 m of buried peat. While the existing embankment is in equilibrium, and a net unloading is contemplated, construction of the box section directly on buried peat, or soil containing significant proportions of peat is not acceptable. Such materials must be subexcavated and replaced with compacted granular material. The contractor should be directed to inspect the subgrade of the excavation and draw to the attention of the Engineer any deleterious conditions which are encountered. Proof rolling may be specified as a method of detecting soft areas in the base of the excavation which will require special attention. It is very important to found the box section on uniform and competent foundation soil conditions to minimize the differential settlement of the structure.

The possibility of excavation below the groundwater level prevailing at the time of construction is also a concern. The contractor should be warned not to excavate to such levels without effective, prior unwatering. An NSSP to this effect should be included in the contract if necessary.

8. CONSTRUCTION INSPECTION AND MONITORING

During construction, all excavation, foundation preparation and embankment construction activities should be monitored by geotechnical personnel to confirm that the foundation recommendations and design are being correctly implemented and that the soil conditions encountered do not differ materially from the interpretation used in this report.

STATEMENT OF GENERAL CONDITIONS

1. STANDARD OF CARE

This study and Report have been prepared in accordance with generally accepted engineering or environmental consulting practices in this area. No other warranty, expressed or implied, is made.

2. COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report which is of a summary nature and is not intended to stand alone without reference to the instructions given to us by the Client, communications between us and the Client, and to any other reports, writings, proposals or documents prepared by us for the Client relative to the specific site described herein, all of which constitute the Report.

IN ORDER TO PROPERLY UNDERSTAND THE SUGGESTIONS, RECOMMENDATIONS AND OPINIONS EXPRESSED HEREIN, REFERENCE MUST BE MADE TO THE WHOLE OF THE REPORT. WE CANNOT BE RESPONSIBLE FOR USE BY ANY PARTY OF PORTIONS OF THE REPORT WITHOUT REFERENCE TO THE WHOLE REPORT.

3. BASIS OF REPORT

The Report has been prepared for the specific site, development, design objectives and purpose that were described to us by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the document are only valid to the extent that there has been no material alteration to or variation from any of the said descriptions provided to us unless we are specifically requested by the Client to review and revise the Report in light of such alteration or variation.

4. USE OF THE REPORT

The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client. NO OTHER PARTY MAY USE OR RELY UPON THE REPORT OR ANY PORTION THEREOF WITHOUT OUR WRITTEN CONSENT. WE WILL CONSENT TO ANY REASONABLE REQUEST BY THE CLIENT TO APPROVE THE USE OF THIS REPORT BY OTHER PARTIES AS "APPROVED USERS". The contents of the Report remain our copyright property and we authorize only the Client and Approved Users to make copies of the Report only in such quantities as are reasonably necessary for the use of the Report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make the Report, or any portion thereof, available to any party without our written permission. Any use which a third party makes of the Report, or any portion of the Report, are the sole responsibility of such third parties. We accept no responsibility for damages suffered by any third party resulting from unauthorized use of the Report.

5. INTERPRETATION OF THE REPORT

- a) Nature and Exactness of Soil and Contaminant Description: Classification and identification of soils, rocks, geological units, contaminant materials and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgemental in nature and even comprehensive sampling and testing programs, implemented with the appropriate equipment by experienced personnel, may fail to locate some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and all persons making use of such documents or records should be aware of, and accept, this risk. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. Where special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.

(see over...)

INTERPRETATION OF THE REPORT *(continued)*

- b) Reliance on Provided Information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to us. We have relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, we cannot accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of misstatements, omissions, misrepresentations, or fraudulent acts of persons providing information.

6. RISK LIMITATION

Geotechnical engineering and environmental consulting projects often have the potential to encounter pollutants or hazardous substances and the potential to cause an accidental release of those substances. In consideration of the provision of the services by us, which are for the Client's benefit, the Client agrees to hold harmless and to indemnify and defend us and our directors, officers, servants, agents, employees, workmen and contractors (hereinafter referred to as the "Company") from and against any and all claims, losses, damages, demands, disputes, liability and legal investigative costs of defence, whether for personal injury including death, or any other loss whatsoever, regardless of any action or omission on the part of the Company, that result from an accidental release of pollutants or hazardous substances occurring as a result of carrying out this Project. This indemnification shall extend to all Claims brought or threatened against the Company under any federal or provincial statute as a result of conducting work on this Project. In addition to the above indemnification, the Client further agrees not to bring any claims against the Company in connection with any of the aforementioned causes.

7. SERVICES OF SUBCONSULTANTS AND CONTRACTORS

The conduct of engineering and environmental studies frequently requires hiring the services of individuals and companies with special expertise and/or services which we do not provide. We may arrange the hiring of these services as a convenience to our Clients. As these services are for the Clients' benefit, the Client agrees to hold the Company harmless and to indemnify and defend us from and against all claims arising through such hirings to the extent that the Client would incur had he hired those services directly. This includes responsibility for payment for services rendered and pursuit of damages for errors, omissions or negligence by those parties in carrying out their work. In particular, these conditions apply to the use of drilling, excavation and laboratory testing services.

8. CONTROL OF WORK AND JOBSITE SAFETY

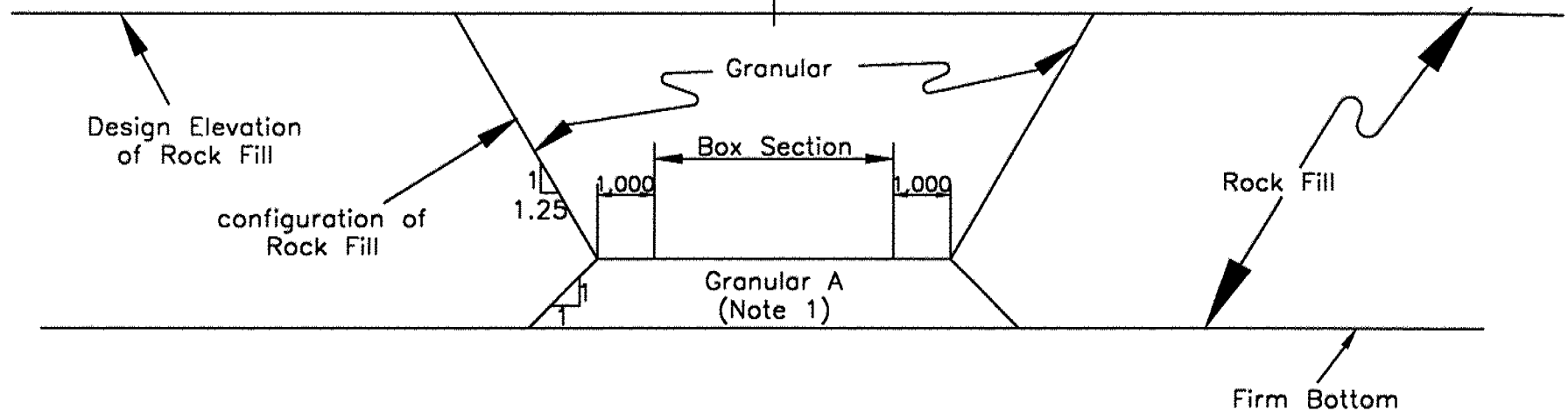
We are responsible only for the activities of our employees on the jobsite. The presence of our personnel on the site shall not be construed in any way to relieve the Client or any contractors on site from their responsibilities for site safety. The Client acknowledges that he, his representatives, contractors or others retain control of the site and that we never occupy a position of control of the site. The Client undertakes to inform us of all hazardous conditions, or other relevant conditions of which the Client is aware. The Client also recognizes that our activities may uncover previously unknown hazardous conditions or materials and that such a discovery may result in the necessity to undertake emergency procedures to protect our employees as well as the public at large and the environment in general. These procedures may well involve additional costs outside of any budgets previously agreed to. The Client agrees to pay us for any expenses incurred as the result of such discoveries and to compensate us through payment of additional fees and expenses for time spent by us to deal with the consequences of such discoveries. The Client also acknowledges that in some cases the discovery of hazardous conditions and materials will require that certain regulatory bodies be informed and the Client agrees that notification to such bodies by us will not be a cause of action or dispute.

9. INDEPENDENT JUDGEMENTS OF CLIENT

The information, interpretations and conclusions in the Report are based on our interpretation of conditions revealed through limited investigation conducted within a defined scope of services. We cannot accept responsibility for independent conclusions, interpretations, interpolations and/or decisions of the Client, or others who may come into possession of the Report, or any part thereof, which may be based on information contained in the Report. This restriction of liability includes decisions made to either purchase or sell land.

Note 1. Top of Granular A to be at least 300mm above the design underside of concrete.

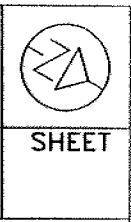
C.L.
BOX SECTION



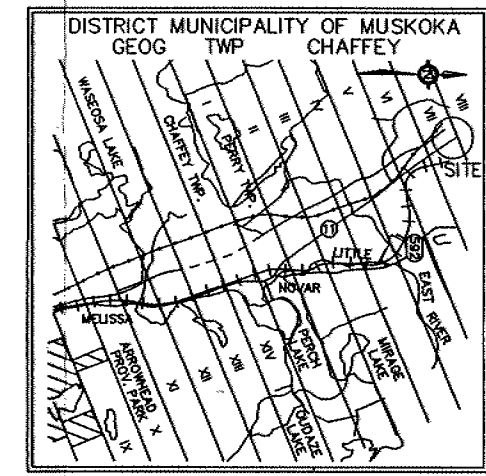
CONFIGURATION OF ROCK FILL WITH TRAPEZOIDAL
WEDGE GAP AND GRANULAR BACKFILL
(SCHEMATIC ONLY)

DIST 52
 CONT No
 WP No 462-93-00

HIGHWAY 11- FOUR LANE
 SNOW MOBILE CROSSING AT APPROX.
 STATION 17+650



THURBER ENGINEERING LTD.



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

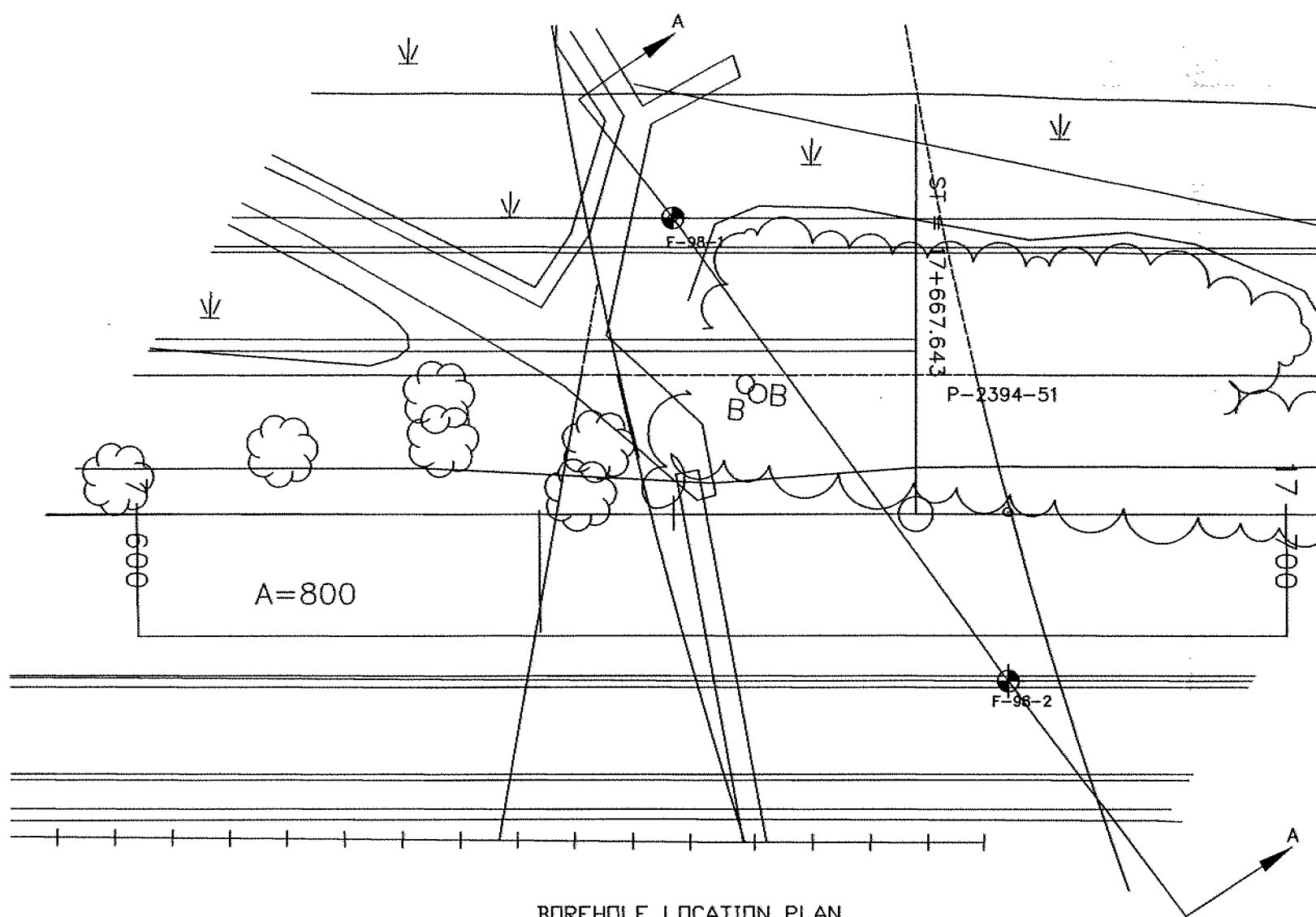
LEGEND



PLAN BASED ON PLAN E-625-11-
 SHT 1 of 2 - SUPPLIED BY CLIENT

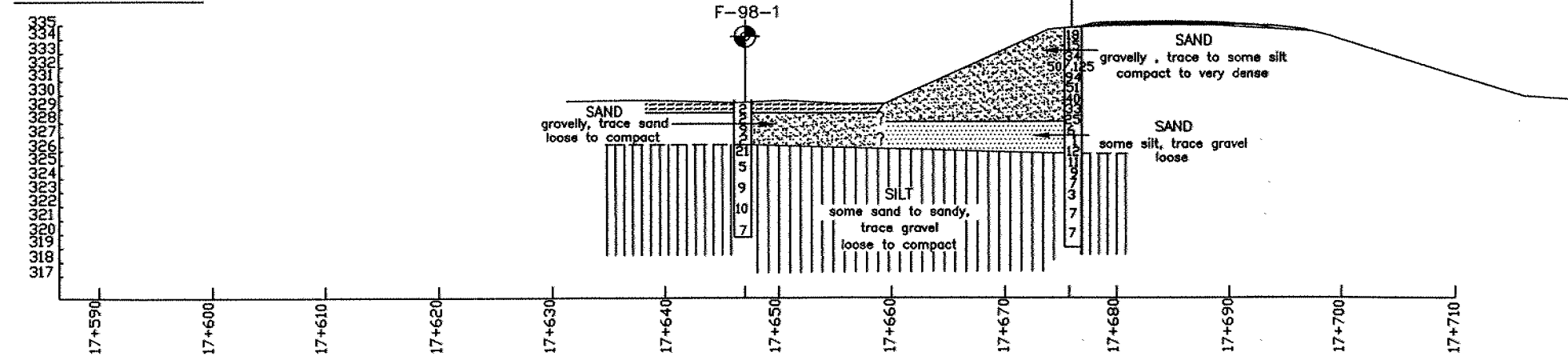
No	ELEV.	LOCATION	
		NORTHING	EASTING
F-98-1	328.5	5 040 485.1	320 428.4
F-98-2	334.6	5 040 526.5	320 439.9

19-1351-7f-01



BOREHOLE LOCATION PLAN

ELEVATION (metres)



SOIL PROFILE A ALONG HWY 11, SNOW MOBILE CROSSING
 SCALE - 1:500

APPENDIX A

BOREHOLE LOGS

- Symbols and Terms Used on Borehole Logs

- Unified Soil Classification

- Borehole Logs F-98-1 to F-98-2

SYMBOLS AND TERMS USED ON TEST HOLE LOGS

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	Less than 10	Less than 2
Soft	10 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 TEST HOLE LOGS

SYMBOLS FOR SAMPLE TYPE	Shelby Tube	A - Casing
<input type="checkbox"/>	SPT	Grab/Auger sample
<input checked="" type="checkbox"/>	No Recovery	Core

● MC - Moisture Content (% by Weight) as determined by sample]

W Water Level

C_{vane} Shear Strength Determination by Field Insitu Vane

C_{pen} Shear Strength Determination by Pocket Penetrometer

C_{lab} Shear Strength Determination using a Laboratory Vane Apparatus

C_u Undrained Shear Strength determined by Unconfined Compression Test

- (1) SPT Standard Penetration Test - refers to the number the blows from a 63.5kg hammer falling through 0.76m to advance a 60 degree truncated cone 0.3m.

UNIFIED SOILS CLASSIFICATION

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

RECORD OF BOREHOLE No F-98-1

1 OF 1

METRIC

W.P. 462-93-00 LOCATION SNOW MOBILE CROSSING - N 5 040 485.1 E 320 428.4 ORIGINATED BY MB
DIST 52 HWY 11 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY WM
DATUM GEODETIC DATE 98.07.16 - 98.07.17 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					
329.5	PEAT, sandy, trace rootlets, trace wood fibers, trace gravel, very soft, dark brown to black, moist		1	SS	2								
328.6			2	SS	2								
0.9			3	SS	9								
			4	SS	2								
326.3	SAND, gravelly, trace silt, very loose to compact, wet (Possible abandoned rail bed)		5A	SS	21								
3.2			5B	SS									
			6	SS	5								
			7	SS	9								
	SILT, trace to some sand, trace gravel, loose to compact, grey, wet		8	SS	10								
			9	SS	7								
319.7	END OF SAMPLING AT 9.75m.												
9.8													
	END OF BOREHOLE AT 15.2m.												

RECORD OF BOREHOLE No F-98-2

1 OF 2

METRIC

W.P. 462-93-00

LOCATION SNOW MOBILE CROSSING - N 5 040 526.5 E 320 439.9

ORIGINATED BY MB

DIST 52 HWY 11

BOREHOLE TYPE 210mm HOLLOW STEM AUGERS

COMPILED BY WM

DATUM GEODETIC

DATE 98.07.17 - 98.07.17

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			20	40	60	80	100		
334.6													
0.0	SAND, gravelly, trace to some silt, compact to dense, brown, moist: (FILL)		1	SS	18	334							
			2	SS	15								
			3	SS	34	333							
			4	SS	50/ .125	332							
	very dense		5	SS	94								
			6	SS	51	331							
			7	SS	40	330							
	becoming wet		8	SS	33	329							
			9	SS	25								
327.9						328							
6.7	SAND, trace gravel, some silt, very loose to compact, dark brown to black 50mm layer of organics, peat		10	SS	6								
			11	SS	1	327							
			12	SS	12	326							
325.6													
9.0	SILT, sandy, compact becoming loose, grey, wet		13	SS	11	325							
			14	SS	9								
			15	SS	7	324							
						323							
			16	SS	3	322							
			17	SS	7	321							
						320							

Continued Next Page

+ 3, x 3: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No F-98-2

2 OF 2

METRIC

W.P. 462-93-00 LOCATION SNOW MOBILE CROSSING - N 5 040 526.5 E 320 439.9 ORIGINATED BY MB
DIST 52 HWY 11 BOREHOLE TYPE 210mm HOLLOW STEM AUGERS COMPILED BY WM
DATUM GEODETTIC DATE 98.07.17 - 98.07.17 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT 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									
318.9			18	SS	7		319							0	0 16 80 4		
15.7	END OF BOREHOLE AT 15.70m.																

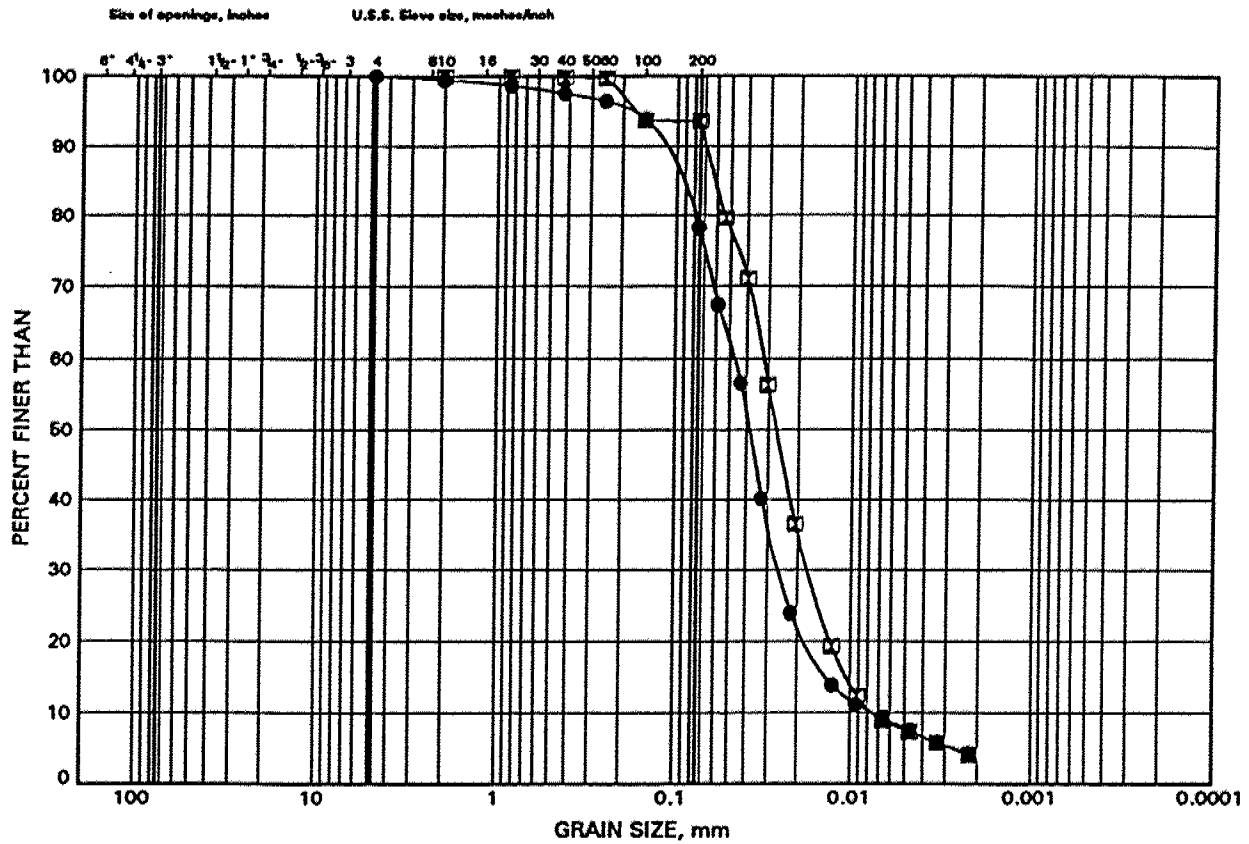
APPENDIX B

LABORATORY TEST RESULTS

- Figures B1 to B2 - Grain Size analyses

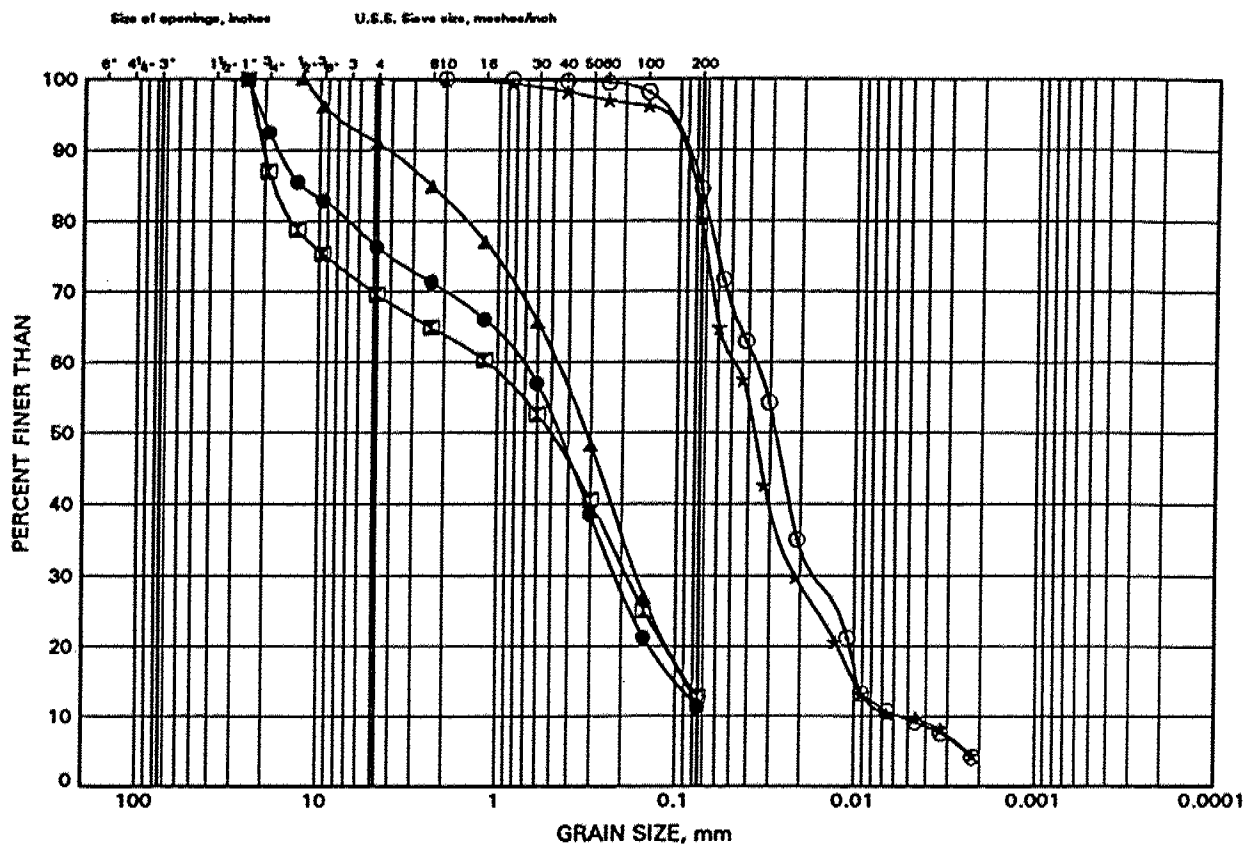
SNOWMOBILE CROSSING GRAIN SIZE DISTRIBUTION

FIGURE B1



SNOWMOBILE CROSSING GRAIN SIZE DISTRIBUTION

FIGURE B2



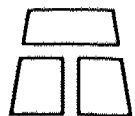
COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL BOREHOLE DEPTH (m) ELEVATION (m)

●	F-98-2	1.83	
◻	F-98-2	5.49	
▲	F-98-2	7.77	
★	F-98-2	9.30	
⊙	F-98-2	15.39	

Date October 1998

Project 462-93-00



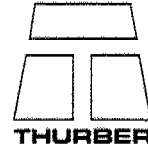
THURBER

Prep'd WM

Chkd. AEG

THURBER ENGINEERING LTD.

Suite 101, 170 Evans Avenue
ETOBICOKE, Ontario M9Z 5Y5
Phone (416) 503-3600
Fax (416) 503-3010



April 6, 1999

File: 19-1351-7f

McCormick Rankin Corporation
2655 North Sheridan Way
Mississauga, Ontario
L5K 2P8

APR - 9 1999

Attention: Mr. Reno Radolli, P.Eng.

**Response to MTO Comments of March 8, 1999
Final Foundation Design Report for
Snowmobile Crossing
Highway 11 Four Laning
6.7 km North of Hwy 60 Northerly 13 km
W.P. 462-93-00
District 52, Huntsville**

Dear Mr. Radolli:

We have reviewed the MTO memorandum dated March 8, 1999, presenting the Foundations Section's comments on our final report for the above referenced project.

1. Foundation Design

No comments for this site.

2. Dewatering

We understand that you have already dealt with the Dewatering NSSP.

3. Earth Pressure

In our report, we recommended that Granular B backfill be placed around the concrete box culvert as a cushion between the concrete and the rock fill. Accordingly, we provided earth pressure parameters for Granular B. A more general treatment of the problem would allow the contractor the option of substituting Granular A. The parameters for both materials are shown overleaf:

Continued....

Wall Type	Earth Pressure Coefficient (K)			
	OPSS Granular A $\phi' = 35^\circ$		OPSS Granular B $\phi' = 30^\circ$	
	Horizontal Ground Surface Behind Wall	Sloping Ground Surface (2H:1V)	Horizontal Ground Surface Behind Wall	Sloping Ground Surface (2H:1V)
Abutment Walls (Restrained Wall)	0.43	-	0.50	-
Wing Walls (Unrestrained Wall)	0.27	0.40	0.33	0.55

The unit weight to be used for granular A is 22.8 kN/m³

4. Culvert Inlet/Outlet Treatment

No comments for this site.

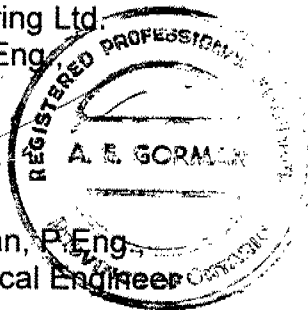
5. Culvert Invert

Inverts will be indicated on the final version of the drawings provided to you.

If you have any questions, do not hesitate to call our office.

Yours truly,
Thurber Engineering Ltd.
P.K. Chatterji, P.Eng.
Review Principal

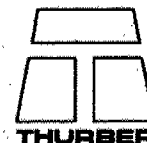

Alastair E. Gorman, P.Eng.
Senior Geotechnical Engineer



AEG/aeg/c:\191351\17\FREVLET02

THURBER ENGINEERING LTD.

Suite 101, 170 Evans Avenue
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Fax (416) 503-3010



February 3, 1999

File: 19-1351-7f

McCormick Rankin Corporation
2655 North Sheridan Way
Mississauga, Ontario
L5K 2P8

Attention: Mr. Reno Radolli, P.Eng.

**MTO Comments
Preliminary Foundation Design Report for
Proposed Snowmobile Crossing
Highway 1 Four Laning
6.7 km North of Hwy 60 Northerly 13 km
W.P. 462-93-00
District 52, Huntsville**

Dear Mr. Radolli:

On December 24, 1998, we received comments on the above referenced report directly from the MTO Foundations Group. A copy of their comments is attached and our responses are set out below.

1. Site #. We requested a site number from your Mr. Gord Firth who advised that site numbers generally are not used on structures with spans less than 6 m.
2. Foundation Drawing. The profile line has been added. The orientation of the drawing has not been changed in this instance.
3. The investigation for the snowmobile crossing was not included in the original terms of reference. The number of boreholes was negotiated with the MTO Northern Region Planning and Design Section.
4. Foundation Soil Disturbance. If excavation is carried out below the water table without prior unwatering, disturbance will occur. A warning has been added to the factual portion of the report and the recommendations section has been augmented.
5. Various Recommendations. Recommendations have been included regarding foundation bearing resistance, frost, bedding and erosion protection (if needed). Due to the method of construction, we do not consider that camber

Continued....

THURBER ENGINEERING

McCormick Rankin Corporation

- 2 -

February 4, 1999

will be an issue for this structure though some camber or gradient may be built in if the design requires that no water can pond in the structure. Similarly, we do not consider that special recommendations are required from a geotechnical perspective for the joints in the structure.

6. Earth Pressure. The different earth pressure cases have been clarified.
7. Blank
8. Proof Rolling. The possibility of using proof rolling has been specifically mentioned.
9. Thickness of native sand added
10. Thickness of silt added.
11. Preparation of Rock Fill Base. Further comments have been added regarding preparation of the rock fill base, incorporation of the bedding material and a geotextile separation layer.

We trust these changes meet the requirements of the Ministry. If you have any questions, do not hesitate to call our office.

Yours truly,

Thurber Engineering Ltd.
P.K. Chatterji, P.Eng.,
Review Principal

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

AEG/aeg/c:119/135/177/PREVLET.

MEMORANDUM



To: V. Minassian, P. Eng.
Senior Project Engineer
Planning and Design, Northern Region

March 8, 1999

From: Pavements and Foundations Section
Room 315, Central Bldg.

Tel: (416) 235-5267

Fax: (416) 235-5240

Re: Technical Review Package
WP 462-93-00
Hwy 11, From 6.7 km N of Hwy 60 Northerly 13.6 km
District 52, Huntsville

We have received the final Foundation Investigation and Design Reports for Jessop's Creek culvert and the snowmobile crossing submitted under your covering memorandum dated March 2, 1999. Review comments are contained in this memorandum.

Jessop's Creek Culvert

1. Foundation Design

The bearing resistance of the founding soil has not been included in the foundation report.

2. Dewatering

It is recommended that a Dewatering NSSP be included in the Contract that alerts the Contractor of the fact that the cohesionless soils present at the site are susceptible to conditions of unbalanced head and that the Contractor is responsible for rendering a stable excavation without inducing soil disturbance.

3. Earth Pressure

Granular backfill could be Granular "A" or Granular "B". Consequently, earth pressure design parameters should have been included in the report.

4. Culvert Inlet/Outlet Treatment

Recommendations for slope treatment (clay seal at inlet, filter material at outlet) should be included in the report.

5. Culvert Invert

The culvert invert elevation should be illustrated on the Borehole and Soil Stratigraphy Drawing.

Snowmobile Crossing

1. Comments 2, 3 and 5 above are also applicable to this structure.

We trust these comments are sufficient for your purposes. If you have any questions, please do not hesitate to contact this office.

T. Sangiuliano, P. Eng.
Foundation Engineer

for

D. Dundas, P. Eng.
Senior Foundation Engineer

cc. T. Kazmierowski

GEOCRES No. 51E-125DIST. 52 REGION W.P. No. 462-93-00CONT. No. W. O. No. STR. SITE No. HWY. No. 11LOCATION From 6.7 Km N of Hwy 60
W'ly 13.6 Km ; Little East River
No of PAGES - North & South

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. REMARKS:

**FINAL FOUNDATION DESIGN REPORT FOR
LITTLE EAST RIVER - SOUTH
HIGHWAY 11 FOUR LANING
6.7 km NORTH OF HWY 60 NORTHERLY 13 km
W.P. 462-93-00, SITE 42-233
DISTRICT 52, HUNTSVILLE**

Report

to

McCormick Rankin Corporation

Thurber Engineering Ltd.
170 Evans Avenue, Suite 101
Etobicoke, Ontario
M8Z 5Y6
Phone: (416) 503 3600
Fax: (416) 503 3010

Direction of fieldwork and engineering analysis by:

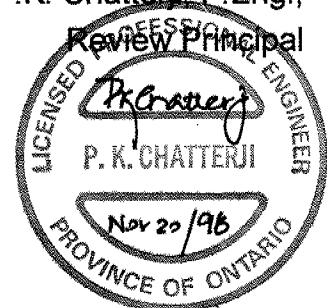
Alastair E. Gorman, P.Eng.,
Senior Geotechnical Engineer



Report reviewed by:

P.K. Chatterji, P.Eng.,

Review Principal



November 19, 1998

File: 19-1351-7B

aeg/AEG/C:19\1351\7B\FINALDES.WPD

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DRAWINGS

19-1351-7b-01	Borehole Location Plan
19-1351-7b-02	Soil Stratigraphy

FIGURES

Figure 1	Filter Layer Configuration
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APPENDICES

Appendix A	Borehole Logs
Appendix B	Laboratory Test Results
Appendix C	Grading of Sand for Backfill of Piles
Appendix D	Non-Standard Special Provisions

**FINAL FOUNDATION DESIGN REPORT FOR
LITTLE EAST RIVER - SOUTH
HIGHWAY 11 FOUR LANING
6.7 km NORTH OF HWY 60 NORTHERLY 13 km
W.P. 462-93-00, SITE 42-233
DISTRICT 52, HUNTSVILLE**

1. INTRODUCTION

This report presents the results of the foundation investigation and engineering analysis carried out by Thurber Engineering Ltd. (Thurber) at the site of the proposed bridge and approach fills to carry Highway 11 NBL across Little East River at Station 24+030, in the Town of Huntsville. The purpose of the investigation was to explore the subsurface and groundwater conditions at the site and based on the data obtained provide borehole logs, soil profile and a written description of the subsurface conditions. The purpose of the analysis of the data obtained during the investigation was to produce geotechnical recommendations for the design and construction of the structure foundations and associated earth works.

Thurber carried out the investigation as a sub-consultant to McCormick Rankin Corporation (MRC) under Ministry of Transportation (MTO) Agreement 9750 - 7424 - 5262.

2. SITE DESCRIPTION

2.1 Site Location

The subject site lies within the project limits of the four-laning of Highway 11 north of Huntsville and is located at the south crossing of the NBL over the Little East River. The site lies at Station 24+030, approximately, C/L Median Highway 11.

Locally, the site may be described as lying east of the existing Highway 11, in a flat, low-lying, treed area south of Stahl's Road. The Little East River meanders extensively across the area. Access to the site was from Highway 11 or Stahl's Road and then along the ROW of the NBL.

2.2 Physiography

Based on The Physiography of Southern Ontario, 3rd Edition, by Chapman and Putnam, the region surrounding the site consists of bedrock ridges with shallow overburden. The bedrock is undifferentiated igneous and metamorphic rock of early Precambrian age and is generally hard and massively jointed.

The Highway 11 corridor, however, lies in a long, narrow sand plain filling a deep valley within the region of shallow bedrock. The typical soils in the corridor consist of sand and silt, with some gravel deposited as glacial outwash or in localized glaciolacustrine environments.

The meandering creek (Little East River) and several wetlands in the area suggest poor drainage and a high groundwater table. Locally the ground is relatively flat, wet at the surface and supports typical vegetative cover for wet areas.

2.3 Site Layout

At this site, the NBL will cross the Little East River. Investigation was carried out on both banks of the creek to provide stratigraphic information related to the design and construction of structure foundations.

3. INVESTIGATION PROCEDURES

3.1 Field Investigation

Between July 13 and 16, 1998, a Nodwell track mounted auger and mud rotary drill rig was used on site for drilling, Standard Penetration Testing (SPT) (following the procedure outlined in ASTM D 1586) and dynamic cone penetration testing. One hole was drilled near each abutment and one at each approach fill, giving a total of four sampled boreholes. The approximate locations of the boreholes are shown on Drawing 19-1351-7b-01.

The holes were initially advanced using hollow stem augers and SPTs were carried out at intervals. Fine uniform sand and silt were encountered below a high water table which caused heaving of the soil into the hollow stem auger when the pilot bit was withdrawn in preparation for SPTs. The hollow stem augers were kept full of drilling mud at all times to counteract the effect of an unbalanced head of groundwater.

When it became apparent that auger drilling had reached its effective depth limit, mud rotary drilling was implemented for the balance of the depth of the hole. Investigation was carried out to a depth of approximately 49 m at the abutments and 6.7 m in the approach fill areas.

The borehole numbers and depths of sampling were as follows:

Borehole No.	Depth of Sampling (m)
B-98-1	6.7
B-98-2	48.8
B-98-3	48.8
B-98-4	6.7

Samples were recovered at intervals throughout the depths of the boreholes in conjunction with Standard Penetration Tests (SPT) (following the test procedure outlined in ASTM D 1586). Samples were generally recovered at intervals of 0.75 m in the upper 3.0 m and thereafter at intervals of 1.5 m to depths which vary between the holes. In the deep holes the sampling interval was increased to 3.0 m after stratigraphic continuity was established.

Dynamic cone penetration tests were conducted in, or adjacent to selected holes as follows

Borehole No.	Depth of Dynamic Cone Test
B-98-2	From a depth of 42.8 m to 48.8 m
B-98-3	From a depth of 43.1 m to 48.8

On completion of drilling and sampling, a standpipe piezometer was installed in Borehole B-98-2, at a depth of 10 m, to monitor the groundwater level.

The results of the drilling and sampling are summarized on the borehole logs in Appendix A.

Due to the suspected presence of artesian groundwater conditions, all boreholes were grouted on the completion of drilling and sampling, with the exception of the borehole interval where a piezometer was installed.

3.2 Laboratory Testing

Geotechnical laboratory testing consisted of natural moisture content determinations and visual classifications of all recovered samples. In

addition, grain size analyses and pH and sulphate content testing were conducted on selected samples. The results of the laboratory testing are presented on the borehole logs in Appendix A, and in Figures B1 to B5 in Appendix B. The results of the pH and sulphate content tests are presented in Table 1.

4. DESCRIPTION OF SUBSURFACE CONDITIONS

4.1 Subsurface Soil Conditions

Detailed descriptions of the subsoil conditions encountered in the boreholes are presented on the borehole logs in Appendix A. The stratigraphic profile inferred from the borehole information is shown on Drawing No. 19-1351-7b-02.

In general, the boreholes indicate a surface layer of peat underlain by silt which extends to depths in the order of 29 to 38 m where it is underlain by silt and sand. The silt and sand is underlain at about 41 m by sand in Borehole B-98-3.

Further description of these major soil units is provided in the following sections. The soils encountered all appeared to be lacustrine or fine outwash deposits and no evidence of boulders was found. However, the possibility of encountering boulders at random locations in the deposits during construction must be recognized.

Grain size distributions of selected samples are shown in Figures B1 to B5 in Appendix B.

Peat and Organic Silt

All boreholes encountered a surface layer of peat and organic silt interbedded with inorganic silt layers in varying thicknesses. The peat is

fibrous and contains numerous roots and the organic silt contains a high proportion of amorphous organic matter. The colour of the peat ranges from brown to dark brown to black while the inorganic silt layers are grey.. Measured moisture contents lie in the range of 30 to 189%. For convenience at this site, these layered materials are collectively referred to as "peat" and the interpreted depths of peat at the boreholes are as follows:

Borehole No.	Depth of Peat (m)
B-98-1	4.0
B-98-2	3.1
B-98-3	2.5
B-98-4	2.1

Actual depths of peat to be stripped may vary from those interpreted at the borehole locations.

Silt

Based on Boreholes B-98-2 and B-98-3, the silt layer extends to depths of approximately 38 to 29 m below existing ground level, respectively, or to approximate Elevations 280.4 and 289.2.

The silt contains trace to some sand and trace clay and thin clay seams were noted throughout the deposit. In both Borehole B-98-2 and B-98-3, between about 5.2 and 6.6 m depth, the silt is described as clayey and contained noticeably more frequent clay seams. Boreholes B-98-1 and B-98-4 encountered the same clayey material at an approximate depth of 5.2 m and terminated in it at depths of 6.7 m.

Based on the SPT values, the density of the silt layer generally ranges

from loose to compact with N values ranging from 5 to 28. Between 17 and 23 m depths in Borehole B-98-3, SPT values of 32 and 38 were recorded, indicating dense conditions. The measured natural moisture contents ranged from 18 to 31%.

Silt and Sand

Below the silt layer, Boreholes B-98-2 and B-98-3 encountered a deposit of silt and sand. In Borehole B-98-2, the silt and sand layer was encountered at about Elevation 280.4 and sampling terminated in this deposit at Elevation 275.3. In Borehole B-98-3, the silt and sand layer extended from Elevation 289.2 to Elevation 277.2. The deposit shows faint layering, is grey and wet.

Based on the recorded SPT values, the silt and sand is compact to dense with N values ranging from 15 to 35. The measured natural moisture contents range from 18 to 25%.

Borehole B-98-3 fully penetrated the silt and sand layer into the underlying sand.

Sand

Below Elevation 277.2, Borehole B-98-3 encountered a sand deposit which extended to the termination of the borehole sampling at 274.9. The sand is fine grained and uniform with trace silt. The grain size distributions for a selected sample is shown in Figure B5. The sand deposit shows faint layering and is grey and wet.

Based on the recorded SPT value of 52 and the dynamic cone penetration test values below that, the sand is in a very dense state. The measured natural moisture content was 23%.

4.2 Groundwater

During drilling, the groundwater level in the open boreholes was recorded as follows

Boreholes B-98-1 and B-98-2	1.5 m
Boreholes B-98-3 and B-98-4	0.9 m

The following groundwater levels were recorded in the piezometer installed in Borehole B-98-2:

Date	Height of Water (above existing ground surface)
July 15, 1998	0.1 m (artesian)
July 31, 1998	0.6 m (artesian)
September 8, 1998	0.4 m (artesian)
October 27, 1998	0.9 m (artesian)

Note: On October 27, 1998, the creek level was estimated to be approximately 1.0 m higher than it was in July and September.

Based on this data and the close proximity of the creek, it is concluded that there is a free groundwater level in the soils close to the ground surface. The level is expected to fluctuate with the creek level.

The data from the piezometer installed at 10 m in Borehole B-98-2, however, indicates a small artesian head (0.6 to 0.9 m) with respect to the ground surface. Examination of the borehole data suggests that the clayey silt layer between depths of 5.2 and 6.6 m acts as an aquiclude or aquitard, and is confining the groundwater in the lower levels of the silt deposit. The

data obtained to date indicates that fluctuations should be expected in the artesian head and in the near surface groundwater table.

5. RECOMMENDATIONS FOR STRUCTURE FOUNDATIONS

5.1 Type of Structure

The proposed structure will be a single-span bridge carrying Highway 11 NBL over the Little East River. Geotechnical recommendations are required for the design of foundations at the north and south abutments.

The span of the proposed structure will be 18.3 m and it is understood that an integral abutment design is preferred, if the foundation conditions are suitable.

Geotechnical recommendations are also required for design and construction of the 4 m high approach fills immediately adjacent to the bridge. It is understood that the main embankment will be constructed of rock fill and that a rock fill shell is desirable for the side and forward slopes at the structure.

5.2 Foundation Soil Conditions

The factual description of the foundation soils is presented in Section 4 of this report. A discussion of the soil conditions is presented below.

The foundation conditions encountered in the Boreholes B-98-2 and B-98-3 consist of a deep deposit of silt underlain by silt and sand, and at greater depth by sand. These soils are considered suitable for the design of an integral abutment bridge with each abutment supported on a single row of H-piles driven to sufficient depth to achieve fixity well below the depth required to provide for movement of the abutment.

At this site, the presence of an artesian (above ground) head in the groundwater must be taken into account and appropriate measures implemented to minimize the risk of piping of soils occurring up the sides of

the piles with accompanying loss of load bearing capacity. The analysis of pile capacity has taken account of the risk of some loss of adhesion due to artesian flow.

The surficial layer of peat should be stripped as discussed in Section 7, Approach Embankment Design.

5.3 Piled Foundations

5.3.1 Axial Capacity

The foundations of the abutments should be supported on HP 310X110 piles.

Static analysis of the axial resistance has been carried out for the HP 310X110 pile in accordance with the OHBDC, using the soil parameters described in the Foundation Investigation Report. The analysis was conducted assuming that the piles are friction piles, using the full steel surface but no end plug. Partial loss of adhesion due to the upward flow caused by artesian pressure has been included in the analysis.

In the analysis of the vertical geotechnical resistance developed by the piles at the abutments, the following assumptions were made:

- the underside of the abutment stem will be approximately at Elevation 316.8 at the south and 316.7 at the north
- the surficial layer of peat will be completely stripped and replaced by fill
- the pile was assumed not to develop any vertical resistance above Elevation 309.5

For the abutments, analysis indicated that an HP 310X110 pile driven to a

total depth of 43.0 m below the base of the abutment stem would have a factored geotechnical resistance at ULS of 1,000 kN and an SLS resistance of 750 kN. This corresponds to a pile tip elevation of approximately 273.5.

The geotechnical resistance should be checked against the structural capacity of the pile.

The factored ULS has been calculated from static analysis using a resistance factor of 0.4. However, in view of the loose to compact soils and the artesian groundwater conditions encountered at the site, and the lack of pile construction experience in these difficult soil conditions, it is recommended that a static pile load test be conducted at the outset of construction. A resistance factor of 0.6 would be applied to the results of the load test which would then be used to confirm the design or to adjust the required driven length of pile.

5.3.2 Lateral Resistance

The lateral resistance of the HP 310X110 pile was assessed in accordance with Clause 6-9.8.1. Assuming flexure in the weak direction the assessed values are 100 kN at ULS and 27 kN at SLS.

The value of the coefficient of horizontal subgrade reaction, k_s (MN/m³), for a pile is given by the equation:

$$k_s = n_h Z/b$$

Z = depth below ground surface (m)

b = pile width(m)

For this site, taking account of the high groundwater level, the recommended values for the constant n_h (MN/m³) are:

Condition	Value of n_h (MN/m ³)
Granular B, above the water table	11.0
Native soil, below the water table	4.5

At this site, it is anticipated that the piles will be resisted only by native soil, or fill with comparable characteristics.

5.3.3 Pile Driving

The piles should be provided with driving shoes in accordance with OPSP 3301.00.

Pile driving should be carefully monitored and controlled employing the Hiley Dynamic Pile Driving Formula in accordance with MTO Standards SS 103-10 or SS 103-11 and assuming an ultimate resistance of 2,000 kN.

The pile driving should be carried out using a hammer delivering at least 50 kJ per blow.

During the driving process, piles which have already been driven should be monitored to determine if they are heaving due to the effects of driving adjacent piles. If this phenomenon occurs, the affected piles must be re-driven.

5.3.4 Pile Driving Note

The pile driving note to be added to the drawings should be Note 7 in Clause 2.5.11 of the Structural Manual. The ultimate resistance to be used

is 2,000 kN. The piles must be driven at least below Elevation 273.5 and not below Elevation 265.0 without approval of the Engineer..

5.3.5 Pile Installation Details

The recommendations contained in this section require that prior dewatering of the site be carried out in accordance with the requirements set out in Section 8 of this report.

It is recommended that special precautions be taken to minimize, as far as practical, the danger of the artesian head in the groundwater causing piping up the sides of the piles and at the same time fulfil the integral abutment requirement for a flexible foundation.

There is a concern that after the driven pile penetrates the aquitard at 5.2 to 6.6 m below existing ground level the underlying artesian head may cause continuing seepage up the side of the pile. If this seepage flow is able to carry fines with it, there will be a loss of ground and accompanying loss of vertical and horizontal pile capacity. To reduce the risk of piping, it is recommended that:

- a. a clay seal be constructed around each pile within the thickness of the aquitard
- b. the backfill around the pile be designed to act as a filter to allow any future upward seepage to dissipate without removal of a significant quantity of fines.

The design of the above items must recognize that a flexible foundation must be maintained for the integral abutment design.

Clay Seal

To achieve a seal around the pile in the top of the aquitard, the required 600 mm CSP should extend from the required top elevation down to Elevation 312.5. A hole large enough to accept a 600 mm CSP and provide an annular space (say a 1.0 m \pm diameter hole) should be drilled to Elevation 312.5. The CSP should be placed in this hole and a clay plug formed by placing granular bentonite, such as "Holeplug" by Baroid Drilling Fluids, Inc. The bentonite should be placed to a depth of 1.0 m inside and at the bottom of the CSP and should also completely fill the annular space outside the CSP for a depth of 1.0 m or up to the base of the excavation, whichever is less. Above the clay plug, the CSP should be filled with loose sand meeting the grading requirements shown in Appendix C. The sand column should be placed within two hours of placing the bentonite, and in any case before the end of the day, and must extend up to the underside of the abutment stem.

Filter

A filter should be constructed completely around all the CSPs installed at the abutment and should completely cover the stripped surface within the sheet piling. To construct the filter, the existing soil should be excavated to Elevation 315.5 and finished with a level base. If stripping the peat results in a lower elevation, a level base should be prepared at that level. Following preparation of the base, the filter should be constructed by placing a layer of fill at least 1.0 m thick and meeting OPSS 1002 Table 1 "Gradation Requirements for Fine Aggregates" (filter sand). The fill should be placed in lifts not exceeding 300 mm in thickness and be compacted to at least 95% Standard proctor maximum dry density (SPMDD).

The foregoing points are illustrated in Figure 1, based on the assumed general arrangement at the south abutment.

5.4 Construction Sequence

The Contractor's plan for the sequence of construction events must address the following important elements:

1. Establish a satisfactory working platform.
2. Install any sheeting or shoring required.
3. Lower the water table to at least 1.0 m below the lowest level of excavation as described in Section 8 of this report.
4. Excavate all organic or otherwise deleterious material within the required area of excavation.
5. From the base of the excavation, auger to Elevation 312.5 a hole approximately 1,000 mm in diameter.
6. Inside this 1,000 mm hole, place a 500 mm layer of granular bentonite such as "Holeplug" by Baroid Drilling Fluids, Inc. and tamp lightly.
7. Seat the 600 mm CSP in the granular bentonite and support.
8. Place a further 500 mm of granular bentonite inside the 600 mm CSP and 500 mm in the annular space between the 600 mm CSP and the side of the hole. Fill the balance of the augered hole to the base of general excavation with filter sand.
9. Fill the 600 mm CSP with sand as specified in Appendix C.

NOTE. Steps 5 through 9 to be repeated for all piles.

10. Construct the specified sand filter around the piles to a thickness of at least 1.0 m or to greater thickness if this facilitates the construction sequence.
11. Proceed with the remaining backfill and pile driving in the most convenient order.
12. Maintain the dewatering system in operation until the later date of either completion of pile driving plus two days or completion of filling to at least Elevation 316.0.

6. EARTH PRESSURE

The lateral earth pressures to be used in design should be computed in accordance with Section 6-7 of the OHBDC .

Granular backfill should be placed behind the integral abutment walls and wing walls to conform to the minimum requirements set out in Section 7 of this report. The granular backfill should conform to Ontario Provincial Standard Specifications (OPSS) 1010 for Granular B, Type I. The fill should be placed in accordance with OPSS 501. A perforated subdrain should be installed behind the base of the walls to maintain the granular fill in a drained condition. The subdrain should be provided with a positive outlet to the highway drainage system.

For the above backfill and drainage conditions, the abutment walls and wing walls may be designed based on the following unfactored earth pressure distributions:

$$P_h = K \gamma h$$

where;

K = earth pressure coefficient, use value from table on following page

γ = unit weight of soil, = 21.2 kN/m³ for Granular B

h = depth below top of wall, m

Wall Type	Earth Pressure Coefficient (K)	
	OPSS Granular B $\phi' = 30^\circ$	
	Horizontal Ground Surface Behind Wall	Sloping Ground Surface (2H:1V)
Abutment Walls (Restrained Wall)	0.50	-
Wing Walls (Unrestrained Wall)	0.33	0.55

If an integral abutment design is used, the abutments will be cast integrally with the deck and therefore the abutment walls should be treated as restrained. If the wing walls will not be connected to the abutments and therefore will be able to accommodate some rotation they may be treated as unrestrained. The above also assumes a horizontal ground surface behind the abutment walls. If concrete approach slabs are not provided, an additional load equivalent to 600 mm of fill should be superimposed on the wall loadings to account for traffic surcharge loading.

An additional lateral pressure of 16 kPa should be added to account for compaction induced forces. The additional pressure must be computed in accordance with Clause 6-7.4.3 of the OHBDC.

7. APPROACH EMBANKMENT DESIGN

Based on Plan E-625-11-7, Sheet 2 of 2, the driving lanes of Highway 11 will be constructed on embankments with a finished grade up to 4 m above existing ground level. These embankments will be constructed across a swampy area with potential artesian groundwater conditions.

As shown on the borehole logs, a surface layer of peat was encountered at the boreholes which measured 2.1 to 4.0 m in thickness. All peat and soft soils should be removed under the approach embankment. It is understood that a geotechnical report has been produced by others which recommends removal of the peat and the use of rock fill for the main embankment.

Rock fill is not considered acceptable in a zone immediately behind the abutment and wing walls of an integral abutment bridge. A zone of granular backfill is required behind the abutment and around the wing walls to provide for the movement requirements of the integral abutment. In profile, this zone of backfill should commence 1.2 m behind the base of the abutment stem and slope upwards at 2H:1V. The fill within this zone should consist of Granular B, as described in Section 6 of this report, Earth Pressure.

Approach embankments constructed of Granular "B" may have side slopes no steeper than 2:1.

If the approach embankment slopes are constructed of rock fill and all peat and soft soils are removed from the embankment area, side slopes of 1.25H:1V are expected to be stable and should be designed following OPSS 203.010.

Embankment fill should be placed in appropriate lift thicknesses and be compacted in accordance with OPSS 501.

The design of the approach embankments and the materials to be used should be coordinated with the recommendations in the Pavement Design Report.

8. EXCAVATION AND GROUNDWATER CONTROL

The groundwater level established by piezometer readings is approximately 0.9 m above existing ground surface, or at Elevation 319.4.

Various elements of construction discussed earlier in this report require work to

be carried out in dry conditions and with control of the underlying artesian pressure. Given the soil and groundwater conditions encountered at this site, it is recommended that the necessary dewatering be achieved by:

- forming a sheeted excavation with interlocking steel sheet piling driven at least 2.0 m beyond the limits of the main piles
- driving the sheeting to Elevation 312.5 to obtain a seal in the inferred aquitard, but no deeper so as to avoid unnecessary penetration of the aquitard
- depressing the water table within the sheeted area to at least 1.0 m below the projected base of the excavation, taking account of the reported artesian head (boiling and/or base heave may occur if proper dewatering is not achieved)

Given the fine grained soil conditions encountered at this site, it is considered that vacuum assisted wellpoints will be required to achieve the necessary dewatering.

The sheeting and dewatering must be maintained at least until backfill within the sheeting is up to Elevation 316.0. At that stage the dewatering system may be decommissioned. As part of the decommissioning, any penetration of the aquitard must be grouted or otherwise sealed to prevent seepage.

All aspects of the dewatering must be designed by a specialist in this field.

From a geotechnical standpoint, the sheeting should be left in place and cut off about ground level to assist in the control of any seepage that may occur.

All excavations must be carried out in accordance with the Occupational Health and Safety Act and the sheeting and dewatering must be designed by a Professional Engineer specializing in that field.

No permanent groundwater control measures are required for the proposed piled foundation.

Appendix D contains a suggested NSSP which should be included in the contract to alert the Contractor to the existence of a high surface groundwater table and an artesian condition at relatively shallow depth. The NSSP also alerts the Contractor to the necessity of controlling the groundwater levels and carrying out certain aspects of the construction in dry conditions.

9. FROST PROTECTION

The design depth of frost penetration for this project is 1.8 m. All pile caps and footings designed for this site must be provided with a minimum depth of soil cover of 1.8 m to protect against the penetration of frost below the foundation elements.

10. CONSTRUCTION CONCERNS

The main construction concerns relate to:

- maintenance of a clay seal around the driven piles at the top of the aquitard to minimize upward seepage
- proper construction of a filter layer around the piles to limit the loss of fines in the event that seepage flow up the side of the pile does occur
- proper dewatering of the site so that the work around the abutment can proceed as recommended

11. CONSTRUCTION INSPECTION AND MONITORING

During construction, all foundation installation, excavation and approach embankment construction activities should be monitored by geotechnical personnel to ensure that the foundation recommendations and design are being correctly implemented and that the soil conditions encountered do not differ materially from the interpretation used in this report.

STATEMENT OF GENERAL CONDITIONS

1. STANDARD OF CARE

This study and Report have been prepared in accordance with generally accepted engineering or environmental consulting practices in this area. No other warranty, expressed or implied, is made.

2. COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report which is of a summary nature and is not intended to stand alone without reference to the instructions given to us by the Client, communications between us and the Client, and to any other reports, writings, proposals or documents prepared by us for the Client relative to the specific site described herein, all of which constitute the Report.

IN ORDER TO PROPERLY UNDERSTAND THE SUGGESTIONS, RECOMMENDATIONS AND OPINIONS EXPRESSED HEREIN, REFERENCE MUST BE MADE TO THE WHOLE OF THE REPORT. WE CANNOT BE RESPONSIBLE FOR USE BY ANY PARTY OF PORTIONS OF THE REPORT WITHOUT REFERENCE TO THE WHOLE REPORT.

3. BASIS OF REPORT

The Report has been prepared for the specific site, development, design objectives and purpose that were described to us by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the document are only valid to the extent that there has been no material alteration to or variation from any of the said descriptions provided to us unless we are specifically requested by the Client to review and revise the Report in light of such alteration or variation.

4. USE OF THE REPORT

The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client. NO OTHER PARTY MAY USE OR RELY UPON THE REPORT OR ANY PORTION THEREOF WITHOUT OUR WRITTEN CONSENT. WE WILL CONSENT TO ANY REASONABLE REQUEST BY THE CLIENT TO APPROVE THE USE OF THIS REPORT BY OTHER PARTIES AS "APPROVED USERS". The contents of the Report remain our copyright property and we authorize only the Client and Approved Users to make copies of the Report only in such quantities as are reasonably necessary for the use of the Report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make the Report, or any portion thereof, available to any party without our written permission. Any use which a third party makes of the Report, or any portion of the Report, are the sole responsibility of such third parties. We accept no responsibility for damages suffered by any third party resulting from unauthorized use of the Report.

5. INTERPRETATION OF THE REPORT

- a) Nature and Exactness of Soil and Contaminant Description: Classification and identification of soils, rocks, geological units, contaminant materials and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgemental in nature and even comprehensive sampling and testing programs, implemented with the appropriate equipment by experienced personnel, may fail to locate some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and all persons making use of such documents or records should be aware of, and accept, this risk. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. Where special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.

(see over...)

INTERPRETATION OF THE REPORT *(continued)*

- b) **Reliance on Provided Information:** The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to us. We have relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, we cannot accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of misstatements, omissions, misrepresentations, or fraudulent acts of persons providing information.

6. RISK LIMITATION

Geotechnical engineering and environmental consulting projects often have the potential to encounter pollutants or hazardous substances and the potential to cause an accidental release of those substances. In consideration of the provision of the services by us, which are for the Client's benefit, the Client agrees to hold harmless and to indemnify and defend us and our directors, officers, servants, agents, employees, workmen and contractors (hereinafter referred to as the "Company") from and against any and all claims, losses, damages, demands, disputes, liability and legal investigative costs of defence, whether for personal injury including death, or any other loss whatsoever, regardless of any action or omission on the part of the Company, that result from an accidental release of pollutants or hazardous substances occurring as a result of carrying out this Project. This indemnification shall extend to all Claims brought or threatened against the Company under any federal or provincial statute as a result of conducting work on this Project. In addition to the above indemnification, the Client further agrees not to bring any claims against the Company in connection with any of the aforementioned causes.

7. SERVICES OF SUBCONSULTANTS AND CONTRACTORS

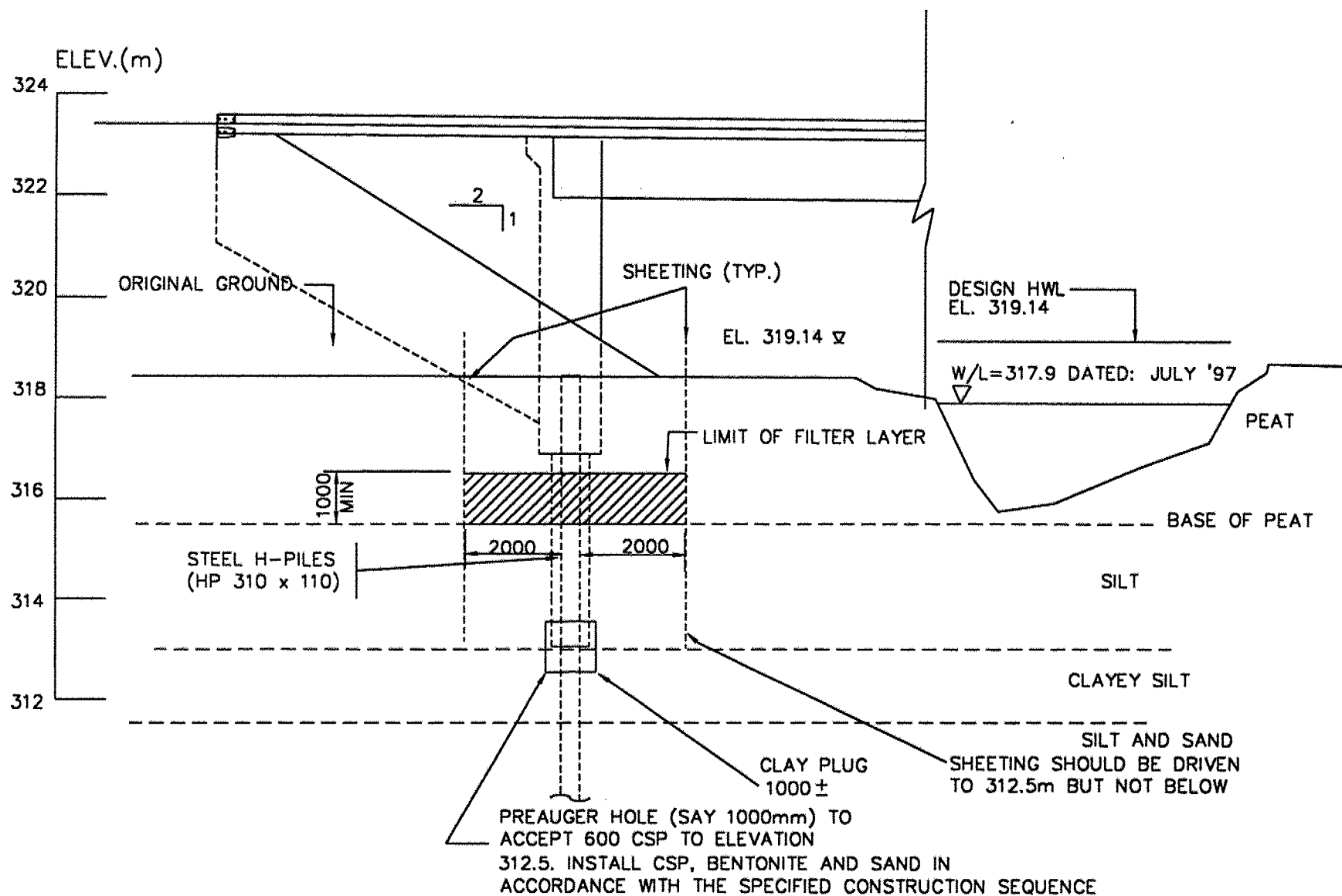
The conduct of engineering and environmental studies frequently requires hiring the services of individuals and companies with special expertise and/or services which we do not provide. We may arrange the hiring of these services as a convenience to our Clients. As these services are for the Clients' benefit, the Client agrees to hold the Company harmless and to indemnify and defend us from and against all claims arising through such hirings to the extent that the Client would incur had he hired those services directly. This includes responsibility for payment for services rendered and pursuit of damages for errors, omissions or negligence by those parties in carrying out their work. In particular, these conditions apply to the use of drilling, excavation and laboratory testing services.

8. CONTROL OF WORK AND JOBSITE SAFETY

We are responsible only for the activities of our employees on the jobsite. The presence of our personnel on the site shall not be construed in any way to relieve the Client or any contractors on site from their responsibilities for site safety. The Client acknowledges that he, his representatives, contractors or others retain control of the site and that we never occupy a position of control of the site. The Client undertakes to inform us of all hazardous conditions, or other relevant conditions of which the Client is aware. The Client also recognizes that our activities may uncover previously unknown hazardous conditions or materials and that such a discovery may result in the necessity to undertake emergency procedures to protect our employees as well as the public at large and the environment in general. These procedures may well involve additional costs outside of any budgets previously agreed to. The Client agrees to pay us for any expenses incurred as the result of such discoveries and to compensate us through payment of additional fees and expenses for time spent by us to deal with the consequences of such discoveries. The Client also acknowledges that in some cases the discovery of hazardous conditions and materials will require that certain regulatory bodies be informed and the Client agrees that notification to such bodies by us will not be a cause of action or dispute.

9. INDEPENDENT JUDGEMENTS OF CLIENT

The information, interpretations and conclusions in the Report are based on our interpretation of conditions revealed through limited investigation conducted within a defined scope of services. We cannot accept responsibility for independent conclusions, interpretations, interpolations and/or decisions of the Client, or others who may come into possession of the Report, or any part thereof, which may be based on information contained in the Report. This restriction of liability includes decisions made to either purchase or sell land.



LEGEND



FILTER LAYER

FILTER LAYER CONFIGURATION (SCHEMATIC)

SHEET

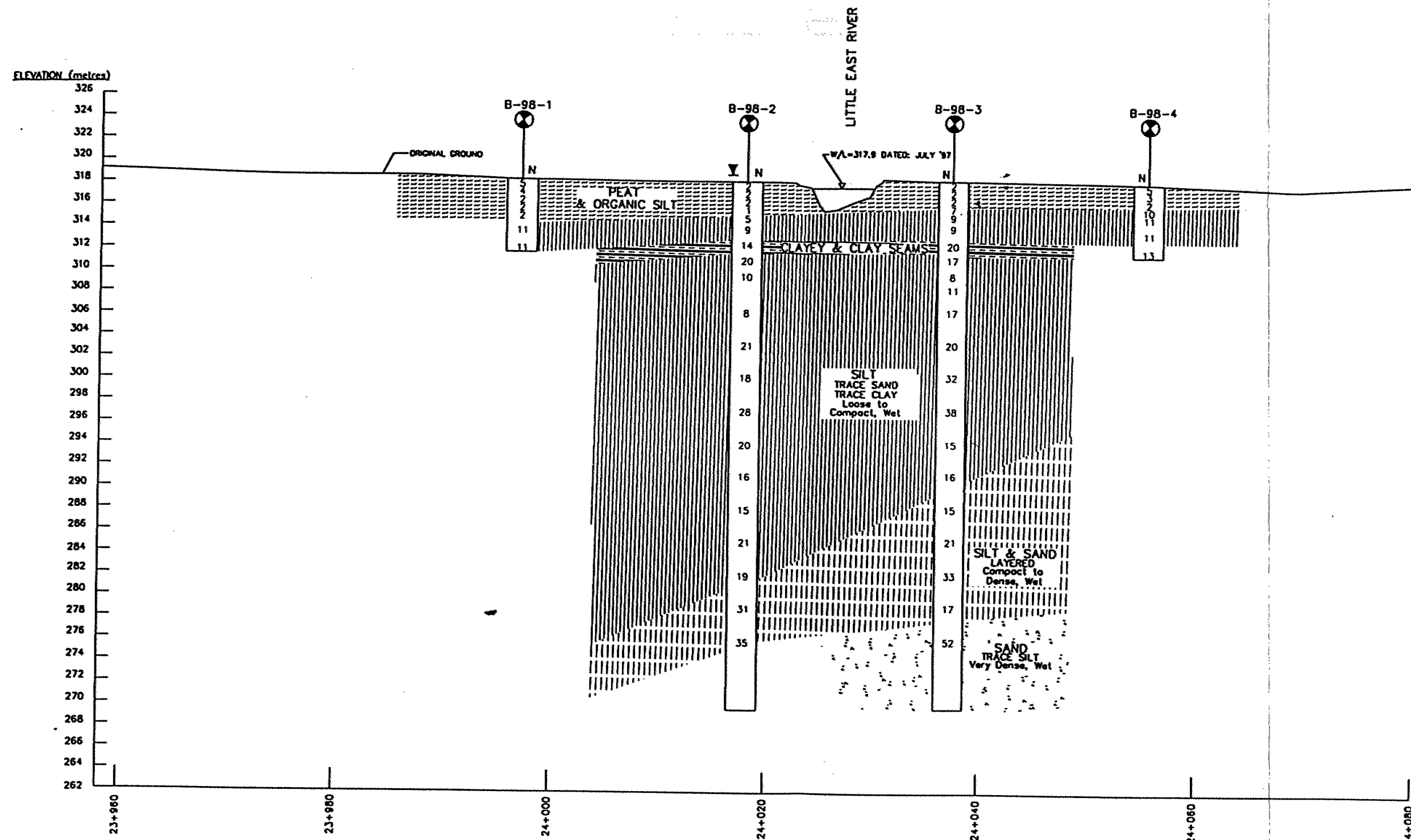
A horizontal number line with tick marks at 1500m, 0, 3000, and 6000m. The segment between 1500m and 3000m is shaded with a light blue pattern.

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES
UNLESS OTHERWISE SHOWN

No	ELEV.	LOCATION	
		NORTHING	EASTING
8-98-1	318.8	5 032 015.0	325 272.7
8-98-2	318.5	5 032 035.9	325 268.4
8-98-3	318.2	5 032 055.2	325 268.0
8-98-4	318.6	5 032 073.5	325 271.0

LOCATION OF BOREHOLE

B-98-1



SOIL PROFILE ALONG HWY 11 NBL OVER LITTLE EAST RIVER

SCALE 1:400

- LEGEND**
- B-98-1
● LOCATION OF BOREHOLE
- Y WATER LEVEL, JULY 31, 1998
- N BLOWS/0.3m (Std Pen Test)

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

PLAN BASED ON PLAN E-625-11-7
SHT 2 of 2 - SUPPLIED BY CLIENT

19-1351-7b-02

APPENDIX A

BOREHOLE LOGS

- Symbols and Terms Used on Borehole Logs
- Unified Soil Classification
- Borehole Logs B-98-1 to B-98-4



RECORD OF BOREHOLE No B-98-1

1 OF 1

METRIC

W.P. 462-93-00 LOCATION LITTLE EAST RIVER - SOUTH, N 5 032 015.0 E 325 272.7 ORIGINATED BY GA
DIST 52 HWY 11 BOREHOLE TYPE 210mm Hollow Stem Augers with Mud COMPILED BY WM
DATUM GEODETIC DATE 98.07.16 - 98.07.16 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										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
318.8 0.0	PEAT & ORGANIC SILT, very loose to loose, layered peat, organic silt and inorganic silt, brown, with grey silt, wet: (PT/OH)		1	SS	5										87.040					
			2	SS	4		318													
			3	SS	2		317										40.370			
			4	SS	2		316										47.060			
			5	SS	2												44.240			
314.8 4.0	SILT, compact, some sand, some thin clay seams, layered, grey, wet: (ML)		6	SS	11		315									0 10				
							314													
			7	SS	11		313													
312.1 6.7	END OF BOREHOLE AT 6.7m. BOREHOLE OPEN TO 6.1m. BOREHOLE BACKFILLED WITH GROUT. Water level at 1.5m on completion.																			

RECORD OF BOREHOLE No B-98-2

1 OF 4

METRIC

W.P. 462-93-00 LOCATION LITTLE EAST RIVER - SOUTH, N 5 032 035.9 E 325 268.4 ORIGINATED BY GA
DIST 52 HWY 11 BOREHOLE TYPE 210mm Hollow Stem Augers/Mud Rotary COMPILED BY WM
DATUM GEODETIC DATE 98.07.15 - 98.07.15 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	20 40 60 80 100	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	
318.5 0.0	PEAT & ORGANIC SILT, very loose, layered peat, organic silt and sandy silt, brown, grey silt seams, wet: (PT/OH)		1	SS	2		318					94.840	
			2	SS	2		317					46.560	
			3	SS	2		316					60.240	
			4	SS	1		315					71.480	
315.4 3.1	SILT, loose, some sand, layered, grey, wet		5	SS	5		314						0 11
			6	SS	9		313						
313.3 5.2	clayey and clay seams: (CL-ML)		7	SS	14		312						0 4 79 18
311.9 6.6	becoming sandy, compact, some loose seams, grey, wet		8	SS	20		311						
			9	SS	10		310						
			10	SS	8		309						
							308						
							307						
							306						
							305						
							304						
303.5													

Continued Next Page

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

RECORD OF BOREHOLE No B-98-2

2 OF 4

METRIC

W.P. 462-93-00 LOCATION LITTLE EAST RIVER - SOUTH, N 5 032 035.9 E 325 268.4 ORIGINATED BY GA
 DIST 52 HWY 11 BOREHOLE TYPE 210mm Hollow Stem Augers/Mud Rotary COMPILED BY WM
 DATUM GEODETIC DATE 98.07.15 - 98.07.15 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 Y kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					
15.0	SILT - continues as above		11	SS	21	303							
						302							
						301							
			12	SS	18	300							
						299							
						298							
			13	SS	28	297							
						296							
						295							
			14	SS	20	294							
						293							
						292							
			15	SS	16	291							
						290							
						289							

288.5

Continued Next Page

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

RECORD OF BOREHOLE No B-98-2

3 OF 4

METRIC

W.P. 462-93-00 LOCATION LITTLE EAST RIVER - SOUTH, N 5 032 035.9 E 325 268.4 ORIGINATED BY GA
DIST 52 HWY 11 BOREHOLE TYPE 210mm Hollow Stem Augers/Mud Rotary COMPILED BY WM
DATUM GEODETIC DATE 98.07.15 - 98.07.15 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 x LAB VANE																
								20 40 60 80 100					WATER CONTENT (%)											
													10 20 30											
													w p w w L											
													NATURAL MOISTURE CONTENT											
													LIQUID LIMIT											
30.0								SILT - continues as above																
			16	SS	15		288							0 11										
							287																	
							286																	
							285																	
			17	SS	21		284																	
							283																	
							282																	
			18	SS	19		281																	
							280							0 25										
280.4							279																	
38.1	SILT and SAND, dense, some layering, grey, wet						278																	
			19	SS	31		277																	
							276																	
							275																	
275.3			20	SS	35		274							0 40										
43.3	END OF SAMPLING AT 43.28m. Water level at 1.5m on completion.																							

Continued Next Page

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

RECORD OF BOREHOLE No B-98-2

4 OF 4

METRIC

W.P. 462-93-00 LOCATION LITTLE EAST RIVER - SOUTH, N 5 032 035.9 E 325 268.4 ORIGINATED BY GA
DIST 52 HWY 11 BOREHOLE TYPE 210mm Hollow Stem Augers/Mud Rotary COMPILED BY WM
DATUM GEODETIC DATE 98.07.15 - 98.07.15 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					
273													
272													
271													
270													
269.8													
48.8	END OF BOREHOLE AT 48.8m.												
	Piezometer Readings: Date Depth (m) 15/07/98 0.1 (artesian) 31/07/98 0.6 (artesian) 08/09/98 0.4 (artesian) 27/10/98 0.9 (artesian)												

RECORD OF BOREHOLE No B-98-3

2 OF 4

METRIC

W.P. 462-93-00 LOCATION LITTLE EAT RIVER - SOUTH, N 5 032 055.2 E 325 268.0 ORIGINATED BY GA
DIST 52 HWY 11 BOREHOLE TYPE 210mm Hollow Stem Augers/Mud Rotary COMPILED BY WM
DATUM GEODETIC DATE 98.07.04 - 98.07.06 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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
							20	40	60	80	100	10	20	30			
15.0	SILT - continues as above		12	SS	20												
			13	SS	32												
			14	SS	38												
			15	SS	15												
			16	SS	16												
289.2																	
29.0	SILT and SAND, compact to dense, layered, grey, wet																
288.2																	

Continued Next Page

+ 3, x 3: Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No B-98-3

3 OF 4

METRIC

W.P. 462-93-00 LOCATION LITTLE EAT RIVER - SOUTH, N 5 032 055.2 E 325 268.0 ORIGINATED BY GA
DIST 52 HWY 11 BOREHOLE TYPE 210mm Hollow Stem Augers/Mud Rotary COMPILED BY WM
DATUM GEODETIC DATE 98.07.04 - 98.07.06 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			20 40 60 80 100	20 40 60 80 100					
30.0	SILT and SAND - continues as above		17	SS	15		288							0 55
							287							
							286							
							285							
			18	SS	21		284							0 55
							283							
							282							
			19	SS	33		281							0 22
							280							
							279							
			20	SS	17		278							
277.2							277							
41.0	SAND, very dense, trace silt, grey, wet: (SP)						276							
							275							0 90
274.9			21	SS	52		274							
43.3	END OF SAMPLING AT 43.3m. Water level at 0.9m on completion.													

Continued Next Page

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

RECORD OF BOREHOLE No B-98-3

4 OF 4

METRIC

W.P. 462-93-00 LOCATION LITTLE EAT RIVER - SOUTH, N 5 032 055.2 E 325 268.0 ORIGINATED BY GA
 DIST 52 HWY 11 BOREHOLE TYPE 210mm Hollow Stem Augers/Mud Rotary COMPILED BY WM
 DATUM GEODETIC DATE 98.07.04 - 98.07.06 CHECKED BY AEG

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE								
269.4												
48.8	END OF BOREHOLE AT 48.77m.											

RECORD OF BOREHOLE No B-98-4

1 OF 1

METRIC

W.P. 462-93-00 LOCATION LITTLE EAST RIVER - SOUTH, N 5 032 073.5 E 325 271.0 ORIGINATED BY GA
DIST 52 HWY 11 BOREHOLE TYPE 210mm Hollow Stem Augers with Mud COMPILED BY WM
DATUM GEODETIC DATE 98.07.06 - 98.07.06 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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
318.6 0.0	PEAT & ORGANIC SILT, very loose to loose, layered peat, organic silt and sandy inorganic silt, brown, grey silt, wet: (PT/OH)		1	SS	5		318									78.730	
			2	SS	3											52.580	
			3	SS	2		317									40.10	
316.5 2.1	SILT, compact, trace sand, trace clay, layered grey, wet: (ML)		4	SS	10		316										
			5	SS	11		315										0 8
			6	SS	11		314										
			7	SS	13		313										
311.9 6.7	END OF BOREHOLE AT 6.70m. Water level at 0.9m on completion.						312										0 7

APPENDIX B

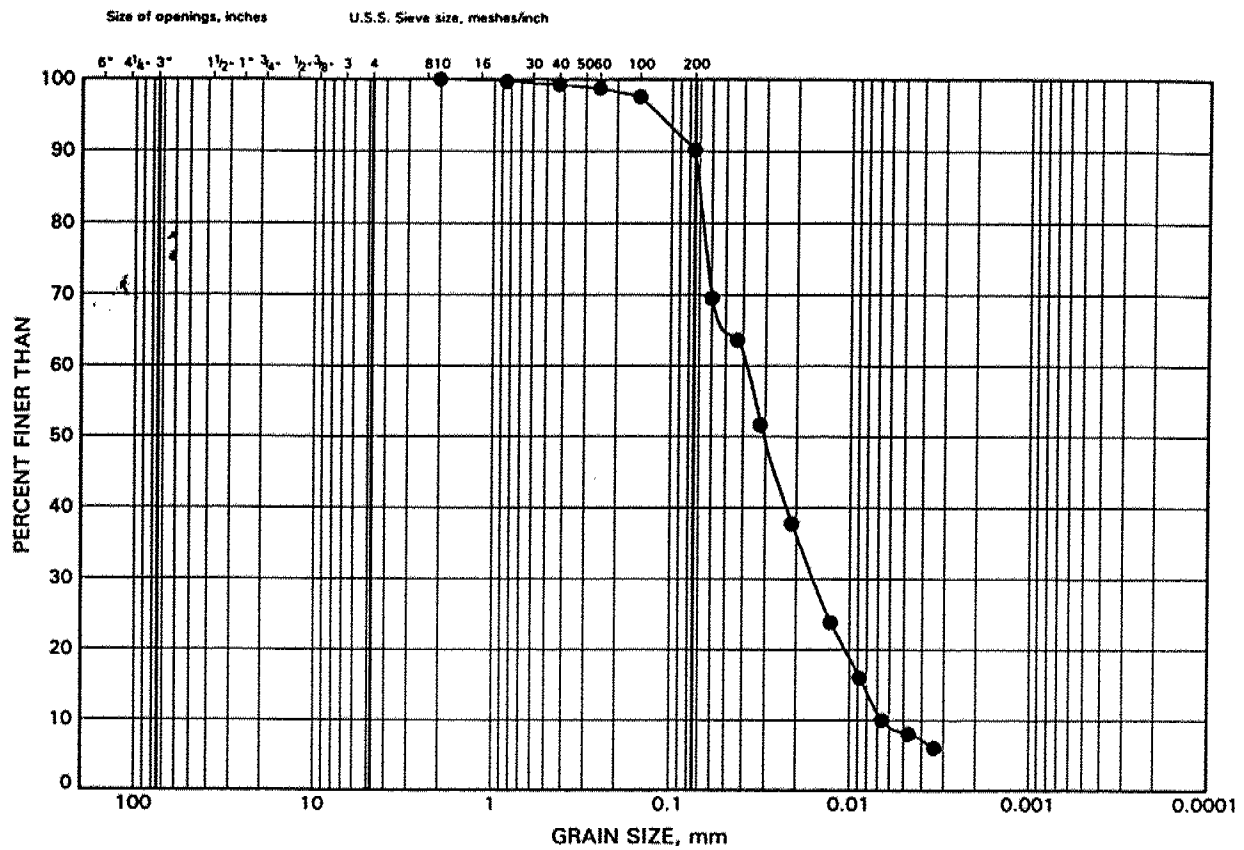
LABORATORY TEST RESULTS

- Figures B1 to B5 - Grain Size analyses

- Table 1 - pH and Sulphate

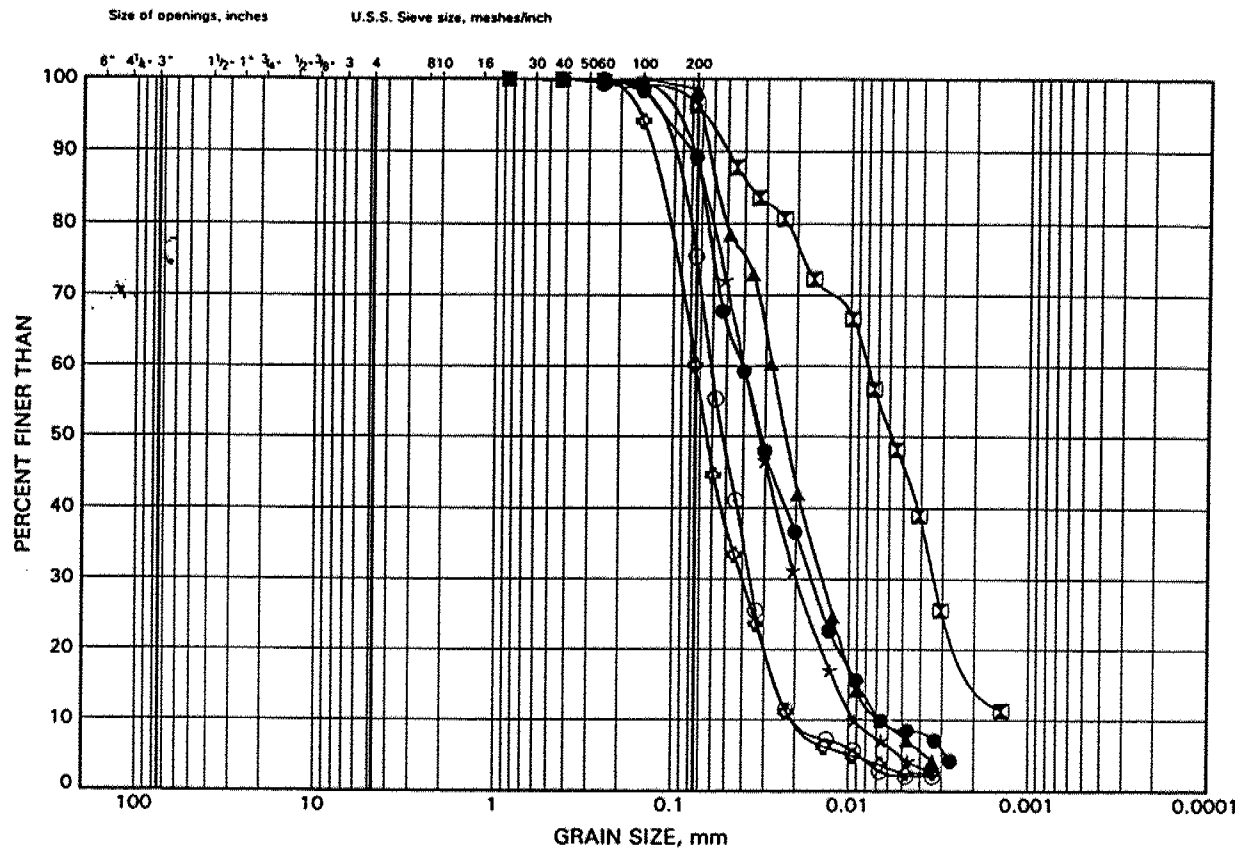
LITTLE EAST RIVER CROSSINGS GRAIN SIZE DISTRIBUTION

FIGURE B1



LITTLE EAST RIVER CROSSINGS GRAIN SIZE DISTRIBUTION

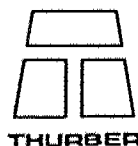
FIGURE B2



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
--------	----------	-----------	---------------

●	B-98-2	3.35	315.19
⊠	B-98-2	5.94	312.60
▲	B-98-2	21.64	296.90
★	B-98-2	30.78	287.76
⊙	B-98-2	38.40	280.14
⊕	B-98-2	42.98	275.56

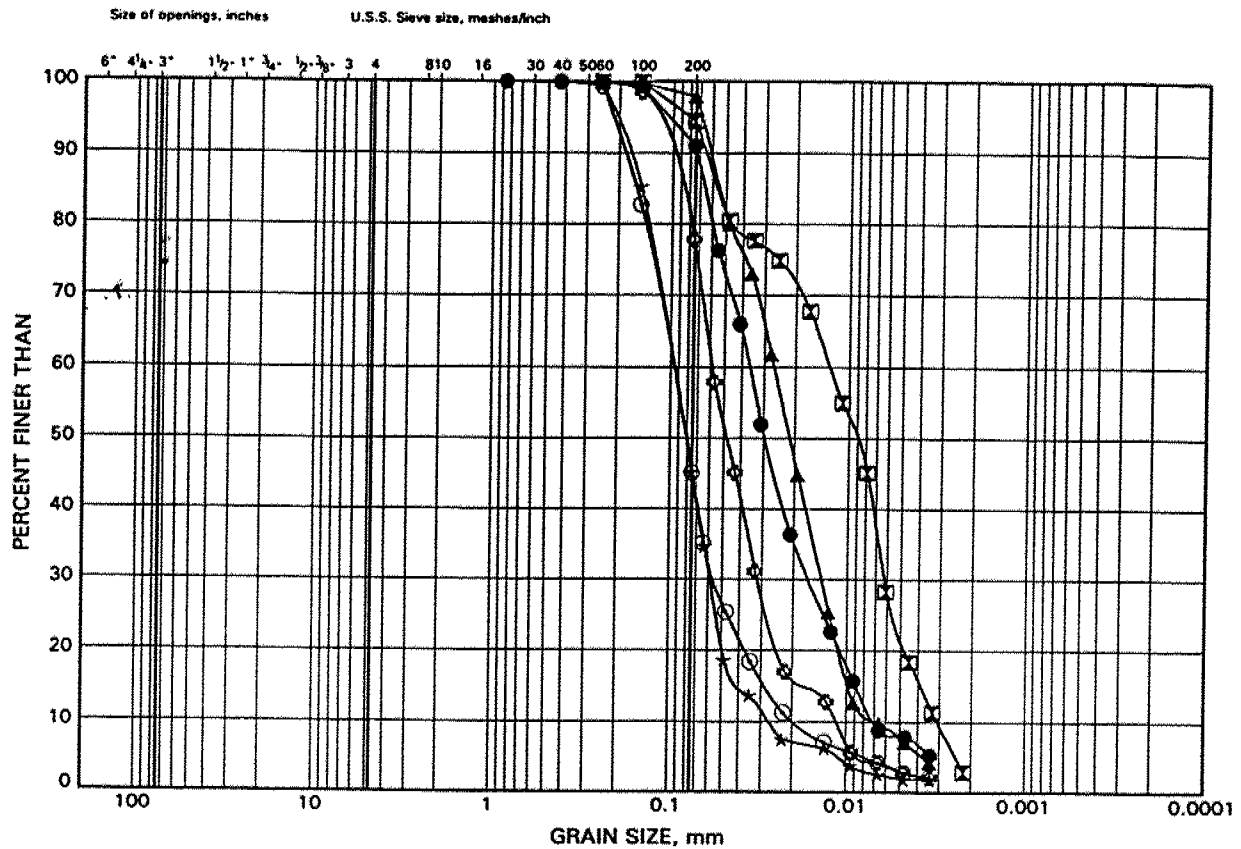
Date August 1998
Project 462-93-00



Prep'd WM
Chkd. AEG

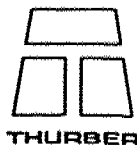
LITTLE EAST RIVER CROSSINGS GRAIN SIZE DISTRIBUTION

FIGURE B3



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

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	B-98-3	3.35	314.82
⊠	B-98-3	7.47	310.70
▲	B-98-3	18.59	299.58
★	B-98-3	30.78	287.39
⊙	B-98-3	33.83	284.34
◇	B-98-3	36.88	281.29



Date August 1998
Project 462-93-00

Prep'd WM
Chkd. AEG

FIGURE B4




●

B-98-3

42.98

275.19



Prep'd WM
Chkd. AEG

LITTLE EAST RIVER CROSSINGS GRAIN SIZE DISTRIBUTION

FIGURE B5

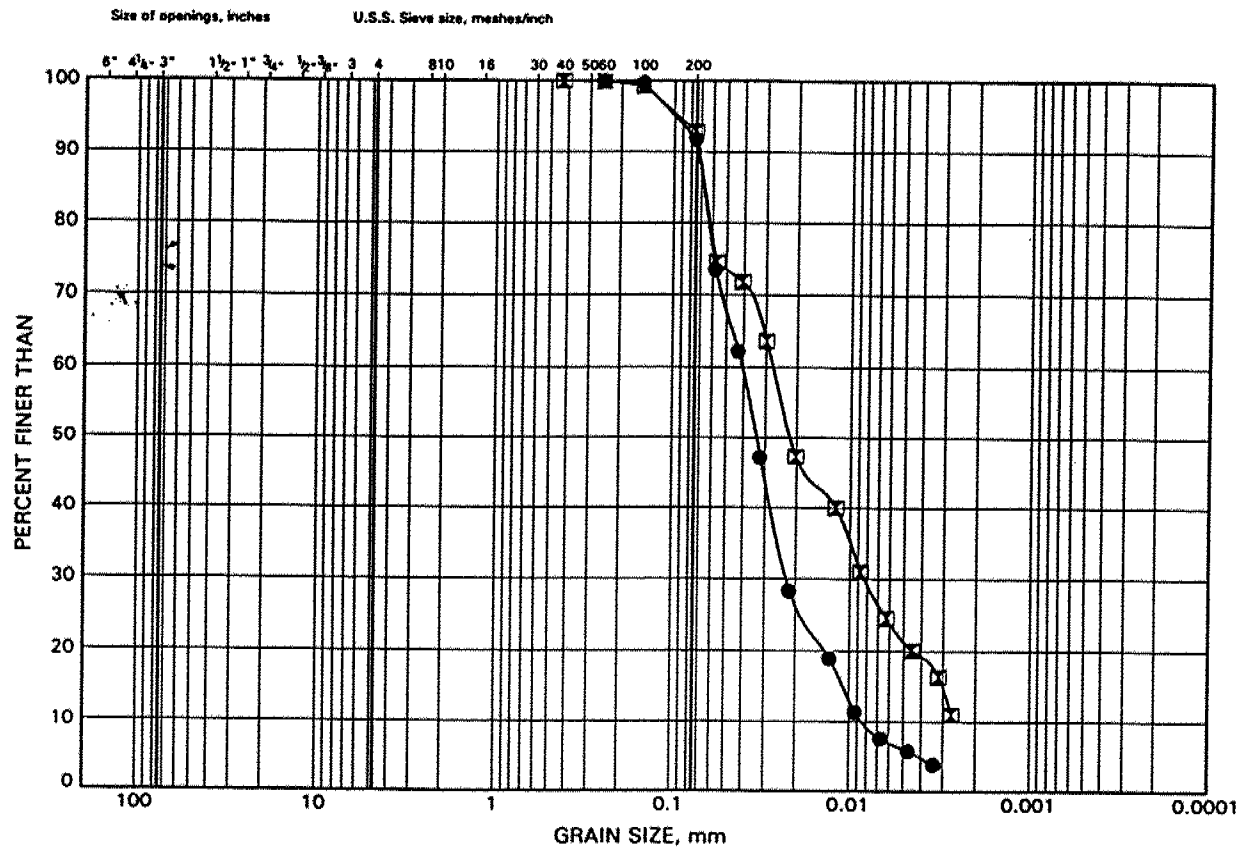


Table 1

Results of pH and Sulphate Testing

Sample	Depth (m)	pH	Sulphates (ppm)
B-98-2 Sa 3	1.5 - 2.1	8.56	109
B-98-2 Sa 6	4.1 - 4.7	5.66	250
B-98-3 Sa 3	1.5 - 2.1	5.18	2030
B-98-3 Sa 6	4.1 - 4.7	8.39	41

APPENDIX C
GRADING OF SAND
for
BACKFILL OF PILES

The space around the piles in the integral abutment design should be filled with sand meeting the following grading:

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

APPENDIX D

NON-STANDARD SPECIAL PROVISIONS

NON-STANDARD SPECIAL PROVISION

Sheet 1 of 1

Date October 30, 1998

WP No _____ CONTRACT No _____ DISTRICT No _____ HWY No _____

LOCATION _____ TYPE OF WORK _____

1. This SP is new .

This non-standard special provision outlines the groundwater conditions on site and draws attention to the need for groundwater control during certain stages of construction.

2.

Item	Spec. No.	Title or Item Description
		Groundwater Conditions and Control

The foundation investigation conducted at this site revealed the presence of two groundwater regimes. The near surface soils contain a free groundwater table which is stabilized near ground surface. A piezometer installed at a depth of approximately 10 m recorded an artesian groundwater condition with a head $0.9 \pm$ m above original ground surface.

During all stages of construction, and especially during stripping, excavation and backfill, and pile driving, appropriate measures must be implemented to control the ground water to depress the free level to at least 1.0 m below the base of the excavation and to prevent disturbance of the soil and to ensure that the necessary construction activities can be carried out in dry conditions.

Pile driving must be undertaken in a manner which will minimize soil disturbance around the pile shaft and clay seal and sand filter must be constructed around the driven piles to minimize subsequent upward seepage of groundwater around the pile shafts.

3.

Initiated By _____

Detailed By _____

Approved By _____

**FINAL FOUNDATION DESIGN REPORT FOR
LITTLE EAST RIVER - NORTH
HIGHWAY 11 FOUR LANING
6.7 km NORTH OF HWY 60 NORTHERLY 13 km
W.P. 462-93-00, SITE 42-234
DISTRICT 52, HUNTSVILLE**

Report

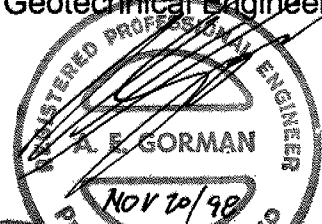
to

McCormick Rankin Corporation

Direction of fieldwork and engineering analysis by:

Thurber Engineering Ltd.
170 Evans Avenue, Suite 101
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Phone: (416) 503 3600
Fax: (416) 503 3010

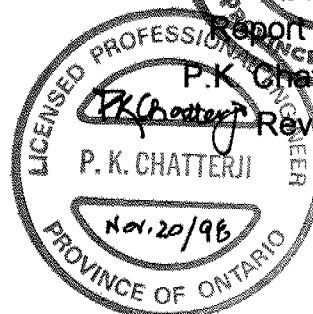
Alastair E. Gorman, P.Eng.,
Senior Geotechnical Engineer



Report reviewed by:

P.K. Chatterji, P.Eng.,

Review Principal



November 19, 1998

File: 19-1351-7C

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DRAWINGS

19-1351-7c-01	Borehole Location Plan
19-1351-7c-02	Soil Stratigraphy

FIGURES

Figure 1	Filter Layer Configuration
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APPENDICES

Appendix A	Borehole Logs
Appendix B	Laboratory Test Results
Appendix C	Grading of Sand for Backfill of Piles
Appendix D	Non-Standard Special Provision

**FINAL FOUNDATION DESIGN REPORT FOR
LITTLE EAST RIVER - NORTH
HIGHWAY 11 FOUR LANING
6.7 km NORTH OF HWY 60 NORTHERLY 13 km
W.P. 462-93-00, SITE 42-234
DISTRICT 52, HUNTSVILLE**

1. INTRODUCTION

This report presents the results of the foundation investigation and engineering analysis carried out by Thurber Engineering Ltd. (Thurber) at the site of the proposed bridge and approach fills to carry Highway 11 NBL across Little East River at Station 24+933, in the Town of Huntsville. The purpose of the investigation was to explore the subsurface soil and groundwater conditions at the site and based on the data obtained provide borehole logs, soil profile and a written description of the subsurface conditions. The purpose of the analysis of the data obtained during the investigation was to produce geotechnical recommendations for the design and construction of the structure foundations and associated earth works.

Thurber carried out the investigation as a sub-consultant to McCormick Rankin Corporation (MRC) under Ministry of Transportation (MTO) Agreement 9750 - 7424 - 5262.

2. SITE DESCRIPTION

2.1 Site Location

The subject site lies within the project limits of the four-laning of Highway 11 north of Huntsville and is located at the north crossing of the NBL over the Little East River. The site lies at Station 24+933, approximately, C/L Median Highway 11.

Locally, the site may be described as lying east of the existing Highway 11, in a flat, low-lying, tree covered area north of Stahl's Road. The Little East River meanders extensively across the area. Access to the site was from Highway 11 or Stahl's Road and then along the ROW of the NBL.

2.2 Physiography

Based on The Physiography of Southern Ontario, 3rd Edition, by Chapman and Putnam, the region surrounding the site consists of bedrock ridges with shallow overburden. The bedrock is undifferentiated igneous and metamorphic rock of early Precambrian age and is generally hard and massively jointed.

The Highway 11 corridor, however, lies in a long, narrow sand plain filling a deep valley within the region of shallow bedrock. The typical soils in the corridor consist of sand and silt, with some gravel deposited as glacial outwash or in localized glaciolacustrine environments.

The meandering creek (Little East River) and several wetlands in the area suggest poor drainage and a high groundwater table. Locally the ground is relatively flat, wet at the surface and supports typical vegetative cover for swampy, wet areas.

2.3 Site Layout

At this site, the NBL will cross the Little East River. Investigation was carried out on both banks of the creek to provide stratigraphic information related to the design and construction of structure foundations.

3. INVESTIGATION PROCEDURES

3.1 Field Investigation

Between July 8 and 13, 1998, a Nodwell track mounted auger and mud rotary drill rig was used on site for drilling, Standard Penetration Testing (SPT) (following the procedure outlined in ASTM D 1586) and dynamic cone penetration testing. One hole was drilled near each abutment and one at each approach fill, giving a total of four sampled boreholes. The approximate locations of the boreholes are shown on Drawing 19-1351-7c-01.

The holes were initially advanced using hollow stem augers and SPTs were carried out at intervals. Fine uniform sand and silt were encountered below a high water table which caused heaving of the soil into the hollow stem auger when the pilot bit was withdrawn in preparation for SPTs. The hollow stem augers were kept full of drilling mud at all times to counteract the effect of an unbalanced head of groundwater.

When it became apparent that auger drilling had reached its effective depth limit, mud rotary drilling was implemented for the balance of the depth of the hole. Investigation was carried out to a depth of approximately 49 m at the abutments and 6.7 m in the approach fill areas.

The boreholes numbers and depths of sampling were as follows:

Borehole No.	Depth of Sampling (m)
C-98-1	6.7
C-98-2	37.2
C-98-3	37.2
C-98-4	6.7

Samples were recovered at intervals throughout the depths of the boreholes in conjunction with Standard Penetration Tests (SPT) (following the test procedure outlined in ASTM D 1586). Samples were generally recovered at intervals of 0.75 m in the upper 3.0 m and thereafter at intervals of 1.5 m to depths of approximately 9 m. At greater depths, the sampling interval was increased to 3.0 m.

Dynamic cone penetration tests were conducted in, or adjacent to selected holes as follows

Borehole No.	Depth of Dynamic Cone Test
C-98-2	From a depth of 37.2 to 42.7 m
B-98-3	From a depth of 37.2 to 42.7 m

On completion of drilling and sampling, a standpipe piezometer was installed in Borehole C-98-3, at a depth of 10 m, to monitor the groundwater level.

The results of the drilling and sampling are summarized on the borehole logs in Appendix A.

Due to the suspected presence of artesian groundwater conditions, all boreholes were grouted on the completion of drilling and sampling, with the exception of the borehole interval where a piezometer was installed.

3.2 Laboratory Testing

Geotechnical laboratory testing consisted of natural moisture content determinations and visual classifications of all recovered samples. In addition, grain size analyses and pH and sulphate content testing were

conducted on selected samples. The results of the laboratory testing are presented on the borehole logs in Appendix A, and in Figures B1 to B5 in Appendix B. The results of the pH and sulphate content tests are presented in Table 1.

4. DESCRIPTION OF SUBSURFACE CONDITIONS

4.1 Subsurface Soil Conditions

Detailed descriptions of the subsoil conditions encountered in the boreholes are presented on the borehole logs in Appendix A. The stratigraphic profile inferred from the borehole information is shown on Drawing No. 19-1351-7c-02.

In general, the boreholes indicate a surface layer of peat and organic silt underlain by silt which extends to depths in the order of 8 to 10 m where it is underlain by fine sand.

Further description of these major soil units is provided in the following sections. The surface soils appear to result from deposition of organic material in swampy conditions, modified by the meandering action of the Little East River and associated deposition of alluvium. The soils encountered at greater depth all appeared to be lacustrine or fine outwash deposits and no evidence of boulders was found. However, the possibility of encountering boulders at random locations in the deposits during construction must be recognized.

Grain size distributions of selected samples are shown in Figures B1 to B5 in Appendix B.

Peat and Organic Silt

All boreholes encountered a surface layer of peat and organic silt interbedded with inorganic silt and sand layers in varying thicknesses. The peat is fibrous and contains numerous roots and the organic silt contains a high proportion of amorphous organic matter. The colour of the peat ranges from brown to dark brown to black while the inorganic silt and sand layers are grey. Measured moisture contents lie in the range of 25 to 189%.

For convenience at this site, these layered materials are collectively referred to as "peat" and the interpreted depths of peat at the boreholes are as follows:

Borehole No.	Depth of Peat (m)
C-98-1	3.0
C-98-2	2.1
C-98-3	3.0
C-98-4	2.3

Actual depths of peat to be stripped may vary from those interpreted at the borehole locations.

Silt

Based on Boreholes C-98-2 and C-98-3, the silt layer underlying the peat layer extends to depths of approximately 8 to 10 m below existing ground level, respectively, or to approximate Elevations 311.0 and 309.0. Boreholes C-98-1 and C-98-4 both terminated within the silt stratum at depths of 6.7 m.

The silt contains trace to some sand and trace clay. The deposit showed faint layering and thin clay seams and partings were noted throughout. In Borehole C-98-2, the percentage of clay and thickness of the clay seams was observed by visual examination to increase towards the bottom of the deposit. This observation is confirmed by the results of the grain size analysis carried out at 7.16 m, and reported on Figure B2 in Appendix B.

Based on the SPT values, the density of the silt layer generally ranges from very loose to loose with N values ranging from 1 to 11. One isolated reading of 11 was recorded in Borehole C-98-1. The measured natural moisture contents ranged from 19 to 33%.

Sand

Below the silt layer, Boreholes C-98-2 and C-98-3 encountered a deposit of fine, uniform sand. Near the top of the deposit, the sand is described as "silty" and at greater depth as "trace silt". The deposit shows faint layering, is grey and wet.

Based on the recorded SPT values, the sand is loose to compact to Elevation 304 with N values ranging from 6 to 13. Below that elevation, the sand is compact, becoming dense below Elevation 295 in Borehole C-98-2 and Elevation 287 in Borehole C-98-3. The measured natural moisture contents range from 16 to 32%.

4.2 Groundwater

During drilling, the groundwater level in the open boreholes was recorded as follows

Boreholes C-98-1 and C-98-2	0.6 m
Borehole C-98-3	0.8 m
Borehole C-98-4	0.7 m

The following groundwater levels were recorded in the piezometer installed in Borehole C-98-3:

Date	Height of Water (above existing ground surface)
July 15, 1998	1.0 m (artesian)
July 16, 1998	1.0 m (artesian)
July 31, 1998	>1.0 m (flowing artesian)
October 27, 1998	1.15 m (artesian)

After the July 31, 1998, reading, the standpipe was extended and the October 27, 1998, reading is believed to be representative of the artesian head at that time.

Based on this data and the close proximity of the creek, it is concluded that there is a free groundwater level close to ground surface. The near surface level is expected to fluctuate with the creek level.

The data from the piezometer installed at 10 m in Borehole C-98-3, however, indicates a small artesian head ($1.15 \pm$ m) with respect to the ground surface. Examination of the borehole data suggests that the silt layer encountered at depths of 2.1 and 3.0 m in Boreholes C-98-2 and C-98-3, respectively, acts as an aquitard and confines the artesian pressure in the lower, sand deposit. The artesian head may fluctuate due to seasonal variations or other influences.

5. RECOMMENDATIONS FOR STRUCTURE FOUNDATIONS

5.1 Type of Structure

The proposed structure will be a single-span bridge carrying the Highway 11 NBL over the Little East River. Geotechnical recommendations are required for the design of foundations at the north and south abutments.

The span of the proposed structure will be 18.3 m and it is understood that an integral abutment design is preferred, if the foundation conditions are suitable.

Geotechnical recommendations are also required for design and construction of the 4 m high approach fills immediately adjacent to the bridge. It is understood that the main embankment will be constructed of rock fill and that a rock fill shell is desirable for the side and forward slopes at the structure.

5.2 Foundation Soil Conditions

The factual description of the foundation soils is presented in Section 4 of this report. A discussion of the soil conditions is presented below.

The foundation conditions encountered in the Boreholes C-98-2 and C-98-3 consist of a deposit of silt underlain by a deep deposit of fine, uniform sand. These soils are considered suitable for the design of an integral abutment bridge with each abutment supported on a single row of H-piles driven to sufficient depth to achieve fixity well below the depth required to provide for movement of the abutment.

At this site, the presence of an artesian (above ground) head in the groundwater must be taken into account and appropriate measures implemented to minimize the risk of piping of soils occurring up the sides of

the piles with accompanying loss of load bearing capacity. The analysis of pile capacity has taken account of the risk of some loss of adhesion due to artesian flow.

The surficial layer of peat should be stripped as discussed in Section 7, Approach Embankment Design.

5.3 Piled Foundations

5.3.1 Axial Capacity

The foundations of the abutments should be supported on HP 310X110 piles.

Static analysis of the axial resistance has been carried out for the HP 310X110 pile in accordance with the OHBDC, using the soil parameters described in the Foundation Investigation Report. The analysis was conducted assuming that the piles are friction piles, using the full steel surface but no end plug. Partial loss of adhesion due to the upward flow caused by artesian pressure has been included in the analysis.

In the analysis of the vertical geotechnical resistance developed by the piles at the abutments, the following assumptions were made:

- the underside of the abutment stem will be approximately at Elevation 318.0
- the surficial layer of peat will be completely stripped and replaced by well compacted fill
- the pile was assumed not to develop any vertical resistance above Elevation 310.0

For the abutments, analysis indicated that an HP 310X110 pile driven to a total depth of 43.0 m below the base of the abutment stem would have a

factored geotechnical resistance at ULS of 1,000 kN and an SLS resistance of 750 kN. This is expected to correspond to a pile tip elevation of approximately 275.0.

The geotechnical resistance should be checked against the structural capacity of the pile.

The factored ULS has been calculated from static analysis using a resistance factor of 0.4. However, in view of the loose to compact soils and the artesian groundwater conditions encountered at the site, and the lack of pile construction experience in these difficult soil conditions, it is recommended that a static pile load test be conducted at the outset of construction. A resistance factor of 0.6 would be applied to the results of the load test which would then be used to confirm the design or to adjust the required driven length of pile.

5.3.2 Lateral Resistance

The lateral resistance of the HP 310X110 pile was assessed in accordance with Clause 6-9.8.1. Assuming flexure in the weak direction the assessed values are 100 kN at ULS and 27 kN at SLS.

The value of the coefficient of horizontal subgrade reaction, k_s (MN/m³), at any depth Z (m) is dependent on the soil characteristics and the pile width of the pile. The value is given by the equation:

$$k_s = n_h Z/b$$

Z = depth below ground surface

b = pile width

For this site, taking account of the high groundwater level, the recommended values for the constant n_h (MN/m³) are:

Condition	Value of n_h (MN/m ³)
Granular B, assumed above the water table	11.0
Native soil, assumed to be below the water table	4.5

5.3.3 Pile Driving

The piles should be provided with driving shoes in accordance with OPSP 3301.00.

Pile driving should be carefully monitored and controlled employing the Hiley Dynamic Pile Driving Formula in accordance with MTO Standards SS 103-10 or SS 103-11 and assuming an ultimate resistance of 2,000 kN.

The pile driving should be carried out using a hammer delivering at least 50 kJ per blow.

During the driving process, piles which have already been driven should be monitored to determine if they are heaving due to the effects of driving adjacent piles. If this phenomenon occurs, the affected piles must be re-driven.

5.3.4 Pile Driving Note

The pile driving note to be added to the drawings should be Note 7 in Clause 2.5.11 of the Structural Manual. The ultimate resistance to be used is 2,000 kN. The piles must be driven at least below Elevation 275.0 and not below Elevation 267.0 without approval of the Engineer.

5.3.5 Pile Installation Details

The recommendations contained in this section require that prior dewatering of the site be carried out in accordance with the requirements set out in Section 8 of this report.

It is recommended that special precautions be taken to minimize, as far as practical, the possibility of groundwater causing piping up the sides of the piles. These precautions are described in the following sections of this report.

Interpretation of the field data led to the conclusion that an aquitard exists in the silt layer at depths of 2 to 8 m in Borehole C-98-2 and 3 to 10 m in Borehole C-98-3. There is a concern that after each driven pile penetrates the aquitard, the underlying artesian head may cause continuing seepage up the side of the pile. If this seepage flow is able to carry fines with it, there will be a loss of ground and accompanying loss of vertical and horizontal pile capacity. To reduce the risk of piping, it is recommended that:

- a. a clay seal be constructed around each pile within the thickness of the aquitard
- b. a filter layer be constructed around the piles to allow any future upward seepage to dissipate without removal of a significant quantity of fines.

The design of the above items must recognize that a flexible foundation must be maintained for the integral abutment design.

Clay Seal

To achieve a seal around the pile in the top of the aquitard, the required 600 mm CSP should extend from the required top elevation down to Elevation 315.0. A hole large enough to accept a 600 mm CSP and provide an annular space (say a 1.0 m \pm diameter hole) should be drilled to Elevation 314.5. The CSP should be placed in this hole and a clay plug formed by placing granular bentonite, such as "Holeplug" by Baroid Drilling Fluids, Inc. The bentonite should be placed to a depth of 1.0 m inside and at the bottom of the CSP and also should completely fill the annular space outside the CSP for a depth of 1.0 m or up to the base of the excavation, whichever is less. Above the clay plug, the CSP should be filled with loose sand meeting the grading requirements shown in Appendix C. The sand column should be placed within two hours of placing the bentonite, and in any case before the end of the day, and must extend up to the underside of the abutment stem.

Filter

A filter should be constructed completely around all the CSPs installed at the abutment and should completely cover the stripped surface within the sheet piling. To construct the filter, the existing soil should be excavated to Elevation 317.0 and finished with a level base. If stripping the peat results in a lower elevation, a level base should be prepared at that level. Following preparation of the base, the filter should be constructed by placing a 1.0 m thick layer of fill meeting OPSS 1002 Table 1 "Gradation Requirements for Fine Aggregates" (filter sand). The fill should be placed in lifts not exceeding 300 mm in thickness and be compacted to at least 95% Standard proctor maximum dry density (SPMDD).

The foregoing points are illustrated in Figure 1, based on the assumed general arrangement at the south abutment.

5.4 Construction Sequence

The Contractor's plan for the sequence of construction events must address the following important elements:

1. Establish a satisfactory working platform.
2. Install any sheeting or shoring required.
3. Lower the water table to at least 1.0 m below the lowest level of excavation as described in Section 8 of this report.
4. Excavate all organic or otherwise deleterious material within the required area of excavation.
5. From the base of the excavation, auger to Elevation 314.5 a hole approximately 1,000 mm in diameter.
6. Inside this 1,000 mm hole, place a 500 mm layer of granular bentonite such as "Holeplug" by Baroid Drilling Fluids, Inc. and tamp lightly.
7. Seat the 600 mm CSP in the granular bentonite and support.
8. Place a further 500 mm of granular bentonite inside the 600 mm CSP and 500 mm in the annular space between the 600 mm CSP and the side of the hole. Fill the balance of the augered hole to the base of general excavation with filter sand.
9. Fill the 600 mm CSP with sand as specified in Appendix C.

NOTE. Steps 5 through 9 to be repeated for all piles.

10. Construct the specified sand filter around the piles to a thickness of at least 1.0 m or to greater thickness if this facilitates the construction sequence.

11. Proceed with the remaining backfill and pile driving in the most convenient order.
12. Maintain the dewatering system in operation until the later date of either:
 - completion of pile driving plus two days or
 - completion of filling to at least Elevation 318.0.

6. EARTH PRESSURE

The lateral earth pressures to be used in design should be computed in accordance with Section 6-7 of the OHBDC .

Granular backfill should be placed behind the integral abutment walls and wing walls to conform to the minimum requirements set out in Section 7 of this report. The granular backfill should conform to Ontario Provincial Standard Specifications (OPSS) 1010 for Granular B, Type I. The fill should be placed in accordance with OPSS 501. A perforated subdrain should be installed behind the base of the walls to maintain the granular fill in a drained condition. The subdrain should be provided with a positive outlet to the highway drainage system.

For the above backfill and drainage conditions, the abutment walls and wing walls may be designed based on the following unfactored earth pressure distributions:

$$P_h = K \gamma h$$

where;

K = earth pressure coefficient, use value from table on following page

γ = unit weight of soil = 21.2 kN/m³ for Granular B

h = depth below top of wall, m

Wall Type	Earth Pressure Coefficient (K)	
	OPSS Granular B $\phi' = 30^\circ$	
	Horizontal Ground Surface Behind Wall	Sloping Ground Surface (2H:1V)
Abutment Walls (Restrained Wall)	0.50	-
Wing Walls (Unrestrained Wall)	0.33	0.55

If an integral abutment design is used, the abutments will be cast integrally with the deck and therefore the abutment walls should be treated as restrained. If the wing walls will not be connected to the abutments and therefore will be able to accommodate some rotation they may be treated as unrestrained. The above also assumes a horizontal ground surface behind the abutment walls. If concrete approach slabs are not provided, an additional load equivalent to 600 mm of fill should be superimposed on the wall loadings to account for traffic surcharge loading.

An additional lateral pressure of 16 kPa should be added to account for compaction induced forces. The additional pressure must be computed in accordance with Clause 6-7.4.3 of the OHBDC.

7. APPROACH EMBANKMENT DESIGN

Based on Drawing No. 06250011173, the driving lanes of Highway 11 will be constructed on embankments with a finished grade up to 4 m above existing ground level. These embankments will be constructed across a swampy area with potential artesian groundwater conditions.

As shown on the borehole logs, a surface layer of peat was encountered at the boreholes which measured 2.1 to 3.1 m in thickness. All peat and soft soils should be removed under the approach embankment. It is understood that a geotechnical report has been produced by others which recommends removal of the peat and construction of rock fill for the main embankment.

Rock fill is not considered acceptable in a zone immediately behind the abutment and wing walls of an integral abutment bridge. A zone of granular backfill is required behind the abutment and around the wing walls to provide for the movement requirements of the integral abutment. In profile, this zone of backfill should commence 1.2 m behind the base of the abutment stem and slope upwards at 2H:1V. The fill within this zone should consist of Granular B, as described in Section 6 of this report, Earth Pressure.

Approach embankments constructed of Granular "B" may have side slopes no steeper than 2:1.

If the approach embankment slopes are constructed of rock fill and all peat and soft soils are removed from the embankment area, side slopes of 1.25H:1V are expected to be stable and should be designed following OPSS 203.010.

Embankment fill should be placed in appropriate lift thicknesses and be compacted in accordance with OPSS 501.

The design of the approach embankments and the materials to be used should be coordinated with the recommendations in the Pavement Design Report.

8. EXCAVATION AND GROUNDWATER CONTROL

The groundwater level established by short term piezometer readings is 1.15 m above existing ground surface, or at Elevation 320.15.

Various elements of construction discussed earlier in this report require work to

be carried out in dry conditions and with control of the underlying artesian pressure. Given the soil and groundwater conditions encountered at this site, it is recommended that the necessary dewatering be achieved by:

- forming a sheeted excavation with interlocking steel sheet piling driven at least 2.0 m beyond the limits of the main piles
- driving the sheeting to Elevation 312.5 to obtain a seal in the inferred aquitard, but no deeper so as to avoid unnecessary penetration of the aquitard
- depressing the water table within the sheeted area to at least 1.0 m below the projected base of the excavation, taking account of the reported artesian head (boiling and/or base heave may occur if proper dewatering is not achieved)

Given the fine grained soil conditions encountered at this site, it is considered that vacuum assisted wellpoints will be required to achieve the necessary dewatering.

The sheeting and dewatering must be maintained at least until backfill within the sheeting is up to Elevation 318.0. At that stage the dewatering system may be decommissioned. As part of the decommissioning, any penetration of the aquitard must be grouted or otherwise sealed to prevent seepage.

All aspects of the dewatering must be designed by a specialist in this field.

From a geotechnical standpoint, the sheeting should be left in place and cut off about ground level to assist in the control of any seepage that may occur.

All excavations must be carried out in accordance with the Occupational Health and Safety Act and the sheeting and dewatering must be designed by a Professional Engineer specializing in that field.

No permanent groundwater control measures are required for the proposed piled foundation.

Appendix D contains a suggested NSSP which should be included in the contract to alert the Contractor to the existence of a high surface groundwater table and an artesian condition at relatively shallow depth. The NSSP also alerts the Contractor to the necessity of controlling the groundwater levels and carrying out certain aspects of the construction in dry conditions.

9. FROST PROTECTION

The design depth of frost penetration for this project is 1.8 m. All pile caps and footings designed for this site must be provided with a minimum depth of soil cover of 1.8 m to protect against the penetration of frost below the foundation elements.

10. CONSTRUCTION CONCERNS

The main construction concerns relate to:

- maintenance of a clay seal around the driven piles at the top of the aquitard to minimize upward seepage
- proper construction of a filter layer around the piles to limit the loss of fines in the event that seepage flow up the side of the pile does occur
- proper dewatering of the site so that the work around the abutment can proceed as recommended

11. CONSTRUCTION INSPECTION AND MONITORING

During construction, all foundation installation, excavation and approach embankment construction activities should be monitored by geotechnical personnel to ensure that the foundation recommendations and design are being correctly implemented and that the soil conditions encountered do not differ materially from the interpretation used in this report.

STATEMENT OF GENERAL CONDITIONS

1. STANDARD OF CARE

This study and Report have been prepared in accordance with generally accepted engineering or environmental consulting practices in this area. No other warranty, expressed or implied, is made.

2. COMPLETE REPORT

All documents, records, data and files, whether electronic or otherwise, generated as part of this assignment are a part of the Report which is of a summary nature and is not intended to stand alone without reference to the instructions given to us by the Client, communications between us and the Client, and to any other reports, writings, proposals or documents prepared by us for the Client relative to the specific site described herein, all of which constitute the Report.

IN ORDER TO PROPERLY UNDERSTAND THE SUGGESTIONS, RECOMMENDATIONS AND OPINIONS EXPRESSED HEREIN, REFERENCE MUST BE MADE TO THE WHOLE OF THE REPORT. WE CANNOT BE RESPONSIBLE FOR USE BY ANY PARTY OF PORTIONS OF THE REPORT WITHOUT REFERENCE TO THE WHOLE REPORT.

3. BASIS OF REPORT

The Report has been prepared for the specific site, development, design objectives and purpose that were described to us by the Client. The applicability and reliability of any of the findings, recommendations, suggestions, or opinions expressed in the document are only valid to the extent that there has been no material alteration to or variation from any of the said descriptions provided to us unless we are specifically requested by the Client to review and revise the Report in light of such alteration or variation.

4. USE OF THE REPORT

The information and opinions expressed in the Report, or any document forming part of the Report, are for the sole benefit of the Client. NO OTHER PARTY MAY USE OR RELY UPON THE REPORT OR ANY PORTION THEREOF WITHOUT OUR WRITTEN CONSENT. WE WILL CONSENT TO ANY REASONABLE REQUEST BY THE CLIENT TO APPROVE THE USE OF THIS REPORT BY OTHER PARTIES AS "APPROVED USERS". The contents of the Report remain our copyright property and we authorize only the Client and Approved Users to make copies of the Report only in such quantities as are reasonably necessary for the use of the Report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make the Report, or any portion thereof, available to any party without our written permission. Any use which a third party makes of the Report, or any portion of the Report, are the sole responsibility of such third parties. We accept no responsibility for damages suffered by any third party resulting from unauthorized use of the Report.

5. INTERPRETATION OF THE REPORT

- a) Nature and Exactness of Soil and Contaminant Description: Classification and identification of soils, rocks, geological units, contaminant materials and quantities have been based on investigations performed in accordance with the standards set out in Paragraph 1. Classification and identification of these factors are judgemental in nature and even comprehensive sampling and testing programs, implemented with the appropriate equipment by experienced personnel, may fail to locate some conditions. All investigations utilizing the standards of Paragraph 1 will involve an inherent risk that some conditions will not be detected and all documents or records summarizing such investigations will be based on assumptions of what exists between the actual points sampled. Actual conditions may vary significantly between the points investigated and all persons making use of such documents or records should be aware of, and accept, this risk. Some conditions are subject to change over time and those making use of the Report should be aware of this possibility and understand that the Report only presents the conditions at the sampled points at the time of sampling. Where special concerns exist, or the Client has special considerations or requirements, the Client should disclose them so that additional or special investigations may be undertaken which would not otherwise be within the scope of investigations made for the purposes of the Report.

(see over...)

INTERPRETATION OF THE REPORT *(continued)*

- b) Reliance on Provided Information: The evaluation and conclusions contained in the Report have been prepared on the basis of conditions in evidence at the time of site inspections and on the basis of information provided to us. We have relied in good faith upon representations, information and instructions provided by the Client and others concerning the site. Accordingly, we cannot accept responsibility for any deficiency, misstatement or inaccuracy contained in the Report as a result of misstatements, omissions, misrepresentations, or fraudulent acts of persons providing information.

6. RISK LIMITATION

Geotechnical engineering and environmental consulting projects often have the potential to encounter pollutants or hazardous substances and the potential to cause an accidental release of those substances. In consideration of the provision of the services by us, which are for the Client's benefit, the Client agrees to hold harmless and to indemnify and defend us and our directors, officers, servants, agents, employees, workmen and contractors (hereinafter referred to as the "Company") from and against any and all claims, losses, damages, demands, disputes, liability and legal investigative costs of defence, whether for personal injury including death, or any other loss whatsoever, regardless of any action or omission on the part of the Company, that result from an accidental release of pollutants or hazardous substances occurring as a result of carrying out this Project. This indemnification shall extend to all Claims brought or threatened against the Company under any federal or provincial statute as a result of conducting work on this Project. In addition to the above indemnification, the Client further agrees not to bring any claims against the Company in connection with any of the aforementioned causes.

7. SERVICES OF SUBCONSULTANTS AND CONTRACTORS

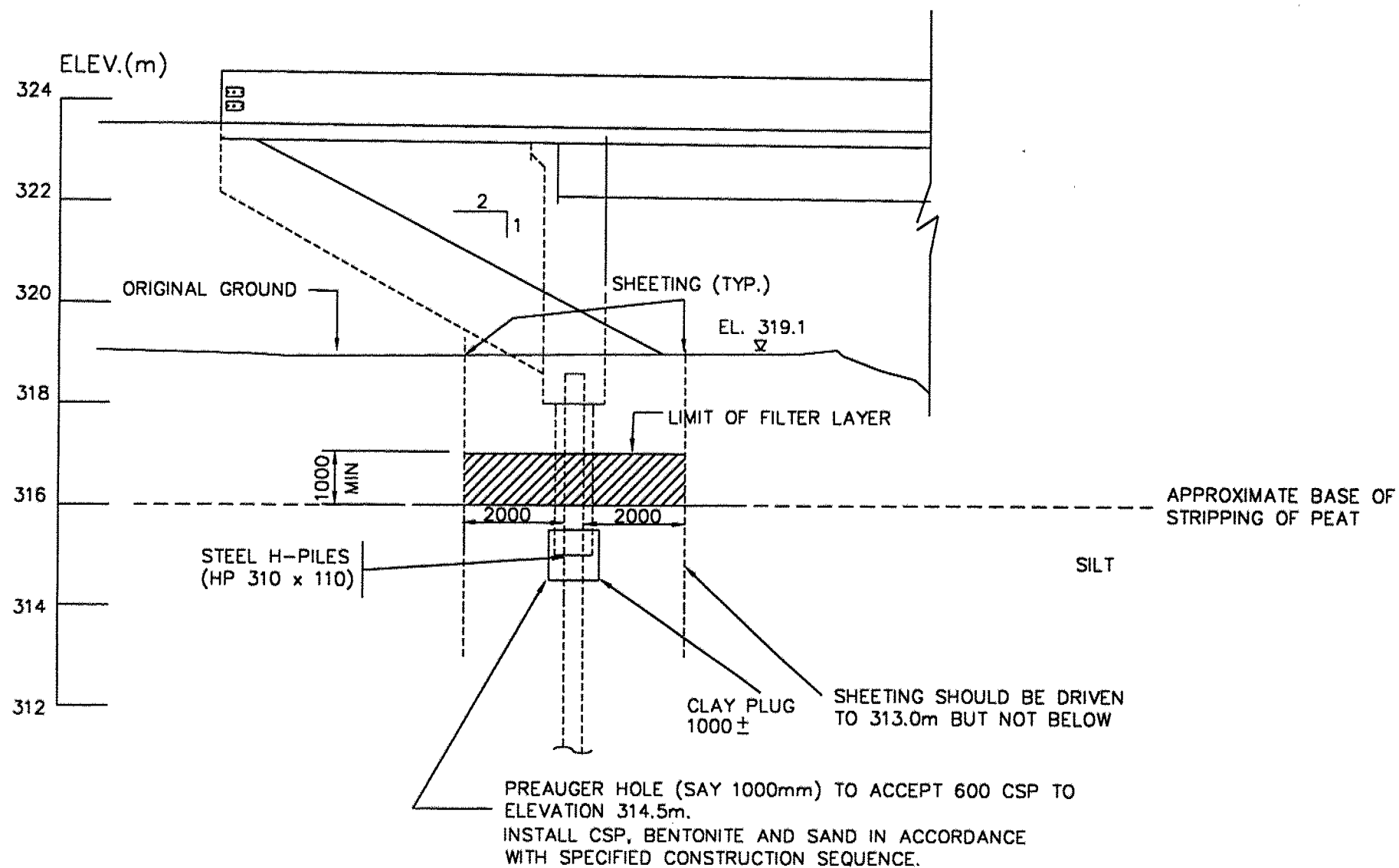
The conduct of engineering and environmental studies frequently requires hiring the services of individuals and companies with special expertise and/or services which we do not provide. We may arrange the hiring of these services as a convenience to our Clients. As these services are for the Clients' benefit, the Client agrees to hold the Company harmless and to indemnify and defend us from and against all claims arising through such hirings to the extent that the Client would incur had he hired those services directly. This includes responsibility for payment for services rendered and pursuit of damages for errors, omissions or negligence by those parties in carrying out their work. In particular, these conditions apply to the use of drilling, excavation and laboratory testing services.

8. CONTROL OF WORK AND JOBSITE SAFETY

We are responsible only for the activities of our employees on the jobsite. The presence of our personnel on the site shall not be construed in any way to relieve the Client or any contractors on site from their responsibilities for site safety. The Client acknowledges that he, his representatives, contractors or others retain control of the site and that we never occupy a position of control of the site. The Client undertakes to inform us of all hazardous conditions, or other relevant conditions of which the Client is aware. The Client also recognizes that our activities may uncover previously unknown hazardous conditions or materials and that such a discovery may result in the necessity to undertake emergency procedures to protect our employees as well as the public at large and the environment in general. These procedures may well involve additional costs outside of any budgets previously agreed to. The Client agrees to pay us for any expenses incurred as the result of such discoveries and to compensate us through payment of additional fees and expenses for time spent by us to deal with the consequences of such discoveries. The Client also acknowledges that in some cases the discovery of hazardous conditions and materials will require that certain regulatory bodies be informed and the Client agrees that notification to such bodies by us will not be a cause of action or dispute.

9. INDEPENDENT JUDGEMENTS OF CLIENT

The information, interpretations and conclusions in the Report are based on our interpretation of conditions revealed through limited investigation conducted within a defined scope of services. We cannot accept responsibility for independent conclusions, interpretations, interpolations and/or decisions of the Client, or others who may come into possession of the Report, or any part thereof, which may be based on information contained in the Report. This restriction of liability includes decisions made to either purchase or sell land.



LEGEND



FILTER LAYER

FILTER LAYER CONFIGURATION (SCHEMATIC)

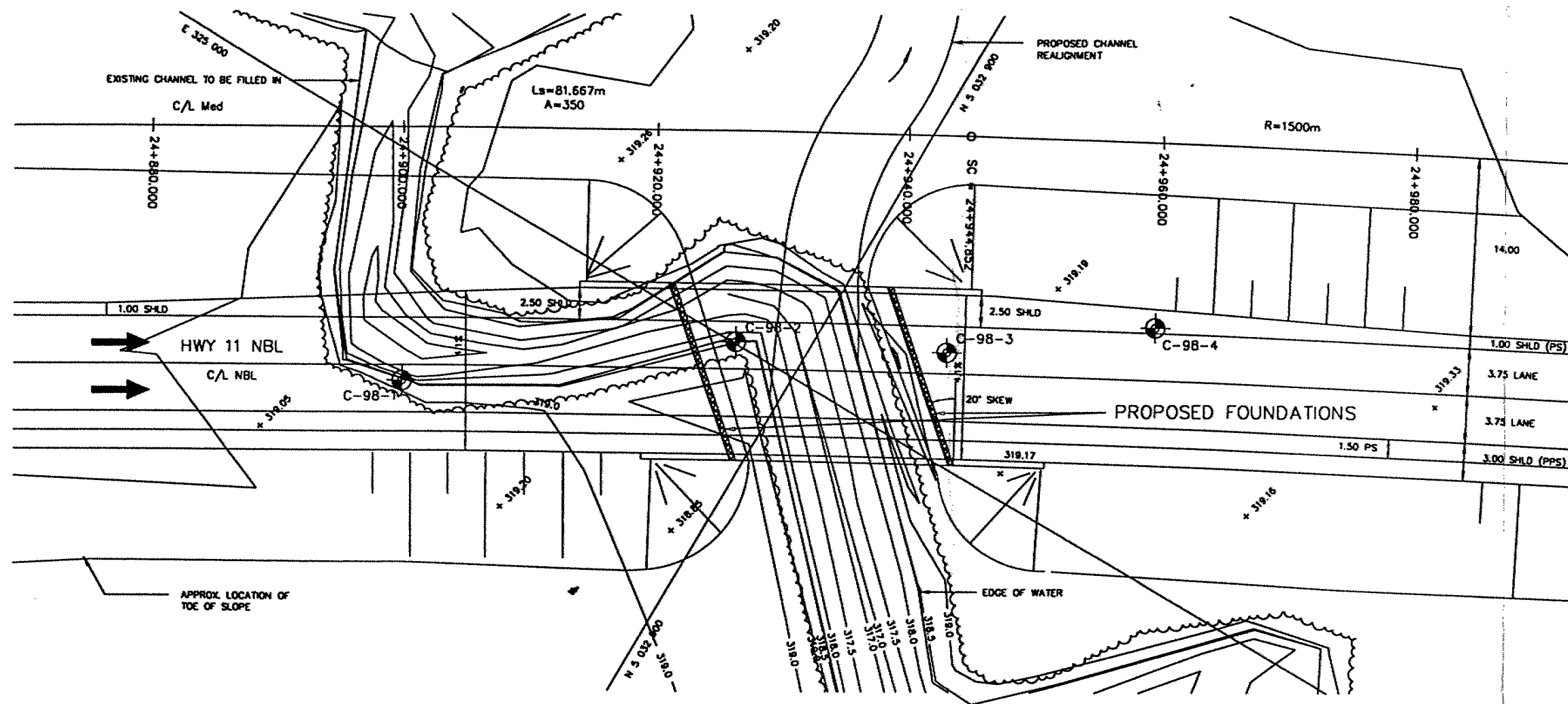
DIST 52
 CONT No
 WP No 458-93-00



HIGHWAY 11- FOUR LANE
 LITTLE EAST RIVER

SHEET

THURBER ENGINEERING LTD.



BOREHOLE LOCATION PLAN

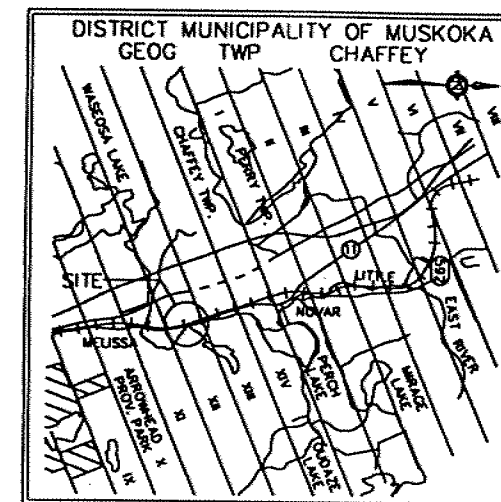
SCALE 1:400

LEGEND



LOCATION OF BOREHOLE

C-98-1



KEY PLAN

1500m 0 3000 6000m

METRIC

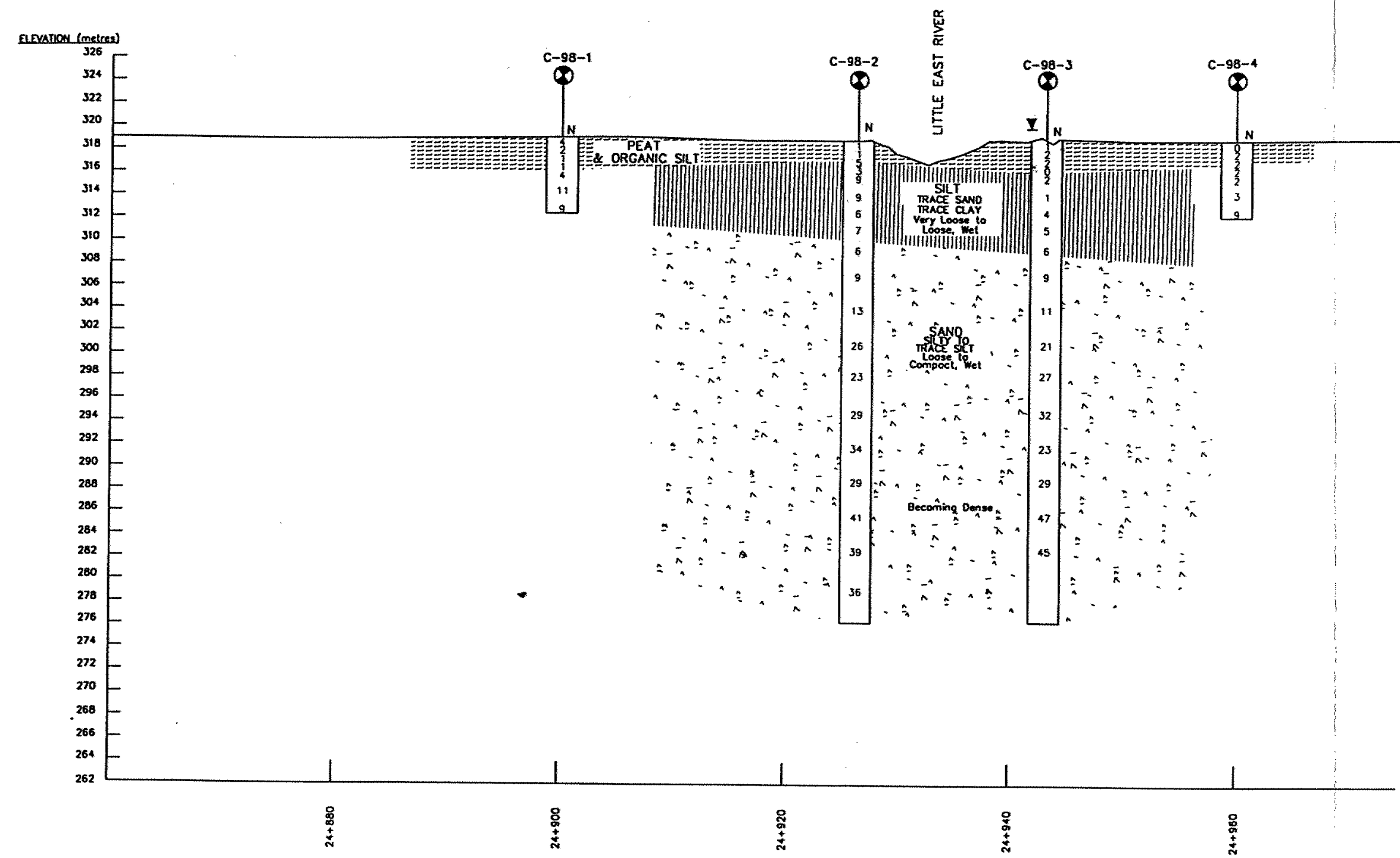
DIMENSIONS ARE IN METRES
 AND/OR MILLIMETRES
 UNLESS OTHERWISE SHOWN

PLAN BASED ON PLAN E-625-11-
 SHT 1 of 2 - SUPPLIED BY CLIENT

No	ELEV.	LOCATION	
		NORTHING	EASTING
C-98-1	319.2	5 032 874.1	325 021.6
C-98-2	319.0	5 032 895.4	325 005.5
C-98-3	319.0	5 032 910.4	324 997.4
C-98-4	319.3	5 032 923.1	324 987.3

19-1351-7c-01

DIST 52	
CONT No	
WP No 458-93-00	
HIGHWAY 11- FOUR LANE LITTLE EAST RIVER	SHEET
THURBER ENGINEERING LTD.	



SOIL PROFILE ALONG HWY 11 NBL OVER LITTLE EAST RIVER

SCALE 1:400

- LEGEND**
- C-98-1 LOCATION OF BOREHOLE
 - WATER LEVEL, JULY 31, 1998.
 - N BLOWS/0.3m (Std Pen Test)

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

PLAN BASED ON PLAN E-625-11-
 SHT 2 of 2 - SUPPLIED BY CLIENT

APPENDIX A

BOREHOLE LOGS

- Symbols and Terms Used on Borehole Logs
- Unified Soil Classification
- Borehole Logs C-98-1 to C-98-4



RECORD OF BOREHOLE No C-98-1

1 OF 1

METRIC

W.P. 462-93-00 LOCATION 24 + 900 19.0m Rt (Median) ORIGINATED BY GA
DIST 52 HWY 11 BOREHOLE TYPE 210mm Hollow Stem Augers with Mud COMPILED BY WM
DATUM GEODETIC DATE 98.07.09 - 98.07.09 CHECKED BY AEG

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT Y 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									
								O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE									
							WATER CONTENT (%)										
							20 40 60 80 100										
							10 20 30										
							PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT										
							W _p W W _L										
319.2	PEAT & ORGANIC SILT, very dense, layered peat, organic silt, sand and silt seams, brown with grey silt, wet: (PT/OH)		1A	SS	4		319							101.8	0 73 26 1		
1B			SS														
2			SS	2													
3			SS	1													92.30
4			SS	1													84.0
316.2	SILT, loose to compact, trace clay, trace sand, some thin clay seams, grey, wet: (ML)		5	SS	4		316								0 2 94 4		
6			SS	11													
312.5	END OF BOREHOLE AT 6.7m. Water level at 0.6m on completion.		7	SS	9		313										
6.7																	

RECORD OF BOREHOLE No C-98-2

1 OF 3

METRIC

W.P. 462-93-00 LOCATION 24 + 926 16.6m Rt (Median) ORIGINATED BY GA
DIST 52 HWY 11 BOREHOLE TYPE 210mm Hollow Stem Augers with Mud COMPILED BY WM
DATUM GEODETIC DATE 98.07.08 - 98.07.09 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	● QUICK TRIAXIAL						
319.0							20	40	60	80	100					
0.0	PEAT & ORGANIC SILT, very loose to loose, layered peat, organic silt, sand and silt, brown with grey silt, wet: (PT/OH)		1	SS	1											
			2	SS	1											
			3	SS	5											
316.9	SILT, loose, trace to some sand, layered, some thin clay seams, grey, wet: (ML)		4	SS	3											
			5	SS	9											
			6	SS	9											
			7	SS	6											
			8	SS	7											
			9	SS	6											
			10	SS	9											
			11	SS	13											
311.0	SAND, loose to compact, trace silt to silty, faint layering, some thin clay seams and partings, grey, wet: (SP)															
8.0																

Continued Next Page

Numbers refer to
Sensitivity

20
15
10

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No C-98-2

3 OF 3

METRIC

W.P. 462-93-00 LOCATION 24 + 926 16.6m Rt (Median) ORIGINATED BY GA
DIST 52 HWY 11 BOREHOLE TYPE 210mm Hollow Stem Augers with Mud COMPILED BY WM
DATUM GEODETIC DATE 98.07.08 - 98.07.09 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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE	WATER CONTENT (%)					
	SAND - continues as above						289							
			17	SS	41		288							
							287							
							286							
			18	SS	39		285							
							284							
							283							
281.8							282							0 91 9
37.2	END OF SAMPLING AT 37.2m.		19	SS	36		281							
							280							
							279							
							278							
							277							
276.3														
42.7	END OF BOREHOLE AT 42.7m. Water level at 0.6m on completion.													

RECORD OF BOREHOLE No C-98-3

1 OF 3

METRIC

W.P. 462-93-00 LOCATION 24 + 943 17.1m Rt (Median) ORIGINATED BY GA
 DIST 52 HWY 11 BOREHOLE TYPE 210mm Hollow Stem Augers with Mud COMPILED BY WM
 DATUM GEODETIC DATE 98.07.13 - 98.07.14 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	20 40 60 80 100					
319.0	PEAT & ORGANIC SILT, very loose, layered peat, organic silt, sand and silt, brown with grey sand and silt, wet: (PT/OH)		1	SS	1	319						17.00	0 14 84 2
			2	SS	2	318						83.30	
			3	SS	2	317						188.20	
			4	SS	0	316							
316.0	SILT, very loose to loose, some sand, trace clay, some thin clay seams and partings, grey, wet: (ML)		5	SS	2	316							
			6	SS	1	315							
						314							
			7	SS	4	313							
			8	SS	5	312							
			9	SS	6	311							
309.0	SAND, loose to compact, trace silt to silty, faint layering, some thin clay seams and partings, grey, wet: (SP)					310							0 74 25 1
			10	SS	9	309							
						308							
						307							
306.0						306							
305.0						305							
304.0						304							

Continued Next Page

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

METRIC

[illegible]

Numbers refer to Sensitivity

RECORD OF BOREHOLE No C-98-3

3 OF 3

METRIC

W.P. 462-93-00 LOCATION 24 + 943 17.1m Rt (Median) ORIGINATED BY GA
DIST 52 HWY 11 BOREHOLE TYPE 210mm Hollow Stem Augers with Mud COMPILED BY WM
DATUM GEODETTIC DATE 98.07.13 - 98.07.14 CHECKED BY AEG

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		UNIT WEIGHT Y 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 (%) 20 40 60 80 100			
								10 20 30		
	SAND - continues as above		16	SS	29					
	becoming dense to very dense below 32m									
			17	SS	47					
									</	

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

RECORD OF BOREHOLE No C-98-4

1 OF 1

METRIC

W.P. 462-93-00 LOCATION 24 + 959 14.5m Rt (Median) ORIGINATED BY GA
DIST 52 HWY 11 BOREHOLE TYPE 210mm Hollow Stem Augers with Mud COMPILED BY WM
DATUM GEODETIC DATE 98.07.14 - 98.07.14 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						WATER CONTENT (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
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319.3	PEAT & ORGANIC SILT, very loose, layered peat, organic silt, sand and silt seams, brown with grey sand and silt, wet: (PT/OH)		1	SS	0																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																														

APPENDIX B

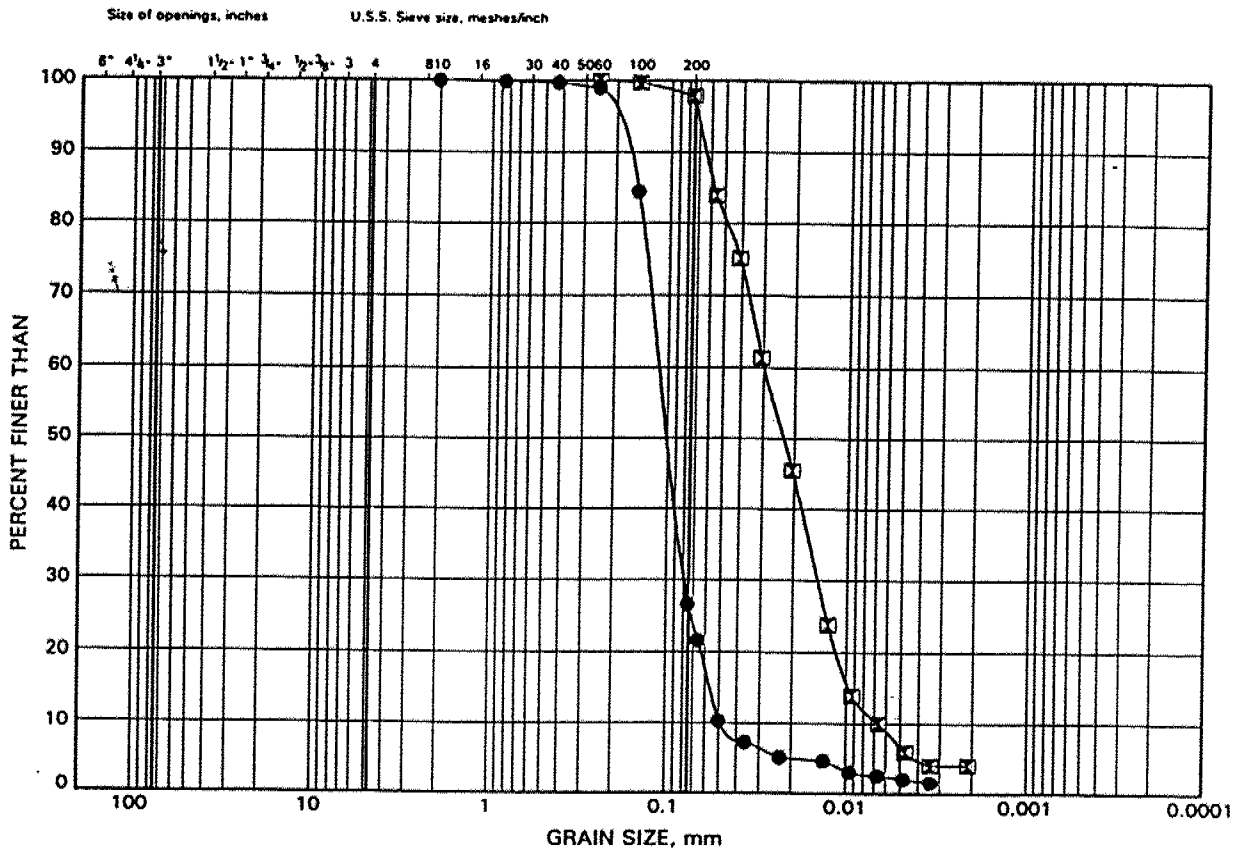
LABORATORY TEST RESULTS

- Figures B1 to B4 - Grain Size analyses

- Table 1 - pH and Sulphate

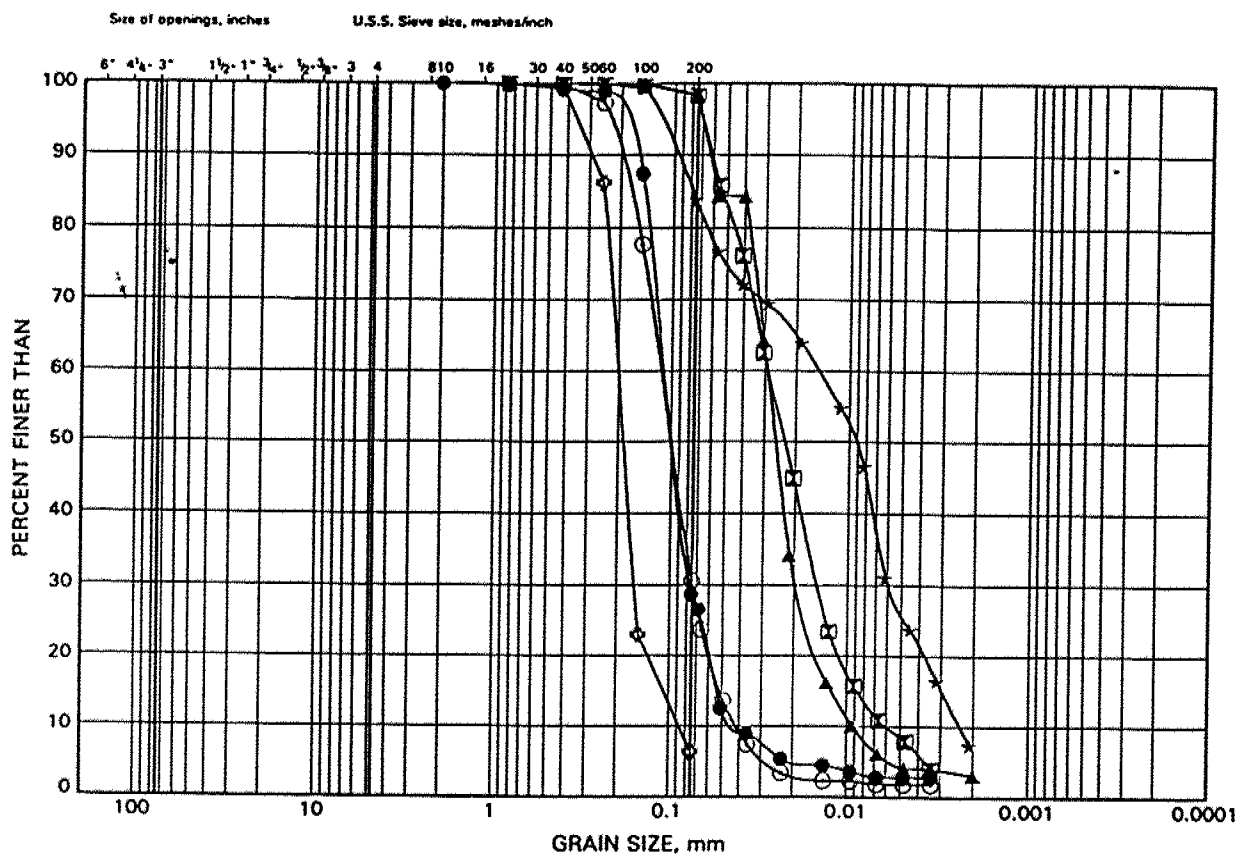
LITTLE EAST RIVER CROSSINGS GRAIN SIZE DISTRIBUTION

FIGURE B1



LITTLE EAST RIVER CROSSINGS GRAIN SIZE DISTRIBUTION

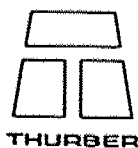
FIGURE B2



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

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	C-98-2	1.07	317.93
⊠	C-98-2	2.59	316.41
▲	C-98-2	5.64	313.36
★	C-98-2	7.16	311.84
⊙	C-98-2	12.50	306.50
⊕	C-98-2	27.74	291.26

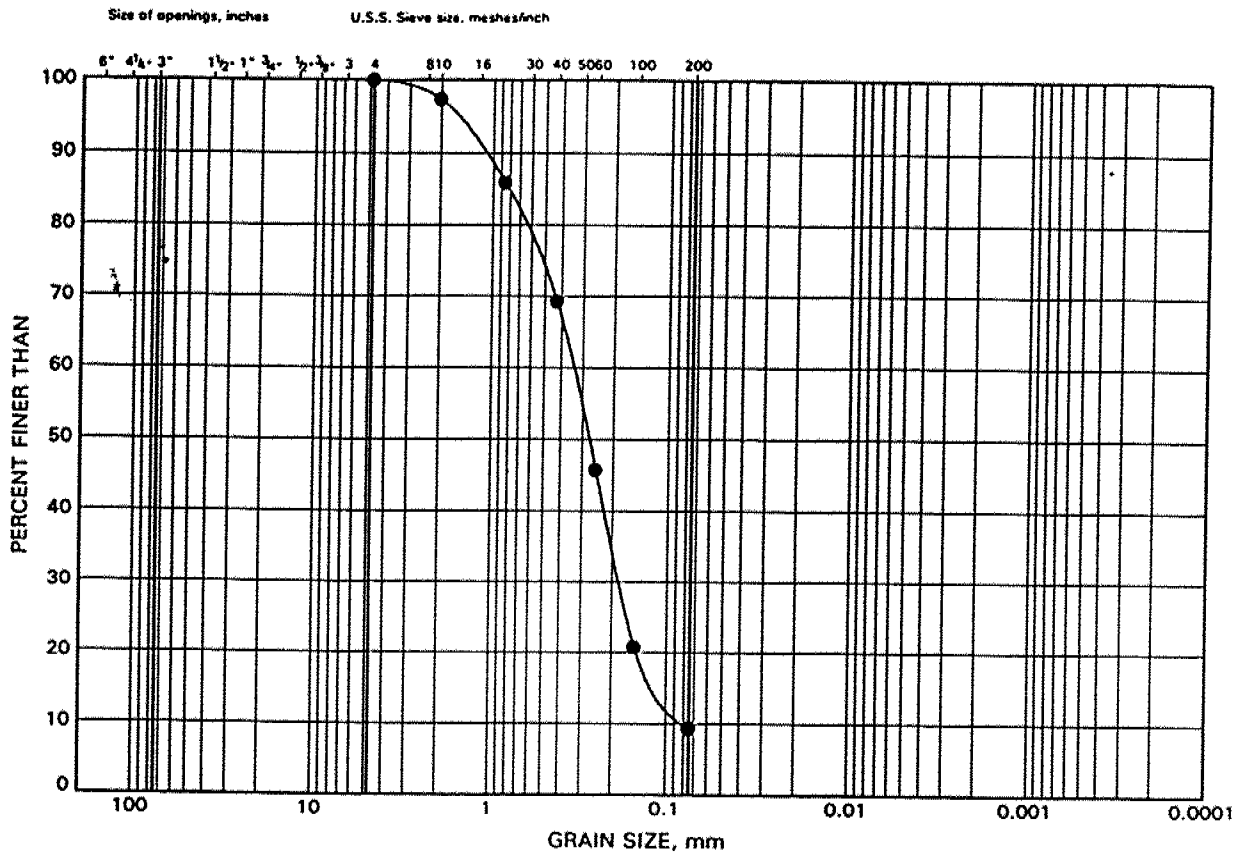
Date August 1998
Project 462-93-00



Prep'd WM
Chkd. AEG

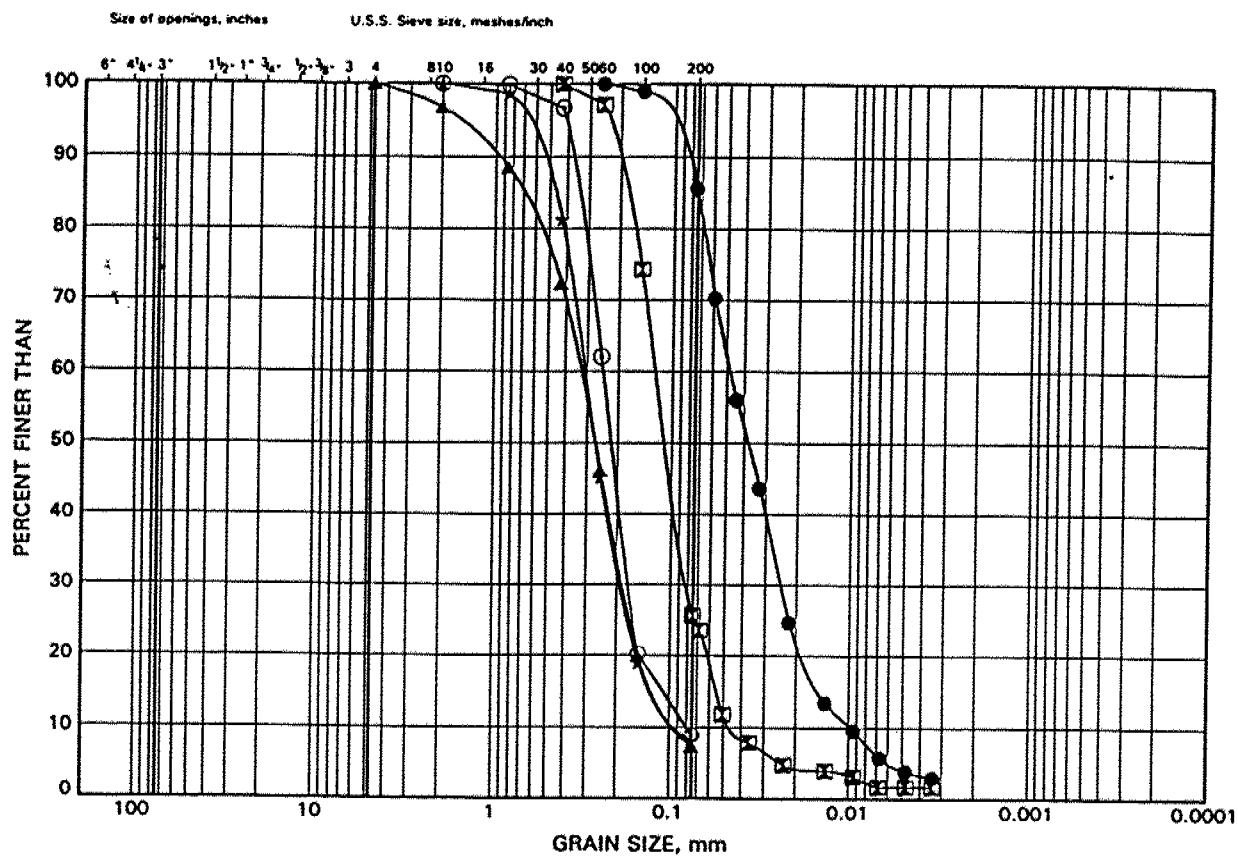
LITTLE EAST RIVER CROSSINGS GRAIN SIZE DISTRIBUTION

FIGURE B3



LITTLE EAST RIVER CROSSINGS GRAIN SIZE DISTRIBUTION

FIGURE B4



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

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	C-98-3	8.79	310.21
⊠	C-98-3	12.50	306.50
▲	C-98-3	18.59	300.41
★	C-98-3	27.74	291.26
⊙	C-98-3	36.88	282.12

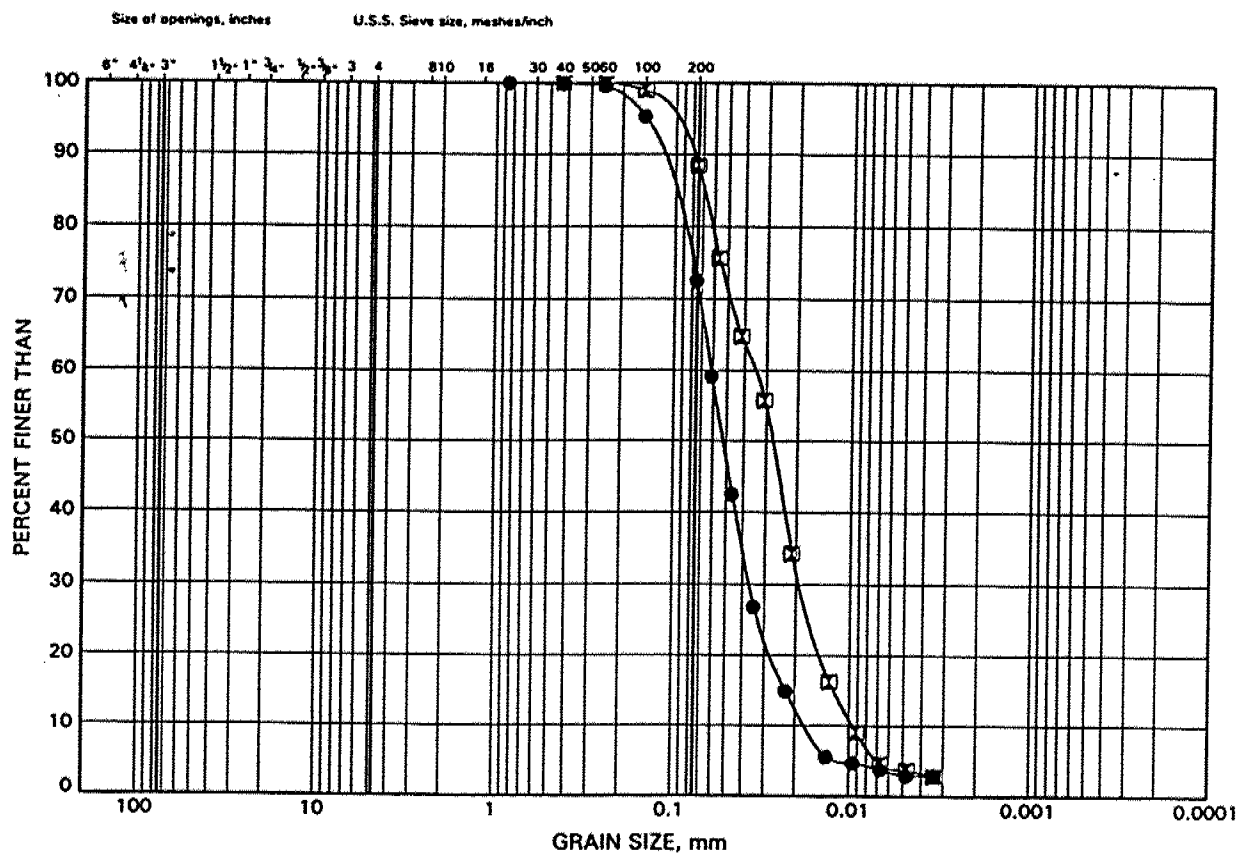
Date August 1998
Project 462-93-00



Prep'd WM
Chkd. AEG

LITTLE EAST RIVER CROSSINGS GRAIN SIZE DISTRIBUTION

FIGURE B5

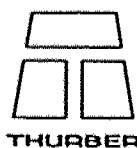


COBBLE SIZE	COARSE	FINE	COARSE	MEDIUM	FINE	SILT and CLAY
	GRAVEL		SAND			FINE GRAINED

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	C-98-4	4.88	314.42
□	C-98-4	6.34	312.96

THURBGS 3517 98/08/11

Date August 1998
Project 462-93-00



Prep'd WM
Chkd. AEG

Table 1

Results of pH and Sulphate Testing

Sample	Depth (m)	pH	Sulphates (ppm)
C-98-2 Sa 3	1.5 - 2.1	3.37	2620

APPENDIX C
GRADING OF SAND
for
BACKFILL OF PILES

The space around the piles in the integral abutment design should be filled with sand meeting the following grading:

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

APPENDIX D

NON-STANDARD SPECIAL PROVISIONS

NON-STANDARD SPECIAL PROVISION

Sheet 1 of 1

Date October 30, 1998

WP No _____ CONTRACT No _____ DISTRICT No _____ HWY No _____

LOCATION _____ TYPE OF WORK _____

1. This SP is new .

This non-standard special provision outlines the groundwater conditions on site and draws attention to the need for groundwater control during certain stages of construction.

2.

Item	Spec. No.	Title or Item Description
		Groundwater Conditions and Control

The foundation investigation conducted at this site revealed the presence of two groundwater regimes. The near surface soils contain a free groundwater table which is stabilized near ground surface. A piezometer installed at a depth of approximately 10 m recorded an artesian groundwater condition with a head $1.2 \pm$ m above original ground surface.

During all stages of construction, and especially during stripping, excavation and backfill, and pile driving, appropriate measures must be implemented to control the ground water to depress the free level to at least 1.0 m below the base of the excavation and to prevent disturbance of the soil and to ensure that the necessary construction activities can be carried out in dry conditions.

Pile driving must be undertaken in a manner which will minimize soil disturbance around the pile shaft and clay seal and sand filter must be constructed around the driven piles to minimize subsequent upward seepage of groundwater around the pile shafts.

3.

Initiated By _____

Detailed By _____

Approved By _____

MEMORANDUM



To: V. Minassian, P. Eng.
Senior Project Engineer
Planning and Design, Northern Region

September 9, 1998

From: Pavements and Foundations Section
Room 315, Central Bldg.

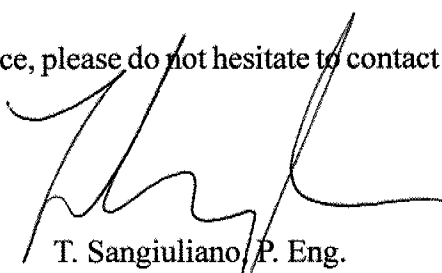
Tel: (416) 235-5267
Fax: (416) 235-5240

Re: Foundation Investigation Report Reviews
Little East River North & South
CNR Overhead (NBL)
WP 462-93-00
Hwy 11, From 6.7 km N of Hwy 60 Northerly 13.6 km
District 52, Huntsville

As requested in your memorandum dated December 3, 1998, we have reviewed the Final Foundation Investigation Reports for the Little East River Northbound and Southbound structures and the CNR Overhead structure. Comments derived from our review of the draft reports were submitted in our memoranda dated July 28, 1998 and September 9, 1998 for the CNR Overhead and the Little East River structures respectively.

In general, our previous comments have been addressed in the Final Foundation Reports. Only the figures illustrating the grain size distribution curves have not been changed. As stated previously, conventionally figures are presented based on soil type rather than borehole and we have no further comments.

We have no further comments. If you require additional assistance, please do not hesitate to contact our office.



T. Sangiuliano, P. Eng.
Foundation Engineer

for

D. Dundas, P. Eng.
Senior Foundation Engineer

cc. T. Kazmierowski

MEMORANDUM



To: V. Minassian, P. Eng.
Senior Project Engineer
Planning and Design, Northern Region

September 9, 1998

From: Pavements and Foundations Section
Room 315, Central Bldg.

Tel: (416) 235-5267
Fax: (416) 235-5240

Re: Foundation Investigation Report Review
Little East River North & South
WP 462-93-00
Hwy 11, From 6.7 km N of Hwy 60 Northerly 13.6 km
District 52, Huntsville

As requested in your memoranda dated August 26, 1998 and September 3, 1998, we have reviewed both the original and revised draft reports prepared by Thurber Engineering for the proposed Little East River - South & North structures. Our review comments are contained in this memorandum.

Our review is based on verifying that the Foundation Investigation and Design Report satisfies the terms of reference for completeness. Accordingly, our review consists of commenting that the terms of reference have been fully addressed, partially addressed or not addressed. The Consultant is responsible for the technical accuracy of the recommendations contained in the report. Any deficiency identified in this memorandum is intended to alert the Consultant but shall not relieve the Consultant of any responsibility for their work..

Proposed Structures

Two separate structures are proposed to carry the Hwy 11 NBL over the meandering Little East River at Stations 24+030 and Stations 24+933. Both structures are single span structures with proposed spans of 18.3 m. Approach fills to the bridges are approximately 4 metres in height.

Subsurface Conditions

In general, the subsurface conditions at the site are comprised of a surficial thickness of peat ranging in thickness from 2.1 m to 3.0 m at the northerly site and from 2.1 m to 4.0 m at the southerly site. The surficial stratum of peat is underlain by extensive thicknesses of cohesionless soils submerged below the prevailing groundwater table. At the northerly site, a deposit of silt of thickness 8 to 10 metres underlies the peat and in turn is underlain by a sand deposit that was explored to a depth of 43 metres below the original ground surface. At the southerly site, the surficial stratum of peat is underlain by sequential silt, silt and sand deposits of thicknesses of approximately 29 to 38m and 5 to 12 m. The silt and sand deposit at the site is in turn underlain by a sand deposit that was explored for a thickness of approximately 2.5 metres.

The cohesionless soils have a denseness ranging generally from loose to compact. At the lower

depths of the investigation, the material denseness increases to dense with SPT 'N' values ranging from 35 blows/0.3m to 50 blows/0.3m. Very dense material and end bearing refusal material was not encountered at either site.

In addition, a cohesive clayey silt layer was encountered at both sites that appears to be an aquiclude or aquitard. This stratum is situated at a depth of approximately 5 to 7 metres below the ground surface and is approximately up to 1.5 metres in thickness. It was observed that the aquitard is confining the groundwater in the lower levels of the silt and sand deposits and hence there exists artesian conditions at both sites up to 1 metre above the ground surface.

Factual Component of the Report

In general, the factual components of both reports that contain the Site Description, Investigation Procedure(Field Investigation and Laboratory Analyses), Subsurface Conditions and Groundwater Conditions are properly formatted and address the terms of reference specified in the RFP. However, the figures contained in Appendix do not conform to MTO standards. The figures should be presented based on soil type rather than by borehole. Appropriate soil classification titles should be included and the sample number should be included.

Not Addressed

Discussion and Recommendations Component of the Report

The Foundation Investigation and Design Report only partially addresses the terms of reference for the design and construction of structure foundations and related earthworks. In view of the absence of a competent end bearing material within the practical depth of the investigation and the presence of an artesian condition, special attention is required in the design and construction of the structure foundations. In this regard, we forward the following comments and questions:

Structure Foundations

Pile Capacity

1. The report should clarify whether the piles are predominantly friction piles. ✓
2. Static analysis has been employed to predict the axial resistance of the piles. Static analysis is not the most accurate method of predicting axial pile capacity. Based on pile load test data available in the MTO Foundations Pile Load Test database report, lower axial capacities were obtained at a site with similar subsurface conditions and pile type. The merit of verifying the recommended axial pile capacities by reviewing pile load test data or perhaps conducting a site specific pile load test should be discussed with the Consultant. ✓
3. In the computation of lateral resistance, the coefficient of horizontal subgrade reaction is a parameter required when finite element modelling techniques are employed. Horizontal subgrade reaction parameters have not been included in the report. ✓

Pile Driving

1. The ultimate resistance of a driven pile using the Hiley Dynamic Formula is taken as two(2) times the Factored Capacity at ULS. ✓
2. Pile Tip Elevations are given as minimum values. For piles as required at the site, there exists a risk that the axial capacity at the design tip elevation will not be achieved. This leaves the potential for significantly larger embedment lengths than estimated. ✓

Pile Installation

The pile installation for piles that derive their supporting capacity from the soil surrounding the shaft of the pile in artesian conditions is critical. Appropriate special provisions shall be included in the contract documents to ensure the quality assurance of the installation of the deep foundation units. An end result special provision for Piling that includes a requirement for a certificate of conformance can be obtained from our office.

Earth Pressure

At the Little East River South structure the report correctly states that "an additional lateral pressure of 16 kPa should be added to account for compaction induced forces". However, at the Little East River North structure, the report states that "design lateral pressures at any depth should not be less than 16 kPa to account for compaction induced forces". The reports should accurately and consistently address lateral earth pressures due to compaction in accordance with Section 6-7 of the OHBDC. ✓

Excavation and Groundwater Control

It is recommended that an NSSP be included in the Contract Documents that alerts the Contractor of the subsurface and groundwater conditions at the site and that the Contractor is responsible for the necessary subexcavation and placement of embankment fill in the dry.

Stamped and Sealed Reports

The reports are to be signed and sealed by two professional Engineers. In this regard, the reports do not address this requirement.

We trust these comments are sufficient for your purposes. If you require additional assistance, please do not hesitate to contact our office.

T. Sangiuliano, P. Eng.
Foundation Engineer
for
D. Dundas, P. Eng.
Senior Foundation Engineer

cc. T. Kazmierowski

MEMORANDUM



To: D. Smith, P. Eng.
Project Soils Engineer
Geotechnical Section, Northern Region

August 19, 1998

From: Pavements and Foundations Section
Room 315, Central Bldg.

Tel: (416) 235-5267
Fax: (416) 235-5240

Re: Embankment Design
WP 462-93-00
Hwy 11, From 6.7 km N of Hwy 60 Northerly 13.6 km
District 52, Huntsville

We have received a copy of the John Emery Geotechnical Engineering Ltd (JEGEL) Draft Pavement Design Report under your covering memorandum dated August 7, 1998. Our comments regarding the recommendation for the mechanically stabilized earth embankments are contained in this memorandum.

Background

John Emery Geotechnical Engineering Ltd (JEGEL) were retained by McCormick Rankin Corporation to carry out a geotechnical investigation for the pavement design and associated works for this project. The scope of the investigation is described in the abovementioned Draft Pavement Design Report.

The report addresses pavement design, aggregate sources, suitability of excavated materials and other issues generally within the purview of the Geotechnical Office. However, the report also includes recommendations for the design of mechanically stabilized earth embankments at two locations. One location is between Stations 22+000 to 22+200 and the second between Stations 22+400 and 22+750. Within these two locations, geometric constraints restrict the slope to a maximum steepness of 2H:1V.

Discussion and Recommendations

Foundation engineering recommendations for the design and construction of roadway embankments must be provided by eligible foundation engineering consultants. It is understood that the Regional Geotechnical Section administers low risk/low complexity foundation works. Low risk/low complexity embankment/excavation cut designs are defined as embankments or excavation cuts that are less than 4.5 metres in height and where requirements for stability and settlement analyses are minimal.

For medium and high complexity foundation work, eligible foundation engineering consultants must

be retained to conduct the necessary foundation investigation and to provide the recommendations based on stability analyses and settlement calculations. The design and construction of mechanically stabilized earth embankments are considered of medium to high complexity.

JEGEL is not registered as an eligible Foundation Engineering consultant and consequently are not eligible to provide the recommendations for the design and construction of the mechanically stabilized earth embankments on this project.

It is recommended that the two areas of embankments be reviewed and investigated by an eligible Foundation Engineering Consultant. Subsurface information and the ground surface must be sufficient and properly presented to conduct the embankment stability assessment and analyses at the site. Depending on the physical and mechanical properties of the native subsoils and the proposed embankment fill heights, an embankment employing conventional rockfill or earthfill materials may adequately address the 2H:1V geometry constraint.

At the site, the native subsoils are comprised of loose to very dense silty sands to sandy silts. In our opinion, conventional embankment earth/rock fills up to approximately eight(8) could conceivably be designed to satisfy the 2H:1V geometry constraint.

We trust these comments are sufficient for your purposes. If you require additional assistance, please do not hesitate to contact our office.

A handwritten signature in black ink, appearing to read 'T. Sangiuliano', with a stylized, flowing script.

T. Sangiuliano, P. Eng.
Foundation Engineer

for

D. Dundas, P. Eng.
Senior Foundation Engineer

cc. T. Kazmierowski

GEOCRES No. _____

DIST. 52 REGION _____W.P. No. 466-93-00

CONT. No. _____

W. O. No. _____

STR. SITE No. _____

HWY. No. 11LOCATION Proposed Culvert @Sta. 19+984 Median CenterlineNo of PAGES -OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS: _____

**FOUNDATION INVESTIGATION AND DESIGN REPORT
FOR PROPOSED CULVERT
AT STATION 19+984 MEDIAN CENTRELINE
STRUCTURE SITE NO.44-304
DISTRICT 52, HUNTSVILLE
W.P. 466-93-00**

Submitted To:

**Delcan Corporation
133 Wynford Drive
North York, Ontario M3C 1K1
Canada**

Submitted By:

**AGRA
104 Crockford Blvd.
Scarborough, Ontario, M1R 3C6
Canada**

**February 2000
TT98820**

February 18, 2000
Ref. No.: TT98820

Delcan Corporation
133 Wynford Drive
North York, Ontario, M3C 1K1
Canada

Attention: Mr. Khaled El-Dalati, P. Eng.
Manager, Transportation and Design

Dear Sir:

**Re: FOUNDATION INVESTIGATION AND DESIGN REPORT
FOR
PROPOSED CULVERT AT STATION 19+984 MEDIAN CENTRELINE
STRUCTURE SITE NO.44-304
DISTRICT 52, HUNTSVILLE
W.P. 466-93-00**

We take pleasure in enclosing eight (8) copies of our Foundation Investigation and Design Report carried out for the above mentioned project and we will be glad to discuss any questions arising from this work.

Soil samples will be retained for a period of one year, and will thereafter be disposed of unless we are otherwise instructed.

We thank you for giving us this opportunity to be of service to you.

Sincerely,

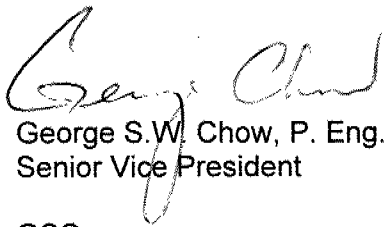

George S.W. Chow, P. Eng.,
Senior Vice President
GSC

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APPENDIX A

Explanation of Terms Used in Report

FIGURES

GRAIN SIZE DISTRIBUTION CURVES 1 - 2

ENCLOSURES

BOREHOLE LOG SHEETS
BOREHOLE LOCATIONS AND SOIL STRATA

DWG. NO. 1

1.0 INTRODUCTION

AGRA, Consulting Geotechnical Engineers, has been retained by Delcan Corporation (Delcan) to conduct a foundation investigation at the site of a proposed rigid frame concrete culvert to be used to realign the water flow of the existing P-3 Tributary. The culvert of 130 m in length will cross the proposed Highway 11 median centreline at about Station 19+984. The proposed works are part of the Highway 11 Four Laning Project, from Emsdale to Burk's Falls, W.P. 466-93-00, District 52, Huntsville, Ontario.

The purpose of this investigation is to obtain more detailed information about the subsurface conditions at the site of the proposed culvert by means of exploratory boreholes. Based on our interpretation of the data obtained from this and previous geotechnical investigations carried out in the vicinity, recommendations for the foundation design of the proposed culvert are provided. Comments are also provided on anticipated construction issues where they may affect the design of the proposed works, from a geotechnical point of view.

At the time of this investigation, the proposed, revised horizontal alignment of the culvert and the existing ground surface profile along the P-3 Tributary were provided to us on plan and profile by Delcan via facsimile transmission on August 10, 1999. The terms of reference for our scope of work are as outlined in our proposal letter, dated August 18, 1999.

2.0 SITE DESCRIPTION AND PHYSIOGRAPHY

The site is located about 900 m south of Star Lake Road at the existing P-3 Tributary crossing of the proposed Highway 11. The existing ground surface elevation at the proposed culvert location slopes gently in a easterly direction along the tributary, from about Elevations 324 m to 323 m. The surrounding area is generally moderately wooded with trees and brushes. The water in the creek is up to about 1 m deep. The proposed grade of Highway 11 above the culvert is at about Elevation 341 m for both the NBL and SBL.

Based on available geologic information, the site is in an area of ice-contact sediments. Generally after the last glacial withdrawal, ice-contact sediments (sands and gravels) followed by glaciofluvial sediments (ranging from deltaic and nearshore sands and gravels to prodeltaic and lake bottom silts and clays) were deposited on top of the existing sandy glacial till or Precambrian bedrock. The area was then inundated by glacial Lake Algonquin, depositing sands, silts and clays in low lying areas. The bedrock generally consists of strongly foliated gneissic to migmatic rocks of the Central Gneiss Belt, which is part of the Grenville Province (a structural subdivision of the Canadian Shield).

3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out during the period of August 19, 20, 23 and 24, 1999, and consisted of drilling and sampling five boreholes (Borehole Nos. 1001 to 1005, inclusive) to depths of 6.0 to 15.3 m below the existing ground surface.

The plan locations of the boreholes along with a stratigraphic section parallel to the culvert alignment are shown on Drawing No. 1. Details of subsurface conditions encountered at each borehole location, including the results of in-situ testing, are presented on the Record of Borehole sheets.

The boreholes were advanced, using a combination of hollow stem continuous flight augers, casings, wash boring and coring equipment, with a track-mounted power auger drill rig (BOA 6M2) owned and operated by Groundworks Drilling Inc., under the full-time supervision of experienced geotechnical personnel from AGRA.

Sampling in the boreholes were carried out at regular intervals of depth by the Standard Penetration Test Method (SPT), as specified in ASTM Method D 1586. This consists of freely dropping a 63.5 kg hammer for a vertical distance of 0.76 m to drive a 51 mm diameter outside diameter split barrel (split-spoon) sampler into the ground. The number of blows of the hammer to drive the sampler into the relatively undisturbed ground for a vertical distance of 0.30 m is recorded as the Standard Penetration Resistance or the 'N'-value of the soil, and this gives an indication of the consistency or the compactness condition of the soil deposit.

In order to advance the boreholes through cobbles and boulders and to prove bedrock, rotary core drilling was carried out in Boreholes 1001 and 1005 utilizing NW size casings and cores were retrieved using an NXL size core barrel.

The borehole locations were established in the field by our engineering staff, in relation to the proposed centreline of Highway 11 already staked out by Dearden and Stanton Limited (retained by Delcan). Due to restrictions by the topography and the vegetation, all five boreholes were positioned along the south bank of the tributary. The borehole co-ordinates and elevations were later taken by Dearsen and Stanton Limited.

The soil samples were transported to our geotechnical laboratory in Toronto (Scarborough) for further examination and classification. A laboratory testing programme, consisting of natural moisture content determinations and grain size analyses, was performed on selected representative soil samples. The results of the laboratory tests are presented on the appropriate Record of Borehole Sheets and also on Figure Nos. 1 and 2, inclusive.

The boreholes were left open until the end of each work day to enable us to take additional water level readings. The boreholes were adequately grouted on completion.

4.0 SUBSURFACE CONDITIONS

The subsurface conditions were explored at five boreholes (Borehole Nos. 1001 to 1005) during the current investigation. Boreholes SB5 and G1 from previous investigations were utilized. The plan locations of the boreholes along with the stratigraphic section along the culvert alignment are shown on Drawing No. 1. Details of subsurface conditions encountered at each borehole location, including the results of in-situ testing, groundwater observations and laboratory test results are presented on the Record of Borehole sheets. The subsurface conditions are summarized in the following.

In general, the subsurface stratigraphy comprises surficial peat and/or topsoil overlying loose to compact sand, which is in turn underlain by dense to very dense sand and gravel to gravelly sand with frequent cobbles and boulders. The depth to the sand and gravel remains relatively constant from the west culvert limit to about the east crest of the SBL, but increases towards the east. The groundwater level is within 1 m depth of the existing ground surface.

4.1 Peat and Topsoil

Peat of 0.8 m to 1.0 m in thickness was encountered at ground surface in Boreholes 1001 and G1.

Topsoil was encountered in Boreholes 1002 to 1005 and SB5, ranging in thickness from 0.15 m to 0.4 m.

4.2 Sand

Below the surficial topsoil or peat, a cohesionless sand deposit with trace to some gravel was encountered to depths of about 9.8 m to 12.8 m in Boreholes 1001, 1002 and G1, and to depths of 4.9 m to 7.0 m in Boreholes 1003 to 1005, and SB5. Occasional sand and gravel to gravelly sand interlayers were present within this deposit.

One grain size analysis was conducted on a sample of each of the sand, sand and gravel and gravelly sand. The grain size curves are presented on Figures 1 and 2. For the sand, the results indicate 0% gravel, 95% sand, 5% silt and 0% clay size particles.

Most measured 'N'-values within the sand in Boreholes 1001, 1002, 1005 and SB5 range from 10 to 21 blows per 0.3 m, indicating a typically compact condition; occasional loose zones are present with 'N' values less than 10 blows per 0.3 m. The sand is loose to very loose throughout Borehole 1003. In Borehole 1005, a high 'N'-value of 80 was measured at 1 m depth and may be attributed to probable cobbles. Measured moisture contents range from about 12 to 28%.

For the sand and gravel to gravelly sand interlayers, the results indicate 28 to 41% gravel, 52 to 64% sand, 7 to 8% silt and 0% clay. It is noted that the cobbles and boulders could not be sampled with the spoon sampler.

4.3 Sand and Gravel to Gravelly Sand

A layer of sand and gravel to gravelly sand underlies the upper sand in Boreholes 1001, 1002 and 1005. Frequent cobbles and/or boulders were inferred or encountered within this layer. This layer extends to the full depth of Boreholes 1001, 1002 and is about 3.3 m thick in 1005. Auger refusal was encountered below the upper sand in Boreholes 1003, 1004, G1 and SB5 at levels which may be inferred as the upper surface of the cobbles and/or boulders. Measured 'N'-values range from 21 to greater than 50 blows per 0.3 m, indicating a compact to very dense, but typically dense to very dense condition. Measured moisture contents range from 12 to 17%.

One grain size distribution analysis was conducted on a sample from this cohesionless deposit, and the resulting grain size curve is presented in Figure 2. The analysis indicates 24% gravel, 48% sand, and 28% silt and clay size particles.

4.4 Bedrock

Bedrock was encountered and cored in Borehole 1005 from 8.2 to 11.0 m depths below existing ground surface. The recovered core samples show that the Precambrian bedrock consists of a massive, moderately closely to closely jointed gneiss with occasional micaceous layer. The percentage of core recovery varies from 78 to 100%. The Rock Quality Designation (R.Q.D.) values increase with depth from 42 to 76%. Based on these values and visual examination of the cores, the rock is considered to be of poor to good quality.

4.5 Groundwater Conditions

Groundwater conditions were observed in the open boreholes during the drilling and at the completion of each borehole. Observed groundwater levels in the open boreholes are within 1 m of the existing ground surface. It should, however, be pointed out that the groundwater at the site would fluctuate seasonally and in response to severe weather events.

5.0 DISCUSSION AND RECOMMENDATIONS

This report contains the findings of our geotechnical investigation, together with our recommendations and comments. These recommendations and comments are based on factual information and are intended only for use of the design engineers. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. The anticipated construction conditions are also discussed, but only to the extent that they may influence design decisions. Construction methods discussed, however, express our opinion only and are not intended to direct the contractors on how to carry out the construction. Contractors should also be aware that the data and their interpretation presented in this report may not be sufficient to assess all the factors that may have an effect upon the construction.

The original design, as depicted on the July 21, 1999 plan, consists of a 3,000 mm by 1,500 mm rigid frame concrete box culvert of about 162 m in length. In order to reduce the length of the culvert, the design was revised (as depicted on the August 16, 1999 plan) and consisted of a rigid frame concrete open footing culvert of the same cross-sectional dimensions, but with an overall length of about 130 m. A rock fill embankment was to be used instead of an earth fill embankment. Subsequent telephone conversations with Delcan indicate that a rigid box culvert is now preferred.

The current proposed design, as depicted in the December, 1999 plan, consists of a 3,000 mm by 3,800 mm rigid frame box culvert of about 130 m in length. The culvert will be constructed with expansion joints that will separate the culvert into four sections of about 32.5 m in length each. Construction joints will be incorporated at regular intervals (approximately every 11 m) in order to attempt to mitigate the adverse effects of differential settlement. A rock fill embankment of up to 18 m in height with side slopes of 1.25H:1V and 2 m wide berms are to be constructed.

As explained in our proposal letter of August 18, 1999, the information obtained by geotechnical probes put down prior to this investigation indicated that the subsoils generally consist of water-bearing sands of loose to compact condition. Refusal to augering was encountered at about 6 m to 12 m depths due to possible cobbles, boulders and/or bedrock. The five boreholes drilled and sampled during this investigation provide additional subsurface information along the proposed culvert alignment. Auger refusal was encountered on the surface of a layer of cobbles and boulders, proven in three boreholes (near the limits of the proposed culvert) and inferred at the remaining locations. Along the culvert alignment, depth to the cobbles/boulders remains relatively constant at about 6 m below existing ground surface between the west limit and the east crest of the southbound lane (SBL), before increasing to 12 m to 13 m depths near the east limit of the culvert. Bedrock was proven at about 8 m depth at Borehole 1005, underlying the cobbles/boulders, near the west limit of the culvert. The groundwater level along the culvert alignment was observed at within 1 m below the existing ground surface.

The existing ground surface along the alignment of the proposed culvert slopes down gently from west to east. The proposed culvert is to be designed for a normal water level of Elevation 324.10 m and a flood level of Elevation 324.88 m.

5.1 Culvert Foundations on Improved Subgrade

The culvert is expected to be founded at about Elevations 322.7 m and 321.7 m at the west and east limits, respectively. The boreholes located to the east of the northbound lane (NBL) centreline encountered peat and organics at between 0.8 m and 1.1 m depths. In order to avoid excessive settlement, to provide a more uniform founding subgrade condition and to improve the load carrying capacity of the upper zones of the founding soils, we recommend that the peat, organics, the underlying sand and otherwise weak or unsuitable zones be removed to a depth of 2 m below the founding depth and replaced with compacted granular fill. At other locations along the culvert alignment, the required depth of sub-excavation and replacement with fill should be maintained at about 2 m below the founding depth, where appropriate.

Assuming an open cut excavation, the plan limits of the excavation base should be at least 2 m beyond the perimeter of the culvert base. The excavations will extend below the groundwater table and therefore groundwater control will be required. Provided adequate groundwater control measures are implemented, temporary excavation side slopes should be stable at an inclination of 2H:1V. Care must be exercised during excavation to avoid disturbing the founding subgrade.

When the excavation reaches the required depth, the subgrade should be inspected and approved by a geotechnical engineer contracted by the Contract Administrator. If necessary, the excavation may need to be deepened to a depth below any peat or organic layers. After its approval, the exposed subgrade at the base of the excavation may need to be compacted, if requested by the representative of the Contract Administrator, to achieve a density of not less than about 95% of the material's Standard Proctor Maximum Dry Density (SPMDD). The fill used to raise the grade inside the excavation should be Granular 'A' material placed when its moisture content is within $\pm 2\%$ of its optimum moisture content. It should be placed in loose lifts not exceeding 200 mm in thickness and should be uniformly compacted to not less than 100% of its SPMDD.

A factored geotechnical resistance at U.L.S. of 500 kPa and a geotechnical resistance at S.L.S. equal to 350 kPa can be assigned to the founding Granular 'A' subgrade prepared in this manner. For the west portion of the culvert (under the SBL embankment), the serviceability condition corresponds to total and differential settlements (between the two limits) in the order of 50 mm and 20 mm, respectively. For the east portion of the culvert (under the NBL embankment), the serviceability condition corresponds to total and differential settlements (between the two limits) in the order of 70 mm and 30 mm, respectively. For a serviceability condition corresponding to a total settlement of 25 mm or less, the geotechnical resistance at S.L.S. is equal to 125 kPa.

The culvert should be designed to resist frost forces, weight of embankment fill and traffic loadings. It is noted that compression of the native sand subgrade induced by up to 18 m of embankment rock fill (Elevations 341 to 323 m) will be in the order of 75 mm at the west end of the culvert, and

150 mm at the east end of the culvert. This global settlement as well as the differential settlement from west (SBL) to east (NBL) should be considered in designing the culvert.

It is recommended that construction joints be introduced at regular intervals along the culvert alignment in order to mitigate the adverse effects of differential settlement. Further, consideration should be given to culvert camber, to achieve more uniform final grades along the alignment.

The unfactored horizontal resistance against sliding between concrete and Granular 'A' type material can be calculated using a friction angle of 32 degrees.

In view of the above design constraints, as well as to enhance structural rigidity, we recommend that preference be given to the rigid box culvert alternative.

Along the culvert alignment, some form of groundwater control will be required during construction as the proposed bottom of excavation is below the water table. Dewatering will need to be capable of drawing the water level to no less than 0.6 m below the bottom of the excavation. If the water table is not properly lowered, the granular soil at the bottom of the excavation can lose its load carrying capacity (in addition to the instability of the side slopes). If this happens and the engineered fill is placed on disturbed and loosened subgrade, excessive settlements can occur after construction of the culvert. We therefore recommend that the contractor investigate the position of the water table before starting the excavation to assess required dewatering. As discussed before, the water levels at the time of our investigation were generally recorded at within 1 m depth below existing grade. Pumping from properly filtered sumps and wells could be appropriate methods of controlling the groundwater.

It is recommended that a dewatering NSSP be included in the contract documents which requires that the Contractor be fully responsible for the proposed dewatering scheme.

5.2 Culvert Foundations on Piles

If the above settlement tolerances are unacceptable, consideration could be given to supporting the culvert on deep foundations in the form of steel H-piles, driven to practical refusal within the upper portion of the very dense sand and gravel with frequent cobbles and boulders. In order to adequately penetrate the sand and gravel as well as occasional cobbles at shallower depths, a heavier section such as HP 310x79 with reinforced tips would be suitable for use.

It is considered likely that the driven H-piles will not be able to penetrate deep into the very dense sand and gravel with frequent cobbles and boulders. Based on the results of the boreholes, the following Table 1 summarizes the estimated average pile tip elevations that may be assumed for design purposes.

TABLE 1

SUPPORT LOCATION	REFERENCE BOREHOLE	ESTIMATED APPROXIMATE PILE TIP ELEVATION (depth) (m)
Station 19+945 60m Rt Med. C/L	1001	309 (14)
Station 19+945 38m Rt Med. C/L	1002	311 (12)
Station 19+975 Med. C/L	1003	314 (9)
Station 19+990 32m Lt Med. C/L	1004	315 (8)
Station 20+010 60m Lt Med. C/L	1005	315 (8)

The borehole results indicate that the founding sand and gravel deposit containing cobbles and/or boulders was encountered at higher elevations in the vicinity of the SBL compared with those in the vicinity of the NBL. Therefore, it may be expected that the piles will terminate at higher elevations within the SBL. In the vicinity of the west limit of the culvert, however, the driven piles may penetrate through the sand and gravel and seat on the bedrock.

5.2.1 Resistance to Axial Loads

For HP 310x79 steel H-piles driven to practical refusal within the very dense sand at or below the elevations shown in Table 1 above, the following axial resistances may be assumed for design.

$$\begin{aligned} \text{Factored Axial Resistance at Ultimate Limit States (U.L.S.)} &= 1,150 \text{ kN} \\ \text{Geotechnical Resistance at Serviceability Limit States (S.L.S.)} &= 1,000 \text{ kN} \end{aligned}$$

The piles should be driven with a suitably heavy hammer capable of delivering a rated capacity of between 40 and 50 kJ per blow. The driving of the piles should be controlled by a recognized pile driving formula, such as the Hiley Formula. The estimated ultimate resistance of the piles driven to practical refusal within the very dense sand at about the elevations quoted above, as given by the Hiley Formula, is approximately 2,300 kN. This value was arrived at by dividing the factored axial resistance at U.L.S. by a resistance factor of 0.5, as per current MTO practice.

Cobbles and/or boulders were inferred or encountered within the very dense sand and gravel in the boreholes drilled at the abutment locations. In view of this and the hard driving conditions anticipated, the pile tips should be reinforced, as per MTO Standards (OPSD 3301.00) to minimize damage to the piles.

Oversize materials (e.g. greater than 75 mm nominal diameter) should not be used in the fills through which piles would be driven.

In accordance with MTO standard practice, the piles should be driven to about 2 to 3 m above the design elevations given in Table 1 and the driving should then be monitored and controlled by the Hiley Formula. If the driven pile encounters refusal above the recommended elevation, the Geotechnical Engineer appointed by the Contract Administrator should be notified immediately.

During the driving process, piles which have already been driven should be monitored to determine if they are heaving due to effects of driving adjacent piles. If this phenomenon occurs, the affected piles should be re-driven. It is recommended that not less than 15% of the piles be re-struck one to two days after initial installation, as a precaution against relaxation. If relaxation occurs, then all piles in that foundation element should be re-tapped.

It is possible that some of the piles may penetrate to one to two metres below the estimated tip elevations and this aspect should be taken into consideration when ordering piles.

The geotechnical resistance at Serviceability Limit States (S.L.S.) corresponds to settlement of the piles. Provided that the piles are designed and installed as recommended above, it is considered that the quoted S.L.S. value corresponds to no more than 25 mm of settlement.

5.3 Backfilling

Backfill arrangements around the culvert should be carried out as per OPSD 803.02. Backfill to the culvert should consist of free-draining, non-frost susceptible granular materials such as OPSS Granular 'A' or 'B'. The excavated material is not suitable for backfilling purposes due to the high (wetter than optimum) natural moisture contents. All granular fill should be placed in loose lifts not exceeding 200 mm thick and be compacted to at least 95% of its SPMDD.

Heavy compaction equipment should not be used adjacent to the walls and roof of the culvert. The height of the backfill to the culvert walls should be maintained equal on both sides of the structure during all stages of backfill placement.

Computation of earth pressures acting against rigid culvert walls should be in accordance with the Ontario Highway Bridge Design Code, 3rd Edition (1991). For design purposes, the following properties can be assumed for backfill:

Compacted Granular 'A'

Angle of Internal Friction $\phi = 35^\circ$ (unfactored)

Unit Weight = 22kN/m³

Coefficient of Lateral Earth Pressures:

Level Backfill	Backfill Sloping at 3H:1V	Backfill Sloping at 2H:1V
$K_a = 0.27$	$K_a = 0.34$	$K_a = 0.40$
$K_b = 0.35$	$K_b = 0.44$	$K_b = 0.50$
$K_o = 0.43$	$K_o = 0.56$	$K_o = 0.62$
$K^* = 0.45$	$K^* = 0.60$	$K^* = 0.66$

Compacted Granular 'B'

Angle of Internal Friction $\phi = 30^\circ$ (unfactored)

Unit Weight = 21 kN/m³

Coefficient of Lateral Earth Pressures:

Level Backfill	Backfill Sloping at 3H:1V	Backfill Sloping at 2H:1V
$K_a = 0.33$	$K_a = 0.42$	$K_a = 0.54$
$K_b = 0.41$	$K_b = 0.52$	$K_b = 0.64$
$K_o = 0.50$	$K_o = 0.66$	$K_o = 0.76$
$K^* = 0.57$	$K^* = 0.74$	$K^* = 0.86$

NOTE: K_a is the coefficient of active earth pressure

K_b is the backfill earth pressure coefficient for an unrestrained structure including compaction effects

K_o is the coefficient of earth pressure at rest

K^* is the earth pressure coefficient for a soil loading a fully restrained structure and includes compaction effects

The earth pressure coefficient adopted will depend on whether the structure is restrained or movements can be allowed such that the active state of earth pressure can develop. If the culvert walls are restrained (such as a rigid box structure) and does not allow lateral yielding, then at rest pressures should be used as per Clause C6-7.1 of the O.H.B.D.C., 3rd Edition.

The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients in accordance with Clause 6-7.4.3 of the O.H.B.D.C., 3rd Edition. Alternatively a compaction surcharge of 16 kPa can be included in the lateral earth pressure for the structural design of the culvert.

Vibratory equipment for use behind culvert walls and retaining walls should be restricted in size as per current MTO practice.

5.4 Embankment Stability

Provided that all recommended procedures regarding rock fill embankment design and construction are followed as per AGRA Pavement Design Report for W.P. 466-93-00, Highway 11 Four Laning, from 2.5 km South of Highway 518E, Northerly 7.3 km at Emsdale, the 18 m high embankment with slope inclination of 1.25 horizontal to 1 vertical, with appropriate benching, should be stable.

Provided that all the peat, surficial organic and otherwise unsuitable materials are removed before placing the embankment fill, and the subgrade is properly compacted from the surface as

.../...

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discussed above, the total settlement of the foundation material would be in the order of 75 mm to 150 mm (west to east) and should be substantially completed within two months of placing the embankment fill to its full height. This settlement will, however, occur concurrently as the rock fill is placed in lifts.

5.5 Construction

Excavations should be carried out in accordance with the Safety Regulations of the Province (i.e. Occupational Health and Safety Act O.Reg. 213/91). The boreholes show that the excavations can be expected to extend through peat, and loose sands below the groundwater table. Provided the water table is properly lowered, open cut excavations can be expected to stand temporarily at 2H:1V side slopes.

We recommend that any water flow in the existing water course be diverted away from the culvert excavation to enable the culvert construction and fill placement to be carried out in the dry. Major problems due to groundwater seepage are not anticipated, provided dewatering is carried out properly. Pumping from properly filtered sumps will be required to augment the dewatering system.

In order to avoid unbalanced loading on the culvert, the height of the rock fill around the culvert should be maintained equal on both sides throughout construction as much as practically possible.

Allowance should be made to place an approximately 150 mm thick layer of lean concrete on the subgrade surface, i.e. excavation base, within four hours of preparation and acceptance of the bearing soil. It should be pointed out that if the foundation soil is disturbed, excessive settlements can occur after structural loads are applied.

5.6 Frost Protection

The culvert design should ensure that a soil cover of 1.8 m or its thermal equivalent is required for frost protection of foundations.

5.7 Erosion Protection

Assuming the embankment is composed of rock fill, erosion protection should still be provided at the culvert inlet (including the slopes and sides) and outlet, especially if head and wing walls are not incorporated. We recommend the use of cutoff walls or seepage seals around the proposed culvert to prevent erosion of the enclosing soil. Where clay seal will be used, this should be at least 0.6 m thick. A rip-rap (rock) filter blanket at the outlet (minimum 0.5 m thick) will also be required.

Rip-rap should also be provided for an adequate distance from the culvert, both upstream and downstream, taking into account the anticipated flow rates and scouring caused by the stream. The stones should be adequately sized (maximum 0.5 m diameter, blocky and angular in shape) to prevent erosion of the stream bed. The rip-rap layer should cover all areas of the channel with which flood flows are likely to be in contact and the rip-rap layer should be at least 0.3 m thick.

.../...

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Assuming the size of rip-rap to be used is that set out in the Concrete Culvert Details Drawing, dated December 1999, a granular filter layer or geotextile is required beneath the rip-rap. Should a granular filter be used, the material should be graded such that it will prevent loss of soil particles and the collapse of the lining material (i.e. Granular 'B' Type I). The granular filter layer should be at least 150 mm thick. Should a geotextile be used, it should consist of non-woven Class II, F.O.S. 50 - 100 μ m. A toe for the filter and rip-rap protection should be provided at the edge of the lining and protective cover to key the lining into the natural ground to provide protection to erosion and scour.


Any footings constructed in the area of the stream should be located below the anticipated scour depth and/or protected by rip-rap blanket.

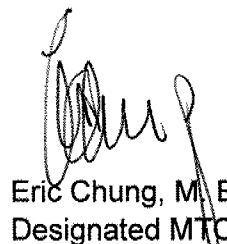
A qualified hydraulics engineer should be consulted to design the specifics of the channel, and culvert outlet and inlet (i.e. thickness and extent of protection).

6.0 CLOSURE

We recommend that once the details of the structures are finalized, our recommendations should be reviewed for their specific applicability.

Sincerely,


Sydney Pang, Ph.D., P. Eng.


Eric Chung, M. Eng., P. Eng.
Designated MTO Contact.




Andrew Drevininkas, P. Eng.



APPENDIX A

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND/OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
C_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_f	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kn/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m ³	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kn/m ³	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kn/m ³	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m ³	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m ³ /s	RATE OF DISCHARGE
γ_d	kn/m ³	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kn/m ³	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kn/m ³	SEEPAGE FORCE
γ'	kn/m ³	UNIT WEIGHT OF SUBMERGED SOIL						

FIGURES

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

SAND

GRAVEL

Fine

Medium

Coarse

Fine

Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)

1 2 3 4 5 10 20 30 40 50 75µm 150µm 300µm 600µm 1.18mm 2.36mm 9.5mm 19.0mm 37.5mm 63.0mm

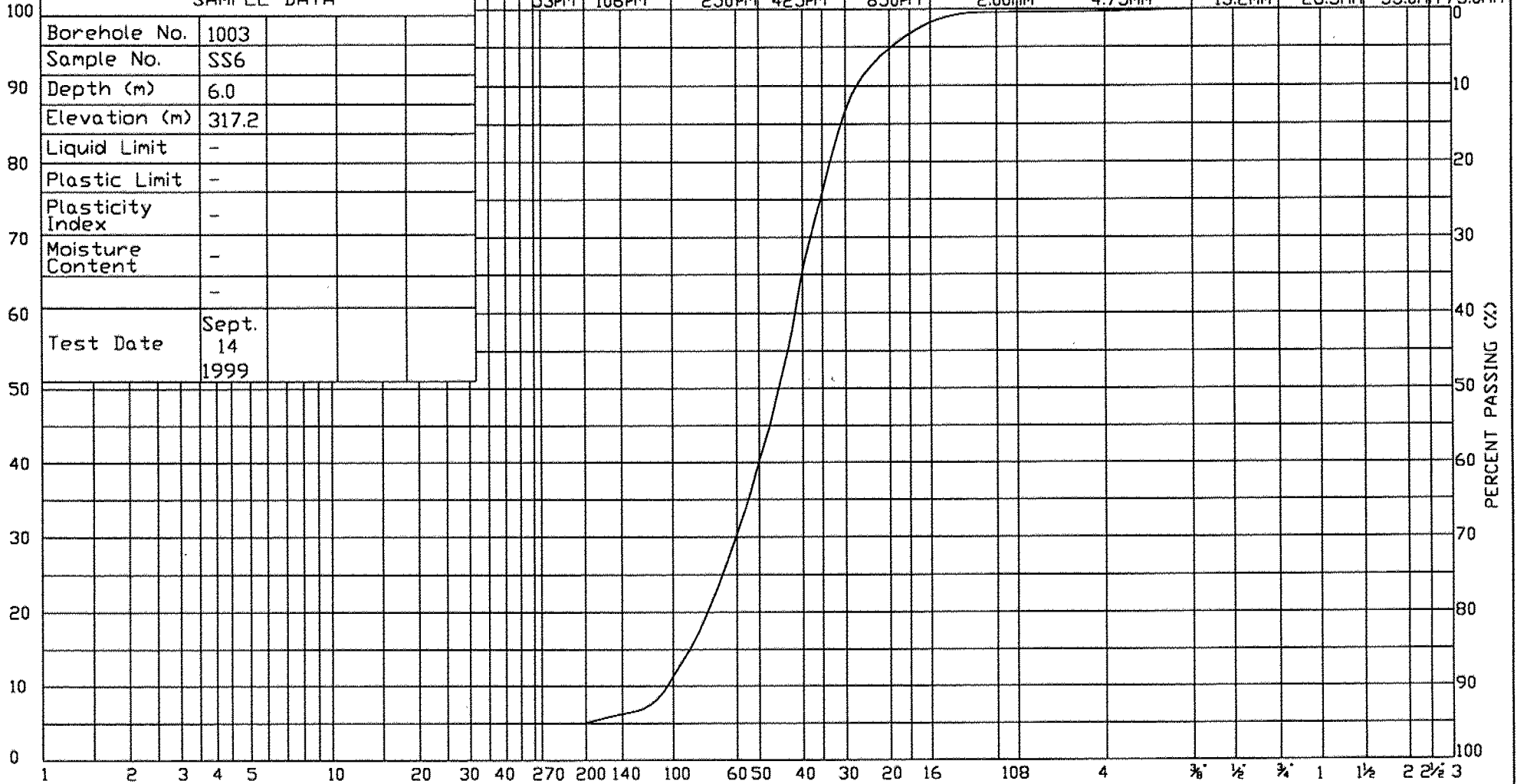
53µm 106µm 250µm 425µm 850µm 2.00mm 4.75mm 13.2mm 26.5mm 53.0mm 75.0mm

SAMPLE DATA

Borehole No. 1003
Sample No. SS6
Depth (m) 6.0
Elevation (m) 317.2
Liquid Limit -
Plastic Limit -
Plasticity Index -
Moisture Content -
Test Date Sept. 14 1999

PERCENT PASSING (%)

PERCENT PASSING (%)



MINISTRY SIEVE DESIGNATION (Imperial)



ENGINEERING GLOBAL SOLUTIONS

GRAIN SIZE DISTRIBUTION

SAND

1003: SS6

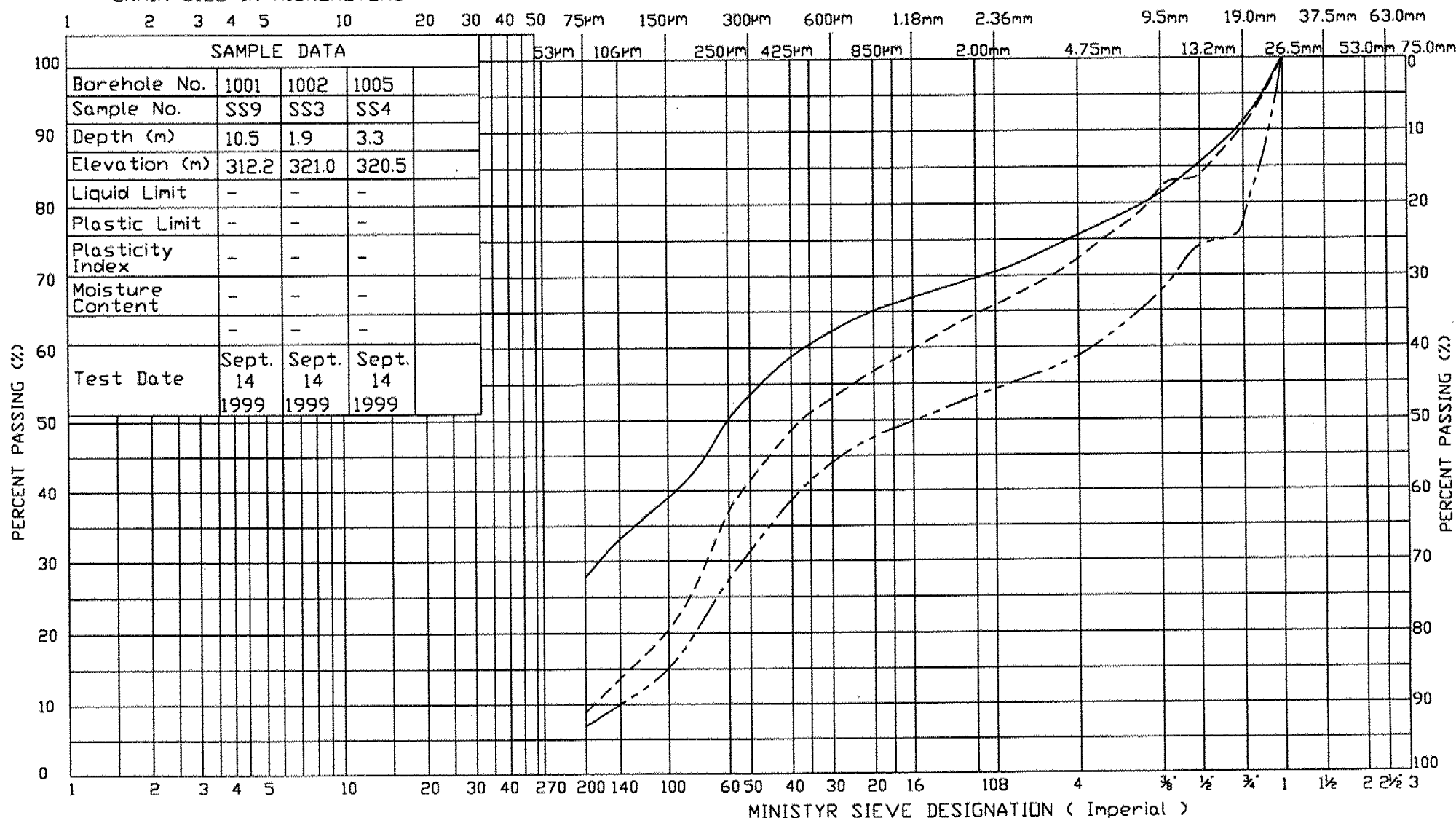
CLIENT:	DELCAN	
JOB NO.:	TT98820	W P 466-93-00
PROJECT:	HWY 11, EMSDALE	
LOCATION:	CULVERT @ STATION 19+984	
DATE:	SEPTEMBER 15, 1999	FIGURE: 1

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



AGRA

ENGINEERING GLOBAL SOLUTIONS

GRAIN SIZE DISTRIBUTION

SAND and GRAVEL
to
GRAVELLY SAND

1001: SS9	_____
1002: SS3	-----
1005: SS4	-----

CLIENT:	DELCAN
JOB NO.:	TT98820 W P 466-93-00
PROJECT:	HWY 11, EMSDALE
LOCATION:	CULVERT @ STATION 19+984
DATE:	SEPTEMBER 15, 1999
FIGURE:	2

ENCLOSURES

RECORD OF BOREHOLE No 1001

1 OF 2

METRIC

W.P. 466-93-00 LOCATION Site No. 44-304 N 5042393.3 E319147.3 ORIGINATED BY MA
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering / Wash Boring / Casing COMPILED BY AD
DATUM Geodetic DATE 20 August 1999 CHECKED BY SP/EYC

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	20 40 60 80 100					
323.4 0.0	PEAT		1	SS	1		323							Station 19+945 59m Rt Med C/L
322.6 0.8			2	SS	15		322							
	light brown SAND some Gravel, some Silt occasional Cobbles, some oxidized stains compact wet		3	SS	11		321							
			4	SS	7		320							
	loose		5	SS	13		319							
			6	SS	22		318							
	grey brown		7	SS	9		317							
			8	SS	11		316							
	trace Gravel loose		9	SS	41		315							
			10	SS	70		314							
313.6 9.8	grey SAND and GRAVEL to GRAVELLY SAND trace to some Silt frequent Cobbles and Boulders dense to very dense wet		11	SS	70/18		313							
							312							Started using NW casing @ 9m depth. 24 48 (28) Coring commenced @ 11.5m depth
							311							
							310							
							309							

Continued Next Page

+ 3 x 3. Numbers refer to
Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 1001

2 OF 2

METRIC

W.P.	466-93-00	LOCATION	Site No. 44-304 N 5042393.3 E319147.3	ORIGINATED BY	MA
DIST	52	HWY	11	BOREHOLE TYPE	Hollow Stem Augering / Wash Boring / Casing
DATUM	Geodetic	DATE	20 August 1999	COMPILED BY	AD
				CHECKED BY	SP/EYC

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT Y kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20					
308.2													
15.3	END OF BOREHOLE WL in open borehole on completion: 0.8m					308							

RECORD OF BOREHOLE No 1002

1 OF 1

METRIC

W.P. 466-93-00 LOCATION Site No. 44-304 N 5042382.2 E319131.9 ORIGINATED BY MA
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering COMPILED BY AD
DATUM Geodetic DATE 19 August 1999 CHECKED BY SP/EYC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE x LAB VANE						
323.3	0.15m TOPSOIL		1	SS	1		323							GR SA SI CL	
0.0			2	SS	20		322							41 52 7 0	
	Sand and Gravel		3	SS	14		321								
			4	SS	10		320								
	light brown SAND trace to some Gravel, some Silt compact wet		5	SS	16		319								
			6	SS	11		317								
	grey brown Gravelly Sand		7	SS	16		316								
			8	SS	5		314								
	fine Sand		9	SS	66		312								
312.8	grey SAND and GRAVEL to GRAVELLY SAND trace to some Silt frequent Cobbles and Boulders very dense wet						311								
10.5							310								
311.4	END OF BOREHOLE REFUSAL TO AUGER ADVANCE														
11.9															
311.0	END OF DCPT REFUSAL TO CONE ADVANCE														
12.3	WL in open borehole on completion: 0.8m														

+ 3, x 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 1003

1 OF 1

METRIC

W.P. 456-93-00 LOCATION Site No 44-304 N 5042372.9 E319083.1 ORIGINATED BY MA
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering COMPILED BY AD
 DATUM Geodetic DATE 19 August 1999 CHECKED BY SP/EYC


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							
324.0	0.2m TOPSOIL		1	SS	4		323								Station 19+975 2m Lt Med C/L
0.0	brown SAND trace Silt, trace Gravel very loose to loose wet		2	SS	3		322								
			3	SS	6		321								
			4	SS	8		320								
			5	SS	9		319								
			6	SS	7		318								
317.0			END OF BOREHOLE REFUSAL TO AUGER ADVANCE DCPT attempts at between 6.1m and 6.7m WL in open borehole on completion: 0.9m						317						
7.0															

RECORD OF BOREHOLE No 1004

1 OF 1

METRIC

W.P. 466-93-00 LOCATION Site No 44-304 N 5042362.5 E319049.7 ORIGINATED BY MA
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering COMPILED BY AD
 DATUM Geodetic DATE 24 August 1999 CHECKED BY SP/EYC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE							
324.4							20	40	60	80	100							
0.0	0.15m TOPSOIL		1	AS														
			2	AS														

RECORD OF BOREHOLE No 1005

1 OF 1

METRIC

W.P. 466-93-00 LOCATION Site No.44-304 N 5042361.5 E319020.3 ORIGINATED BY MA
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering / Wash Boring / Casing COMPILED BY AD
DATUM Geodetic DATE 23 August 1999 CHECKED BY SP/IEYC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w_p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w_L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL						
324.5							20	40	60	80	100					GR SA SI CL
0.0	0.15m TOPSOIL dark brown SAND, trace Silt and organics		1	SS	4											Station 20+010 55m Lt Med C/L
323.8																
0.7			2	SS	80											Probable cobble
	brown SAND some Gravel trace Silt compact to dense wet		3	SS	22											
			4	SS	31											28 64 8 0
	----- grey		5	SS	18											Auger refusal @ 5m depth, started using NW casing @ 5m depth.
319.7																
4.9	grey SAND and GRAVEL to GRAVELLY SAND trace Silt frequent Cobbles and Boulders		6	SS	21											
			7	SS	32											
			8	RC												Coring commenced @ 7.9m depth
316.3																
8.2			9	RC												RC 8 REC=83% RQD=42%
	GNEISS BEDROCK massive, moderately closely to closely jointed. occasional micaceous layer		10	RC												RC 9 REC=100% RQD=57%
			11	RC												RC 10 REC=78% RQD=72%
																RC 11 REC=100% RQD=76%
313.5																
11.0	END OF BOREHOLE															
	WL on completion: Not stabilized due to water used for coring, but likely at about 0.8m depth															

RECORD OF BOREHOLE No G1

1 OF 1

METRIC

W.P. 466-93-00 LOCATION Station 19+950 30m Rt Med C/L ORIGINATED BY MA
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem Augering COMPILED BY AD
DATUM Geodetic DATE 20 January 1999 CHECKED BY SP/EYC

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								
323.2 0.0	PEAT						323						
322.1 1.1							322						
	some Gravel						321						
							320						
							319						
							318						
							317						
	gray SAND trace Gravel wet						316						
							315						
							314						
	occasional cobbles						313						
							312						
							311						
310.4 12.8	END OF BOREHOLE REFUSAL TO AUGER ADVANCE												

RECORD OF BOREHOLE No SB5

1 OF 1

METRIC

W.P. 466-93-00 LOCATION Station 20+000 19m Lt Med C/L ORIGINATED BY MA
 DIST 52 HWY 11 BOREHOLE TYPE Solid Stem Augering COMPILED BY AD
 DATUM Geodetic DATE 20 January 1999 CHECKED BY SP/EYC

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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								WATER CONTENT (%)	
323.5							20	40	60	80	100						
0.0	0.4m TOPSOIL		1	SS	5												
			2	SS	14												
			3	SS	13												
			4	SS	10												
															</		

OVERSIZE
DRAWING(S)

**FOUNDATION INVESTIGATION REPORT
FOR PROPOSED CULVERT
AT STATION 19+984 MEDIAN CENTRELINE
STRUCTURE SITE NO.44-304
DISTRICT 52, HUNTSVILLE
W.P. 466-93-00**

Submitted To:

**Delcan Corporation
133 Wynford Drive
North York, Ontario M3C 1K1
Canada**

Submitted By:

**AGRA
104 Crockford Blvd.
Scarborough, Ontario, M1R 3C6
Canada**

**February 2000
TT98820**

February 18, 2000
Ref. No.: TT98820

Delcan Corporation
133 Wynford Drive
North York, Ontario, M3C 1K1
Canada

Attention: Mr. Khaled El-Dalati, P. Eng.
Manager, Transportation and Design

Dear Sir:

**Re: FOUNDATION INVESTIGATION REPORT
FOR
PROPOSED CULVERT AT STATION 19+984 MEDIAN CENTRELINE
STRUCTURE SITE NO.44-304
DISTRICT 52, HUNTSVILLE
W.P. 466-93-00**

We take pleasure in enclosing eight (8) copies of our Foundation Investigation Report carried out for the above mentioned project and we will be glad to discuss any questions arising from this work.

Soil samples will be retained for a period of one year, and will thereafter be disposed of unless we are otherwise instructed.

We thank you for giving us this opportunity to be of service to you.

Sincerely,

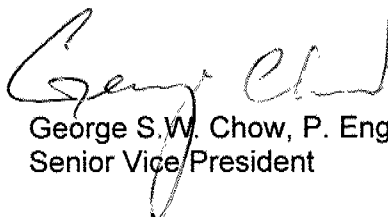

George S.W. Chow, P. Eng.,
Senior Vice President
GSC

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APPENDIX A

Explanation of Terms Used in Report

FIGURES

GRAIN SIZE DISTRIBUTION CURVES 1 - 2

ENCLOSURES

BOREHOLE LOG SHEETS

BOREHOLE LOCATIONS AND SOIL STRATA

DWG. NO. 1

1.0 INTRODUCTION

AGRA, Consulting Geotechnical Engineers, has been retained by Delcan Corporation (Delcan) to conduct a foundation investigation at the site of a proposed rigid frame concrete culvert to be used to realign the water flow of the existing P-3 Tributary. The culvert of 130 m in length will cross the proposed Highway 11 median centreline at about Station 19+984. The proposed works are part of the Highway 11 Four Laning Project, from Emsdale to Burk's Falls, W.P. 466-93-00, District 52, Huntsville, Ontario.

The purpose of this investigation is to obtain more detailed information about the subsurface conditions at the site of the proposed culvert by means of exploratory boreholes. Based on our interpretation of the data obtained from this and previous geotechnical investigations carried out in the vicinity, recommendations for the foundation design of the proposed culvert are provided. Comments are also provided on anticipated construction issues where they may affect the design of the proposed works, from a geotechnical point of view.

At the time of this investigation, the proposed, revised horizontal alignment of the culvert and the existing ground surface profile along the P-3 Tributary were provided to us on plan and profile by Delcan via facsimile transmission on August 10, 1999. The terms of reference for our scope of work are as outlined in our proposal letter, dated August 18, 1999.

2.0 SITE DESCRIPTION AND PHYSIOGRAPHY

The site is located about 900 m south of Star Lake Road at the existing P-3 Tributary crossing of the proposed Highway 11. The existing ground surface elevation at the proposed culvert location slopes gently in a easterly direction along the tributary, from about Elevations 324 m to 323 m. The surrounding area is generally moderately wooded with trees and brushes. The water in the creek is up to about 1 m deep. The proposed grade of Highway 11 above the culvert is at about Elevation 341 m for both the NBL and SBL.

Based on available geologic information, the site is in an area of ice-contact sediments. Generally after the last glacial withdrawal, ice-contact sediments (sands and gravels) followed by glaciofluvial sediments (ranging from deltaic and nearshore sands and gravels to prodeltaic and lake bottom silts and clays) were deposited on top of the existing sandy glacial till or Precambrian bedrock. The area was then inundated by glacial Lake Algonquin, depositing sands, silts and clays in low lying areas. The bedrock generally consists of strongly foliated gneissic to migmatic rocks of the Central Gneiss Belt, which is part of the Grenville Province (a structural subdivision of the Canadian Shield).

3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out during the period of August 19, 20, 23 and 24, 1999, and consisted of drilling and sampling five boreholes (Borehole Nos. 1001 to 1005, inclusive) to depths of 6.0 to 15.3 m below the existing ground surface.

The plan locations of the boreholes along with a stratigraphic section parallel to the culvert alignment are shown on Drawing No. 1. Details of subsurface conditions encountered at each borehole location, including the results of in-situ testing, are presented on the Record of Borehole sheets.

The boreholes were advanced, using a combination of hollow stem continuous flight augers, casings, wash boring and coring equipment, with a track-mounted power auger drill rig (BOA 6M2) owned and operated by Groundworks Drilling Inc., under the full-time supervision of experienced geotechnical personnel from AGRA.

Sampling in the boreholes were carried out at regular intervals of depth by the Standard Penetration Test Method (SPT), as specified in ASTM Method D 1586. This consists of freely dropping a 63.5 kg hammer for a vertical distance of 0.76 m to drive a 51 mm diameter outside diameter split barrel (split-spoon) sampler into the ground. The number of blows of the hammer to drive the sampler into the relatively undisturbed ground for a vertical distance of 0.30 m is recorded as the Standard Penetration Resistance or the 'N'-value of the soil, and this gives an indication of the consistency or the compactness condition of the soil deposit.

In order to advance the boreholes through cobbles and boulders and to prove bedrock, rotary core drilling was carried out in Boreholes 1001 and 1005 utilizing NW size casings and cores were retrieved using an NXL size core barrel.

The borehole locations were established in the field by our engineering staff, in relation to the proposed centreline of Highway 11 already staked out by Dearden and Stanton Limited (retained by Delcan). Due to restrictions by the topography and the vegetation, all five boreholes were positioned along the south bank of the tributary. The borehole co-ordinates and elevations were later taken by Dearsen and Stanton Limited.

The soil samples were transported to our geotechnical laboratory in Toronto (Scarborough) for further examination and classification. A laboratory testing programme, consisting of natural moisture content determinations and grain size analyses, was performed on selected representative soil samples. The results of the laboratory tests are presented on the appropriate Record of Borehole Sheets and also on Figure Nos. 1 and 2, inclusive.

The boreholes were left open until the end of each work day to enable us to take additional water level readings. The boreholes were adequately grouted on completion.

4.0 SUBSURFACE CONDITIONS

The subsurface conditions were explored at five boreholes (Borehole Nos. 1001 to 1005) during the current investigation. Boreholes SB5 and G1 from previous investigations were utilized. The plan locations of the boreholes along with the stratigraphic section along the culvert alignment are shown on Drawing No. 1. Details of subsurface conditions encountered at each borehole location, including the results of in-situ testing, groundwater observations and laboratory test results are presented on the Record of Borehole sheets. The subsurface conditions are summarized in the following.

In general, the subsurface stratigraphy comprises surficial peat and/or topsoil overlying loose to compact sand, which is in turn underlain by dense to very dense sand and gravel to gravelly sand with frequent cobbles and boulders. The depth to the sand and gravel remains relatively constant from the west culvert limit to about the east crest of the SBL, but increases towards the east. The groundwater level is within 1 m depth of the existing ground surface.

4.1 Peat and Topsoil

Peat of 0.8 m to 1.0 m in thickness was encountered at ground surface in Boreholes 1001 and G1.

Topsoil was encountered in Boreholes 1002 to 1005 and SB5, ranging in thickness from 0.15 m to 0.4 m.

4.2 Sand

Below the surficial topsoil or peat, a cohesionless sand deposit with trace to some gravel was encountered to depths of about 9.8 m to 12.8 m in Boreholes 1001, 1002 and G1, and to depths of 4.9 m to 7.0 m in Boreholes 1003 to 1005, and SB5. Occasional sand and gravel to gravelly sand interlayers were present within this deposit.

One grain size analysis was conducted on a sample of each of the sand, sand and gravel and gravelly sand. The grain size curves are presented on Figures 1 and 2. For the sand, the results indicate 0% gravel, 95% sand, 5% silt and 0% clay size particles.

Most measured 'N'-values within the sand in Boreholes 1001, 1002, 1005 and SB5 range from 10 to 21 blows per 0.3 m, indicating a typically compact condition; occasional loose zones are present with 'N' values less than 10 blows per 0.3 m. The sand is loose to very loose throughout Borehole 1003. In Borehole 1005, a high 'N'-value of 80 was measured at 1 m depth and may be attributed to probable cobbles. Measured moisture contents range from about 12 to 28%.

For the sand and gravel to gravelly sand interlayers, the results indicate 28 to 41% gravel, 52 to 64% sand, 7 to 8% silt and 0% clay. It is noted that the cobbles and boulders could not be sampled with the spoon sampler.

4.3 Sand and Gravel to Gravelly Sand

A layer of sand and gravel to gravelly sand underlies the upper sand in Boreholes 1001, 1002 and 1005. Frequent cobbles and/or boulders were inferred or encountered within this layer. This layer extends to the full depth of Boreholes 1001, 1002 and is about 3.3 m thick in 1005. Auger refusal was encountered below the upper sand in Boreholes 1003, 1004, G1 and SB5 at levels which may be inferred as the upper surface of the cobbles and/or boulders. Measured 'N'-values range from 21 to greater than 50 blows per 0.3 m, indicating a compact to very dense, but typically dense to very dense condition. Measured moisture contents range from 12 to 17%.

One grain size distribution analysis was conducted on a sample from this cohesionless deposit, and the resulting grain size curve is presented in Figure 2. The analysis indicates 24% gravel, 48% sand, and 28% silt and clay size particles.

4.4 Bedrock


Bedrock was encountered and cored in Borehole 1005 from 8.2 to 11.0 m depths below existing ground surface. The recovered core samples show that the Precambrian bedrock consists of a massive, moderately closely to closely jointed gneiss with occasional micaceous layer. The percentage of core recovery varies from 78 to 100%. The Rock Quality Designation (R.Q.D.) values increase with depth from 42 to 76%. Based on these values and visual examination of the cores, the rock is considered to be of poor to good quality.


4.5 Groundwater Conditions

Groundwater conditions were observed in the open boreholes during the drilling and at the completion of each borehole. Observed groundwater levels in the open boreholes are within 1 m of the existing ground surface. It should, however, be pointed out that the groundwater at the site would fluctuate seasonally and in response to severe weather events.

5.0 CLOSURE

Sincerely,


Sydney Pang, Ph.D., P. Eng.


Eric Chung, M. Eng., P. Eng.
Designated MTO Contact.

.../...
M:\reports\1999\culvert.wpd




Andrew Drevininkas, P. Eng.



APPENDIX A

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_a	1	RATE OF SECONDARY CONSOLIDATION
c_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

P_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kn/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
P_w	kg/m ³	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kn/m ³	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kn/m ³	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m ³	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m ³ /s	RATE OF DISCHARGE
γ_d	kn/m ³	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kn/m ³	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kn/m ²	SEEPAGE FORCE
γ'	kn/m ³	UNIT WEIGHT OF SUBMERGED SOIL						

FIGURES

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

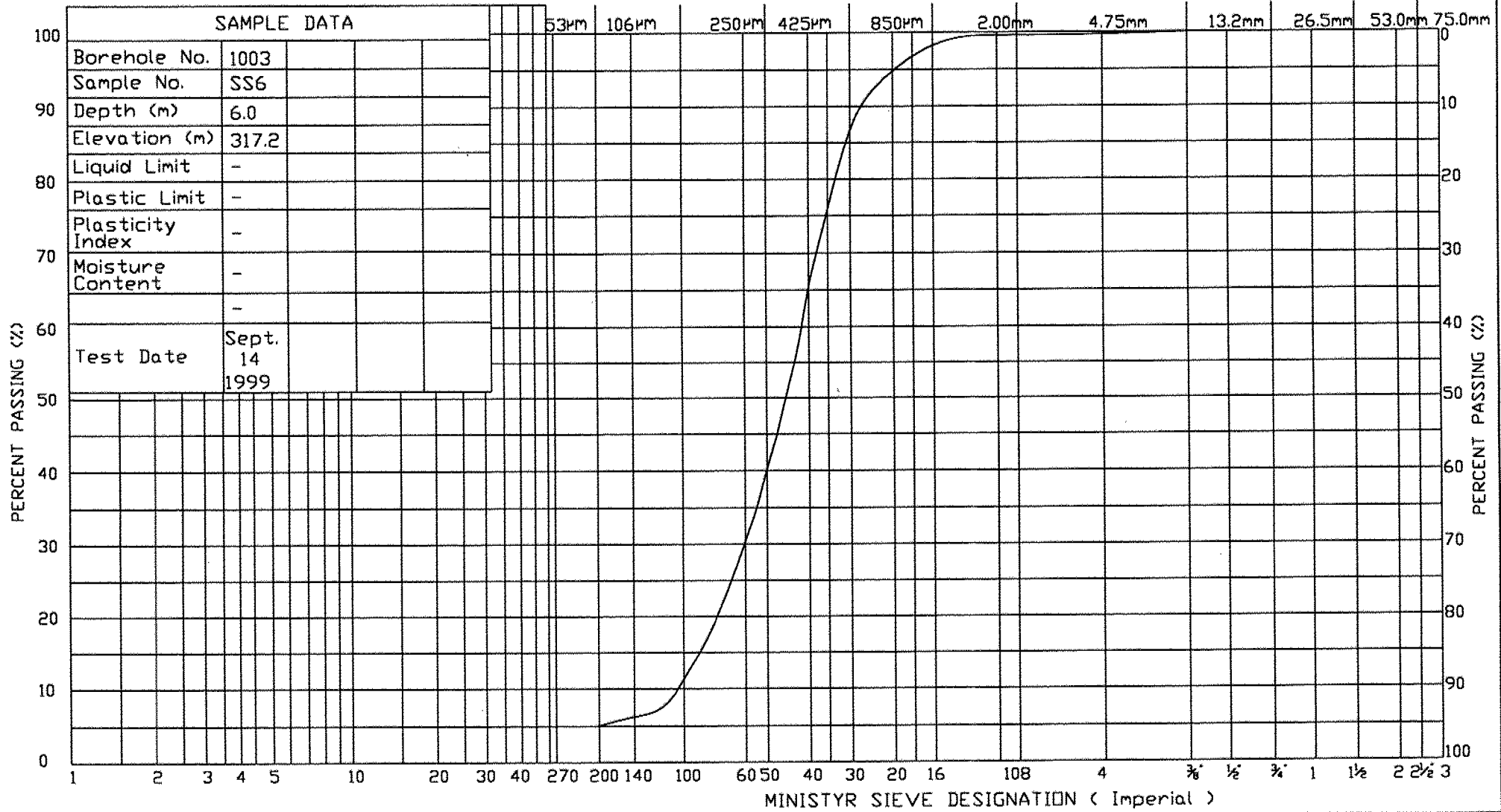
GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)

1 2 3 4 5 10 20 30 40 50 75µm 150µm 300µm 600µm 1.18mm 2.36mm 9.5mm 19.0mm 37.5mm 63.0mm

SAMPLE DATA

Borehole No. 1003
 Sample No. SS6
 Depth (m) 6.0
 Elevation (m) 317.2
 Liquid Limit -
 Plastic Limit -
 Plasticity Index -
 Moisture Content -
 Test Date Sept. 14 1999



AGRA

ENGINEERING GLOBAL SOLUTIONS

GRAIN SIZE DISTRIBUTION

SAND

1003: SS6

CLIENT:	DELCAN
JOB NO.:	TT98820 W P 466-93-00
PROJECT:	HWY 11, EMSDALE
LOCATION:	CULVERT @ STATION 19+984
DATE:	SEPTEMBER 15, 1999
FIGURE:	1

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

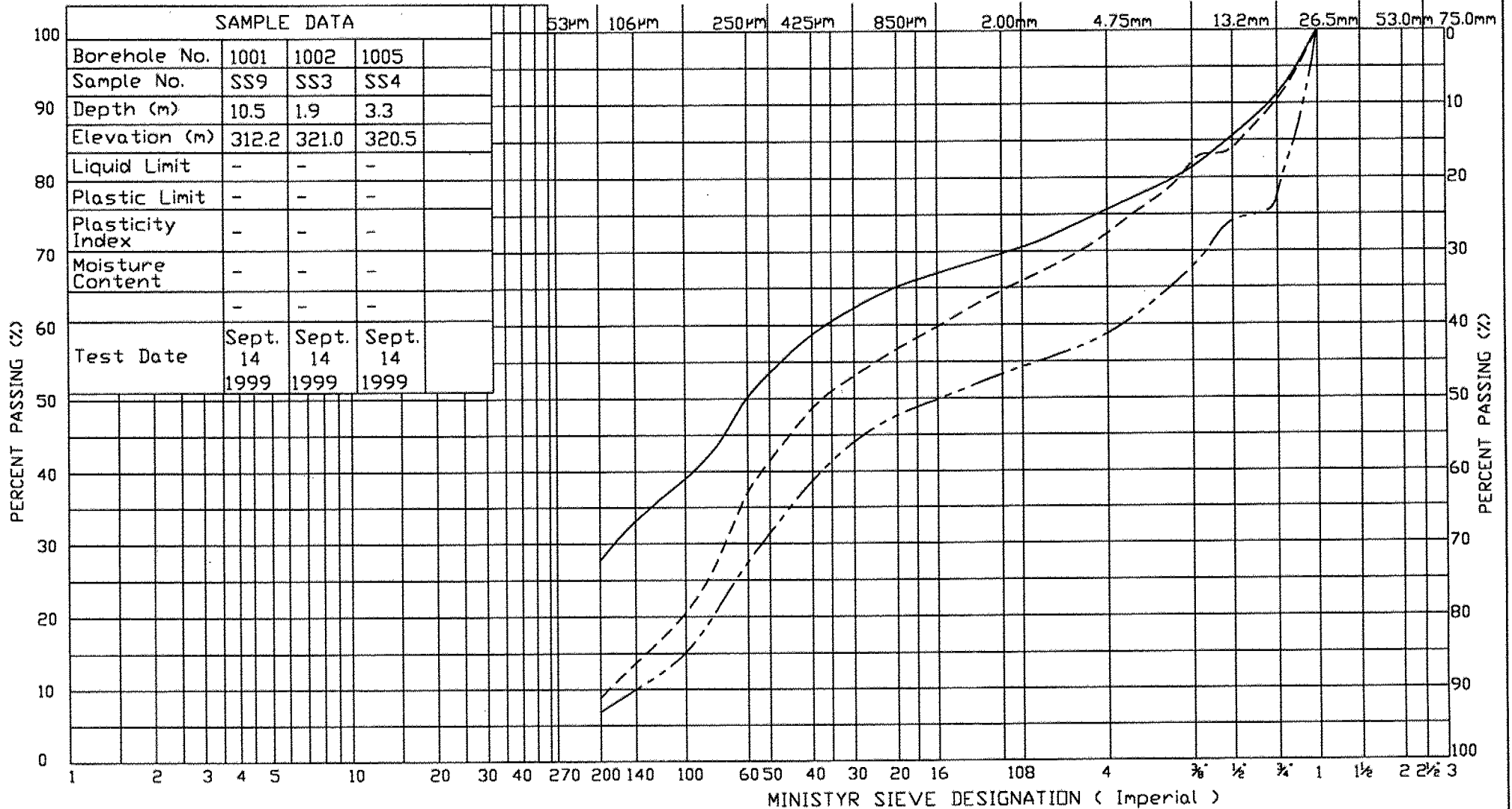
GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)

1 2 3 4 5 10 20 30 40 50 75µm 150µm 300µm 600µm 1.18mm 2.36mm 9.5mm 19.0mm 37.5mm 63.0mm

SAMPLE DATA

Borehole No.	1001	1002	1005
Sample No.	SS9	SS3	SS4
Depth (m)	10.5	1.9	3.3
Elevation (m)	312.2	321.0	320.5
Liquid Limit	-	-	-
Plastic Limit	-	-	-
Plasticity Index	-	-	-
Moisture Content	-	-	-
Test Date	Sept. 14 1999	Sept. 14 1999	Sept. 14 1999



GRAIN SIZE DISTRIBUTION

SAND and GRAVEL
to
GRAVELLY SAND

1001: SS9
1002: SS3
1005: SS4

CLIENT:	DELCAN
JOB NO.:	TT98820 W P 466-93-00
PROJECT:	HWY 11, EMSDALE
LOCATION:	CULVERT @ STATION 19+984
DATE:	SEPTEMBER 15, 1999
FIGURE:	2

ENCLOSURES

RECORD OF BOREHOLE No 1001

1 OF 2

METRIC

W.P. 466-93-00 LOCATION Site No.44-304 N 5042393.3 E319147.3 ORIGINATED BY MA
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering / Wash Boring / Casing COMPILED BY AD
DATUM Geodetic DATE 20 August 1999 CHECKED BY SP/EYC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)			
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE										
							20	40	60	80	100	10	20	30				
323.4 0.0	PEAT		1	SS	1		323									Station 19+945 59m Rt Med C/L		
322.6 0.8	light brown SAND some Gravel, some Silt occasional Cobbles, some oxidized stains compact wet		2	SS	15		322											
			3	SS	11		321											
								320										
			loose		4		SS	7	319									
									318									
			5	SS	13		317											
							316											
	grey brown		6	SS	22		315											
							314											
	trace Gravel loose		7	SS	9		313											
						312												
313.6 9.8	grey SAND and GRAVEL to GRAVELLY SAND trace to some Silt frequent Cobbles and Boulders dense to very dense wet		8	SS	11	311								Started using NW casing @ 9m depth. 24 48 (28) Coring commenced @ 11.5m depth				
							310											
					9	SS	41	309										
			10	SS	70													
			11	SS	70/18													

Continued Next Page

+ 3 . X 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 1001

2 OF 2

METRIC

W.P. 466-93-00 LOCATION Site No.44-304 N 5042393.3 E319147.3 ORIGINATED BY MA
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering / Wash Boring / Casing COMPILED BY AD
 DATUM Geodetic DATE 20 August 1999 CHECKED BY SP/EYC

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					
308.2													
15.3	END OF BOREHOLE WL in open borehole on completion: 0.8m					308							

RECORD OF BOREHOLE No 1002

1 OF 1

METRIC

W.P. 466-93-00 LOCATION Site No.44-304 N 5042382.2 E319131.9 ORIGINATED BY MA
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering COMPILED BY AD
 DATUM Geodetic DATE 19 August 1999 CHECKED BY SP/EYC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
323.3	0.15m TOPSOIL		1	SS	1		323							GR SA SI CL 40m Rt Med C/L	
0.0			2	SS	20		322								
	Sand and Gravel		3	SS	14		321								41 52 7 0
			4	SS	10		320								
	light brown SAND trace to some Gravel, some Silt compact wet		5	SS	16		319								
			6	SS	11		318								
	grey brown Gravelly Sand		7	SS	16		317								
			8	SS	5		316								
	fine Sand						315								
312.8	grey SAND and GRAVEL to GRAVELLY SAND trace to some Silt frequent Cobbles and Boulders very dense wet		9	SS	66		314								
10.5							313								
311.4	END OF BOREHOLE REFUSAL TO AUGER ADVANCE						312								
311.0	END OF DCPT REFUSAL TO CONE ADVANCE						311								
12.3	WL in open borehole on completion: 0.8m						310								

RECORD OF BOREHOLE No 1003

1 OF 1

METRIC

W.P. 466-93-00 LOCATION Site No 44-304 N 5042372.9 E319083.1 ORIGINATED BY MA
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering COMPILED BY AD
 DATUM Geodetic DATE 19 August 1999 CHECKED BY SP/EYC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE 20 40 60 80 100	PLASTIC LIMIT W _p NATURAL MOISTURE CONTENT W LIQUID LIMIT W _L WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
324.0	0.2m TOPSOIL		1	SS	4						Station 19+975 2m Lt Med C/L
0.0	brown SAND trace Silt, trace Gravel very loose to loose wet		2	SS	3						
			3	SS	6						
			4	SS	8						
			5	SS	9						
			6	SS	7						0 95 5 0
317.0	END OF BOREHOLE REFUSAL TO AUGER ADVANCE DCPT attempts at between 6.1m and 6.7m WL in open borehole on completion: 0.9m										
7.0											

RECORD OF BOREHOLE No 1004

1 OF 1

METRIC

W.P. 466-93-00 LOCATION Site No 44-304 N 5042362.5 E319049.7 ORIGINATED BY MA
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering COMPILED BY AD
 DATUM Geodetic DATE 24 August 1999 CHECKED BY SP/EYC

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					
324.4	0.15m TOPSOIL		1	AS			324										Station 19+990 33m Lt Med C/L
							323										
							322										
							321										
							320										Station 19+990 33m Lt Med C/L
							319										
318.4	END OF BOREHOLE REFUSAL TO AUGER ADVANCE																
6.0																	

RECORD OF BOREHOLE No 1005

1 OF 1

METRIC

W.P. 466-93-00 LOCATION Site No.44-304 N 5042361.5 E319020.3 ORIGINATED BY MA
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering / Wash Boring / Casing COMPILED BY AD
DATUM Geodetic DATE 23 August 1999 CHECKED BY SP/EYC

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							
								20 40 60 80 100							
								○ UNCONFINED + FIELD VANE							
								● QUICK TRIAXIAL x LAB VANE							
								20 40 60 80 100							
														</	

RECORD OF BOREHOLE No G1

1 OF 1

METRIC

W.P. 466-93-00 LOCATION Station 19+950 30m Rt Med C/L ORIGINATED BY MA
 DIST 52 HWY 11 BOREHOLE TYPE Solid Stem Augering COMPILED BY AD
 DATUM Geodetic DATE 20 January 1999 CHECKED BY SP/EYC

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			20	40						60	80
323.2 0.0	PEAT						323									
322.1 1.1							322									
	some Gravel						321									
							320									
							319									
							318									
							317									
	grey SAND trace Gravel wet						316									
							315									
							314									
	occasional cobbles						313									
							312									
							311									
310.4 12.8	END OF BOREHOLE REFUSAL TO AUGER ADVANCE															

RECORD OF BOREHOLE No SB5

1 OF 1

METRIC

W.P. 466-93-00 LOCATION Station 20+000 19m Lt Med C/L ORIGINATED BY MA
 DIST 52 HWY 11 BOREHOLE TYPE Solid Stem Augering COMPILED BY AD
 DATUM Geodetic DATE 20 January 1999 CHECKED BY SP/EYC

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		20	40	60	80	100	w_p	w	w_L		
323.5	0.4m TOPSOIL		1	SS	5											
						323										
			2	SS	14											
						322										
			3	SS	13											
						321										
			4	SS	10											
						320										
						319										
						318										
317.5	END OF BOREHOLE REFUSAL TO AUGER ADVANCE															
6.0																

OVERSIZE DRAWING(S)

**DRAFT
FOUNDATION INVESTIGATION AND DESIGN REPORT
FOR
PROPOSED CULVERT AT STATION 19+984 MEDIAN CENTRELINE
DISTRICT 52, HUNTSVILLE
W.P. 466-93-00**

Submitted To:

**Delcan Corporation
133 Wynford Drive
North York, Ontario, M3C 1K1
Canada**

Submitted By:

**AGRA
104 Crockford Blvd.
Scarborough, Ontario, M1R 3C6
Canada**

**September 1999
TT98820**

*Memo: Date Oct. 4/99
To: File
This draft report was received by
me with no cover. Consequently my
remarks are to file.
I have evaluated the consultants
performance in providing
Foundation Engineering Services
for this project and consider
that performance to be acceptable.
I have no further comments.*

September 29, 1999.
Ref. No.: TT98820

Delcan Corporation
133 Wynford Drive
North York, Ontario, M3C 1K1
Canada

Attention: Mr. Khaled El-Dalati, P. Eng.

Dear Sir:

**Re: DRAFT
FOUNDATION INVESTIGATION AND DESIGN REPORT
FOR
PROPOSED CULVERT AT STATION 19+984 MEDIAN CENTRELINE
DISTRICT 52, HUNTSVILLE
W.P. 466-93-00**

We take pleasure in enclosing four (4) copies of our Draft Geotechnical Investigation and Design Report carried out for the above mentioned project and we will be glad to discuss any questions arising from this work.

Soil and rock samples will be retained for a period of one year, and will thereafter be disposed of unless we are otherwise instructed.

We thank you for giving us this opportunity to be of service to you.

Sincerely,



George S.W. Chow, P. Eng.,
Designated MTO Contact.
GSWC/dee

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APPENDICES

APPENDIX A:	Limitations of Report
APPENDIX B:	Explanation of Terms Used in Report

FIGURES

GRAIN SIZE DISTRIBUTION CURVES	Figures 1 - 2
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ENCLOSURES

BOREHOLE LOCATIONS AND SOIL STRATA	DWG. NO. 1
RECORD OF BOREHOLE SHEETS	

1.0 INTRODUCTION

AGRA, Consulting Geotechnical Engineers, has been retained by Delcan Corporation (Delcan) to conduct a foundation investigation at the site of a proposed rigid frame concrete culvert to be used to realign the water flow of the existing P-3 Tributary. The culvert of 130 m in length will cross the proposed Highway 11 median centreline at about Station 19+984. The proposed works is part of the Highway 11 Four Laning Project, from Emsdale to Burk's Falls, W.P. 466-93-00, District 52, Huntsville, Ontario.

The purpose of this investigation is to obtain more detailed information about the subsurface conditions at the site of the proposed culvert by means of exploratory boreholes. Based on our interpretation of the data obtained from this and previous geotechnical investigations carried out in the vicinity, recommendations for the foundation design of the proposed culvert are provided. Comments are also provided on anticipated construction issues where they may affect the design of the proposed works, from a geotechnical point of view.

At the time of this investigation, the proposed, revised horizontal alignment of the culvert and the existing ground surface profile along the P-3 Tributary were provided to us on plan and profile by Delcan via facsimile transmission on August 10, 1999. The terms of reference for our scope of work are as outlined in our proposal letter, dated August 18, 1999.

2.0 SITE DESCRIPTION AND PHYSIOGRAPHY

The site is located some 900 m south of Star Lake Road at the existing P-3 Tributary crossing of the proposed Highway 11. The existing ground surface elevation at the proposed culvert location slopes gently in a easterly direction along the tributary, from about Elevations 324 m to 323 m. The surrounding area is generally moderately wooded with trees and brushes. The water in the creek is up to about 1 m deep. The proposed grade of Highway 11 above the culvert is at about Elevation 341 m for both the NBL and SBL. (15m P.I.)

Based on available geologic information, the site is in an area of ice-contact sediments. Generally after the last glacial withdrawal, ice-contact sediments (sands and gravels) followed by glaciofluvial sediments (ranging from deltaic and nearshore sands and gravels to prodeltaic and lake bottom silts and clays) were deposited on top of the existing sandy glacial till or Precambrian bedrock. The area was then inundated by glacial Lake Algonquin, depositing sands, silts and clays in low lying areas. The bedrock generally consists of strongly foliated gneissic to migmatic rocks of the Central Gneiss Belt, which is part of the Grenville Province (a structural subdivision of the Canadian Shield).

3.0 INVESTIGATION PROCEDURES

The field work for the investigation was carried out during the period of August 19, 20, 23 and 24, 1999, and consisted of drilling and sampling five boreholes (Borehole Nos. 1001 to 1005, inclusive) to depths of 6.0 to 15.3 m below the existing ground surface.

The plan locations of the boreholes along with a stratigraphic section parallel to the culvert alignment are shown on Drawing No. 1. Details of subsurface conditions encountered at each borehole location, including the results of in-situ testing, are presented on the Record of Borehole sheets.

The boreholes were advanced, using a combination of hollow stem continuous flight augers, casings, wash boring and coring equipment, with a track-mounted power auger drill rig (BOA 6M2) owned and operated by Groundworks Drilling Inc., under the full-time supervision of a Geotechnical Engineer from AGRA.

Sampling in the boreholes were carried out at regular intervals of depth by the Standard Penetration Test Method (SPT), as specified in ASTM Method D 1586. This consists of freely dropping a 63.5 kg hammer for a vertical distance of 0.76 m to drive a 51 mm diameter outside diameter split barrel (split-spoon) sampler into the ground. The number of blows of the hammer to drive the sampler into the relatively undisturbed ground for a vertical distance of 0.30 m is recorded as the Standard Penetration Resistance or the 'N'-value of the soil, and this gives an indication of the consistency or the compactness condition of the soil deposit.

In order to advance the boreholes through cobbles and boulders and to prove bedrock, rotary core drilling was carried out in Boreholes 1001 and 1005 utilizing NW size casings and cores were retrieved using a NXL size core barrel.

The borehole locations were established in the field by our engineering staff, in relation to the proposed centreline of Highway 11 already staked out by Dearden and Stanton Ltd (retained by Delcan). Due to restrictions by the topography and the vegetation, all five boreholes were positioned along the south bank of the tributary. The as-drilled borehole elevations were not surveyed with reference to Geodetic datum. Based on existing topographic information and visual observation on site, the vicinity along the tributary is flat-lying and therefore borehole elevations are estimated based on the ground surface profile of the tributary available to us. We understand that survey results of the as-drilled boreholes are to be provided to us by Delcan.

The soil samples were transported to our geotechnical laboratory in Toronto (Scarborough) for further examination and classification. A laboratory testing programme, consisting of natural moisture content determinations and grain size analyses, was performed on selected representative soil samples. The results of the laboratory tests are presented on the appropriate Record of Borehole Sheets and also on Figure Nos. 1 and 2, inclusive.

The boreholes were left open until the end of each work day to enable us to take additional water level readings. The boreholes were adequately grouted on completion.

4.0 SUBSURFACE CONDITIONS

The subsurface conditions were explored at five boreholes (Borehole Nos. 1001 to 1005) during the current investigation. Boreholes SB5 and G1 from previous investigations were utilized. The plan locations of the boreholes along with the stratigraphic section along the culvert alignment are shown on Drawing No. 1. Details of subsurface conditions encountered at each borehole location, including the results of in-situ testing, groundwater observations and laboratory test results are presented on the Record of Borehole sheets. The subsurface conditions are summarized in the following.

In general, the subsurface stratigraphy comprises surficial peat and/or topsoil overlying loose to compact sand, which is in turn underlain by dense to very dense sand and gravel to gravelly sand with frequent cobbles and boulders. The depth to the sand and gravel remains relatively constant from the west culvert limit to about the east crest of the SBL, but increases towards the east. The groundwater level is within 1 m depth of the existing ground surface.

4.1 PEAT AND TOPSOIL

Peat of 0.8 m to 1.0 m in thickness was encountered at ground surface in Boreholes 1001 and G1.

Topsoil was encountered in Boreholes 1002 to 1005 and SB5, ranging in thickness from 0.15 m to 0.4 m.

4.2 SAND

Below the surficial topsoil or peat, a cohesionless sand deposit with trace to some gravel was encountered to depths of about 9.8 m to 12.8 m in Boreholes 1001, 1002 and G1, and to depths of 4.9 m to 7.0 m in Boreholes 1003 to 1005, and SB5. Occasional sand and gravel to gravelly sand interlayers were present within this deposit.

One grain size analysis was conducted on a sample of each of the sand, sand and gravel, gravelly sand. The grain size curves are presented on Figures 1 and 2. For the sand, the results indicate 0% gravel, 95% sand, 5% silt and 0% clay size particles.

Most measured 'N'-values within the sand in Boreholes 1001, 1002, 1005 and SB5 range from 10 to 21 blows per 0.3 m, indicating a typically compact condition; occasional loose zones are present with 'N' values less than 10 blows. The sand is loose to very loose throughout Borehole 1003. In Borehole 1005, a high 'N'-value of 80 was measured at 1 m depth and may be attributed to probable cobbles. Measured moisture contents range from about 12 to 28%.

For the sand and gravel to gravelly sand interlayers, the results indicate 28 to 41% gravel, 52 to 64% sand, 7 to 8% silt and 0% clay. It is noted that the cobbles and boulders could not be sampled with the spoon sampler.

4.3 SAND AND GRAVEL TO GRAVELLY SAND

A layer of sand and gravel to gravelly sand underlies the upper sand in Boreholes 1001, 1002 and 1005. Frequent cobbles and/or boulders were inferred or encountered within this layer. This layer extends to the full depth of Boreholes 1001, 1002 and is about 3.3 m thick in 1005. Auger refusal was encountered below the upper sand in Boreholes 1003, 1004, G1 and SB5 at levels which may be inferred as the upper surface of the cobbles and/or boulders. Measured 'N'-values range from 21 to greater than 50 blows per 0.3 m, indicating a compact to very dense, but typically dense to very dense condition. Measured moisture contents range from 12 to 17%.

One grain size distribution analysis was conducted on a sample from this cohesionless deposit, and the resulting grain size curve is presented in Figure 2. The analysis indicates 24% gravel, 48% sand, and 28% silt and clay size particles.

4.4 BEDROCK

Bedrock was encountered and cored in Borehole 1005 from 8.2 to 11.0 m depths below existing ground surface. The recovered core samples show that the Precambrian bedrock consists of a massive, moderately closely to closely jointed gneiss with occasional micaceous layer. The percentage of core recovery varies from 78 to 100%. The Rock Quality Designation (R.Q.D.) values increase with depth from 42 to 76%. Based on these values and visual examination of the cores, the rock is considered to be of poor to good quality.

4.5 GROUNDWATER CONDITIONS

Groundwater conditions were observed in the open boreholes during the drilling and at the completion of each borehole. Observed groundwater levels in the open boreholes are within 1 m of the existing ground surface. It should, however, be pointed out that the groundwater at the site would fluctuate seasonally and in response to severe weather events.

5.0 DISCUSSION AND RECOMMENDATIONS

The original design, as depicted on the July 21, 1999 plan, consists of a 3,000 mm by 1,500 mm rigid frame concrete box culvert of about 162 m in length. In order to reduce the length of the culvert, we understand that the design has been revised and consists of a rigid frame concrete open footing culvert of the same cross-sectional dimensions, but with an overall length of about 130 m. A rock fill embankment of up to 18 m in height with side slopes of 1.25H:1V and 2 m wide berms are to be used instead of an earth fill embankment. The new design is shown on a plan dated August 6, 1999, which is further updated on August 16, 1999. Subsequent telephone conversations with Delcan indicate that a rigid box culvert is now preferred. In addition, the culvert will be separated into two structural units in the vicinity of the median centreline, and joints will be introduced at regular intervals in an attempt to mitigate the adverse effects of differential settlement.

// This should be a recommendation

As explained in our proposal letter of August 18, 1999, the information obtained by geotechnical probes put down prior to this investigation indicated that the subsoils generally consist of water-bearing sands of loose to compact condition. Refusal to augering was encountered at about 6 m to 12 m depths due to possible cobbles, boulders and/or bedrock. The five boreholes drilled and sampled during this investigation provide additional subsurface information along the proposed culvert alignment. Auger refusal was encountered on the surface of a layer of cobbles and boulders, proven in three boreholes (near the limits of the proposed culvert) and inferred at the remaining locations. Along the culvert alignment, depth to the cobbles/boulders remains relatively constant at about 6 m below existing ground surface between the west limit and the east crest of the southbound lane (SBL), before increasing to 12 m to 13 m depths near the east limit of the culvert. Bedrock was proven at about 8 m depth at Borehole 1005, underlying the cobbles/boulders, near the west limit of the culvert. The groundwater level along the culvert alignment was observed at within 1 m below the existing ground surface.

The existing ground surface along the alignment of the proposed culvert slopes down gently from west to east. The proposed culvert is to be designed for a normal water level of Elevation 324.10 m and a flood level of Elevation 324.88 m.

5.1 CULVERT FOUNDATIONS ON IMPROVED SUBGRADE

The culvert extensions are expected to be founded at about Elevations 322 m and 321.5 m at the west and east limits, respectively. The boreholes located to the east of the northbound lane (NBL) centreline encountered peat and organics at between 0.8 m and 1.1 m depths. In order to avoid excessive settlement, to provide a more uniform founding subgrade condition and to improve the load carrying capacity of the upper zones of the founding soils, we recommend that the peat, organics, the underlying sand and otherwise weak or unsuitable zones be removed to a depth of 2 m below the founding depth and replaced with compacted granular fill. At other locations along the culvert alignment, the required depth of sub-excavation and replacement with fill should be maintained at about 2 m below the founding depth, where appropriate.

Assuming an open cut excavation, the plan limits of the excavation base should be at least 2 m beyond the perimeter of the culvert base. The excavations will extend below the groundwater table and therefore groundwater control will be required. Provided adequate groundwater control measures are implemented, temporary excavation side slopes should be stable at an inclination of 2H:1V. Care must be exercised during excavation to avoid disturbing the founding subgrade.

When the excavation reaches the required depth, the subgrade should be inspected and approved by the Geotechnical Engineer. If necessary, the excavation may need to be deepened to a depth below any peat or organic layers. After its approval, the exposed subgrade at the base of the excavation may need to be compacted, if requested by the Geotechnical Engineer, to achieve a density of not less than about 95% of the material's Standard Proctor Maximum Dry Density (SPMDD). The fill used to raise the grade inside the excavation should be Granular 'A' material placed when its moisture content is within $\pm 2\%$ of its optimum moisture content. It should be placed in loose lifts not exceeding 200 mm in thickness and should be uniformly compacted to not less than 100% of its SPMDD.

A factored geotechnical resistance at U.L.S. of 500 kPa and a geotechnical resistance at S.L.S. equal to 350 kPa can be assigned to the founding Granular 'A' subgrade prepared in this manner. For the west portion of the culvert (under the SBL embankment), the serviceability condition corresponds to total and differential settlements (between the two limits) in the order of 50 mm and 20 mm, respectively. For the east portion of the culvert (under the NBL embankment), the serviceability condition corresponds to total and differential settlements (between the two limits) in the order of 70 mm and 30 mm, respectively.

The culvert should be designed to resist frost forces, weight of embankment fill and traffic loadings. It is noted that compression of the native sand subgrade induced by up to 18 m of embankment fill (Elevations 341 to 323 m) will be in the order of 75 mm at the west end of the culvert, and 150 mm at the east end of the culvert. This global settlement as well as the differential settlement from west (SBL) to east (NBL) should be considered in designing the culvert.

The unfactored horizontal resistance against sliding between concrete and Granular 'A' type material can be calculated using a friction angle of 32 degrees.

35°

Along the culvert alignment, some form of groundwater control will be required during construction as the proposed bottom of excavation is below the water table. Dewatering will need to be capable of drawing the water level to no less than 0.6 m below the bottom of the excavation. If the water table is not properly lowered, the granular soil at the bottom of the excavation can lose its load carrying capacity (in addition to the instability of the side slopes). If this happens and the engineered fill is placed on disturbed and loosened subgrade, excessive settlements can occur after construction of the culvert extension. We therefore recommend that the contractor investigate the position of the water table before starting the excavation to assess required dewatering. The proposed groundwater control scheme should be reviewed by the Geotechnical Engineer familiar with the findings of this report and appointed by the Contract Administrator. As discussed before, the water levels at the time of our investigation were generally recorded at within 1 m depth below

existing grade. Pumping from properly filtered sumps and wells could be appropriate methods of controlling the groundwater.

5.2 CULVERT FOUNDATIONS ON PILES

If the above settlement tolerances are unacceptable, consideration could be given to supporting the culvert on deep foundations in the form of steel H-piles, driven to practical refusal within the upper portion of the very dense sand and gravel with frequent cobbles and boulders. In order to adequately penetrate the sand and gravel as well as occasional cobbles at shallower depths, a heavier section such as HP 310x79 with reinforced tips would be suitable for use.

It is considered likely that the driven H-piles will not be able to penetrate deep into the very dense sand and gravel with frequent cobbles and boulders. Based on the results of the boreholes, the following Table 1 summarizes the estimated average pile tip elevations that may be assumed for design purposes.

TABLE 1

SUPPORT LOCATION	REFERENCE BOREHOLE	ESTIMATED APPROXIMATE PILE TIP ELEVATION (depth) (m)
Station 19+945 60m Rt Med. C/L	1001	309 (14)
Station 19+945 38m Rt Med. C/L	1002	311 (12)
Station 19+975 Med. C/L	1003	314 (9)
Station 19+990 32m Lt Med. C/L	1004	315 (8)
Station 20+010 60m Lt Med. C/L	1005	315 (8)

The borehole results indicate that the founding sand and gravel deposit containing cobbles and/or boulders was encountered at higher elevations in the vicinity of the SBL compared with those in the vicinity of the NBL. Therefore, it may be expected that the piles will terminate at higher elevations within the SBL. In the vicinity of the west limit of the culvert, however, the driven piles may penetrate through the sand and gravel into the bedrock.

5.2.1 Resistance to Axial Loads

For HP 310x79 steel H-piles driven to practical refusal within the very dense sand at or below the elevations shown in Table 1 above, the following axial resistances may be assumed for design.

Factored Axial Resistance at Ultimate Limit States (U.L.S.) = 1,150 kN
Geotechnical Resistance at Serviceability Limit States (S.L.S.) = 1,000 kN

too low!

The piles should be driven with a suitably heavy hammer capable of delivering a rated capacity of between 40 and 50 kJ per blow. The driving of the piles should be controlled by a recognized pile driving formula, such as the Hiley Formula. The estimated ultimate resistance of the piles driven to practical refusal within the very dense sand at about the elevations quoted above, as given by the Hiley Formula, is approximately 2,300 kN. This value was arrived at by dividing the factored axial resistance at U.L.S. by a resistance factor of 0.5, as per current MTO practice.

Cobbles and/or boulders were inferred or encountered within the very dense sand and gravel in the boreholes drilled at the abutment locations. In view of this and the hard driving conditions anticipated, the pile tips should be reinforced, as per MTO Standards (OPSD 3301.00) to minimize damage to the piles.

Oversize materials (e.g. greater than 75 mm nominal diameter) should not be used in the fills through which piles would be driven.

In accordance with MTO standard practice, the piles should be driven to about 2 to 3 m above the design elevations given in Table 1 and the driving should then be monitored and controlled by the Hiley Formula. If the driven pile encounters refusal above the recommended elevation, the Geotechnical Engineer appointed by the Contractor Administrator should be notified immediately.

During the driving process, piles which have already been driven should be monitored to determine if they are heaving due to effects of driving adjacent piles. If this phenomenon occurs, the affected piles should be re-driven. It is recommended that not less than 15% of the piles be re-struck one to two days after initial installation, as a precaution against relaxation. If relaxation occurs, then all piles in that foundation element should be re-tapped.

It is possible that some of the piles may penetrate to one to two metres below the estimated tip elevations and this aspect should be taken into consideration when ordering piles.

The geotechnical resistance at Serviceability Limit States (S.L.S.) corresponds to settlement of the piles. Provided that the piles are designed and installed as recommended above, it is considered that the quoted S.L.S. value corresponds to no more than 25 mm of settlement.

5.3 BACKFILLING

Backfill arrangements around the culvert should be carried out as per OPSD 803.02. Backfill to the culvert should consist of free-draining, non-frost susceptible granular materials such as OPSS Granular 'A' or 'B'. The excavated material is not suitable for backfilling purposes due to the high (wetter than optimum) natural moisture contents. All granular fill should be placed in loose lifts not exceeding 200 mm thick and be compacted to at least 95% of its SPMDD.

Heavy compaction equipment should not be used adjacent to the walls and roof of the culvert. The height of the backfill to the culvert walls should be maintained equal on both sides of the structure during all stages of backfill placement.

Computation of earth pressures acting against rigid culvert walls should be in accordance with the Ontario Highway Bridge Design Code, 3rd Edition (1991). For design purposes, the following properties can be assumed for backfill:

Compacted Granular 'A'

Angle of Internal Friction $\phi = 35^\circ$ (unfactored)

Unit Weight = 22kN/m³

Coefficient of Lateral Earth Pressures:

Level Backfill	Backfill Sloping at 3H:1V	Backfill Sloping at 2H:1V
$K_a = 0.27$	$K_a = 0.34$	$K_a = 0.40$
$K_b = 0.35$	$K_b = 0.44$	$K_b = 0.50$
$K_o = 0.43$	$K_o = 0.56$	$K_o = 0.62$
$K^* = 0.45$	$K^* = 0.60$	$K^* = 0.66$

Compacted Granular 'B'

Angle of Internal Friction $\phi = 30^\circ$ (unfactored)

Unit Weight = 21 kN/m³

Coefficient of Lateral Earth Pressures:

Level Backfill	Backfill Sloping at 3H:1V	Backfill Sloping at 2H:1V
$K_a = 0.33$	$K_a = 0.42$	$K_a = 0.54$
$K_b = 0.41$	$K_b = 0.52$	$K_b = 0.64$
$K_o = 0.50$	$K_o = 0.66$	$K_o = 0.76$
$K^* = 0.57$	$K^* = 0.74$	$K^* = 0.86$

NOTE: K_a is the coefficient of active earth pressure

K_b is the backfill earth pressure coefficient for an unrestrained structure including compaction effects

K_o is the coefficient of earth pressure at rest

K^* is the earth pressure coefficient for a soil loading a fully restrained structure and includes compaction effects

The earth pressure coefficient adopted will depend on whether the structure is restrained or movements can be allowed such that the active state of earth pressure can develop. If the culvert walls are restrained (such as a rigid box structure) and does not allow lateral yielding, then at rest pressures should be used as per Clause C6-7.1 of the O.H.B.D.C., 3rd Edition.

The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients in accordance with Clause 6-7.4.3 of the O.H.B.D.C., 3rd Edition. Alternatively a compaction surcharge of 16 kPa can be included in the lateral earth pressure for the structural design of the culvert.

Vibratory equipment for use behind culvert walls and retaining walls should be restricted in size as per current MTO practice.

5.4 EMBANKMENT STABILITY

*Cumbar
- construction joint*

Provided that all recommended procedures regarding rock fill embankment design and construction are followed as per AGRA Pavement Design Report for W.P. 466-93-00, Highway 11 Four Laning, from 2.5 km South of Highway 518E, Northerly 7.3 km at Emsdale, the 18 m high embankment with slope inclination of 1.25 horizontal to 1 vertical, with appropriate benching, should be stable.

Provided that all the peat, surficial organic and otherwise unsuitable materials are removed before placing the embankment fill, and the subgrade is properly compacted from the surface as discussed above, the total settlement of the foundation material would be in the order of 75 mm to 150 mm (west to east) and should be substantially completed within two months of placing the embankment fill to its full height. This settlement will, however, occur concurrently as the rock fill is placed in lifts.

5.5 CONSTRUCTION

dewatering MSP

Excavations should be carried out in accordance with the Safety Regulations of the Province (i.e. Occupational Health and Safety Act O.Reg. 213/91). The boreholes show that the excavations can be expected to extend through peat, and loose sands below the groundwater table. Provided the water table is properly lowered, open cut excavations can be expected to stand temporarily at 2H:1V side slopes.

We recommend that any water flow in the existing water course be diverted away from the culvert excavation to enable the culvert construction and fill placement to be carried out in the dry. Major problems due to groundwater seepage are not anticipated, provided dewatering is carried out properly. Pumping from properly filtered sumps will be required to augment the dewatering system.

In order to avoid unbalanced loading on the culvert, the height of the rock fill around the culvert should be maintained equal on both sides throughout construction as much as practically possible.

Allowance should be made to place an approximately 150 mm thick layer of lean concrete on the subgrade surface, i.e. excavation base, within four hours of preparation and acceptance of the bearing soil. It should be pointed out that if the foundation soil is disturbed, excessive settlements can occur after structural loads are applied.

5.6 FROST PROTECTION

The culvert design should ensure that a soil cover of 1.8 m or its thermal equivalent is required for frost protection of foundations.

5.7 EROSION PROTECTION

Erosion protection should be provided at the culvert inlet (including the slopes and sides) and outlet, especially if head and wing walls are not incorporated. We recommend the use of cutoff walls or seepage seals around the proposed culvert to prevent erosion of the enclosing soil. Where clay seal will be used at the inlet, this should be at least 0.6 m thick. A rip-rap (rock) filter blanket at the outlet will also be required.

Rip-rap should also be provided at both upstream and downstream of the culvert. The stones should be adequately sized to prevent erosion of the stream bed. The rip-rap layer should cover all areas of the embankment slope with which flood flows are likely to be in contact. A granular filter layer may also be required beneath the rip-rap, depending on the sizes of the rip-rap.

6.0 CLOSURE

We recommend that once the details of the structures are finalized, our recommendations should be reviewed for their specific applicability.

The Limitations of Report, as quoted in Appendix A, is an integral part of this report.

Sincerely,

Sydney Pang

Sydney Pang, P. Eng.

George S.W. Chow
George S.W. Chow, P. Eng.,
Designated MTO Contact.
SP/dee

Eric Chung
Eric Chung, P. Eng.

APPENDIX A

AGRA

LIMITATIONS OF REPORT

The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environmental aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. It is recommended practice that the Geotechnical Engineer be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in testholes. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report. Since all details of the design may not be known, we recommend that we be retained during the final design stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices. No other warranty is expressed or implied.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. AGRA accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

APPENDIX B

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_f	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kn/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m ³	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kn/m ³	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kn/m ³	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m ³	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m ³ /s	RATE OF DISCHARGE
γ_d	kn/m ³	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kn/m ³	UNIT WEIGHT OF SATURATED SOIL	I_c	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kn/m ³	SEEPAGE FORCE
γ'	kn/m ³	UNIT WEIGHT OF SUBMERGED SOIL						

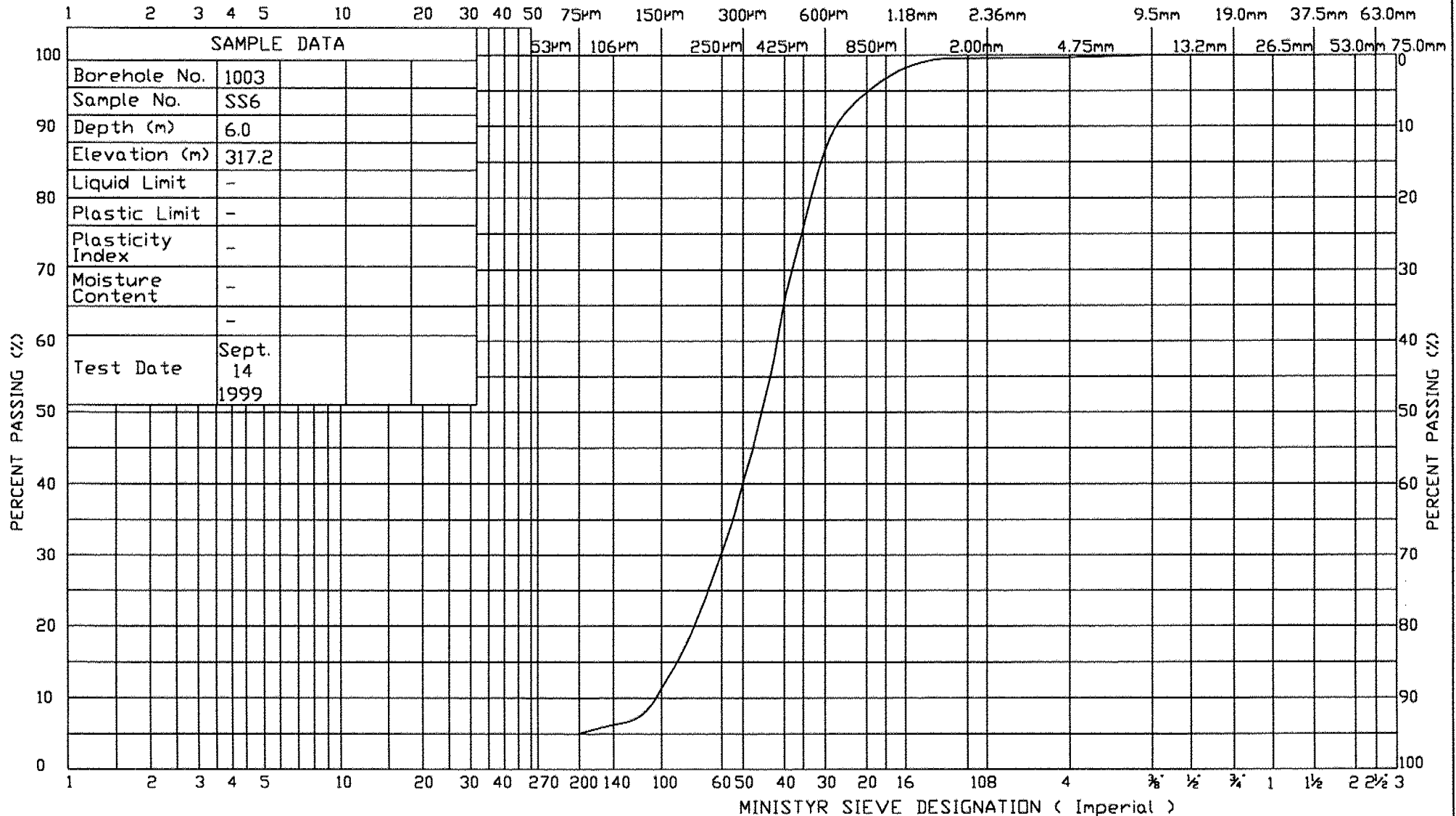
FIGURES

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



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ENGINEERING GLOBAL SOLUTIONS

GRAIN SIZE DISTRIBUTION

SAND

1003: SS6

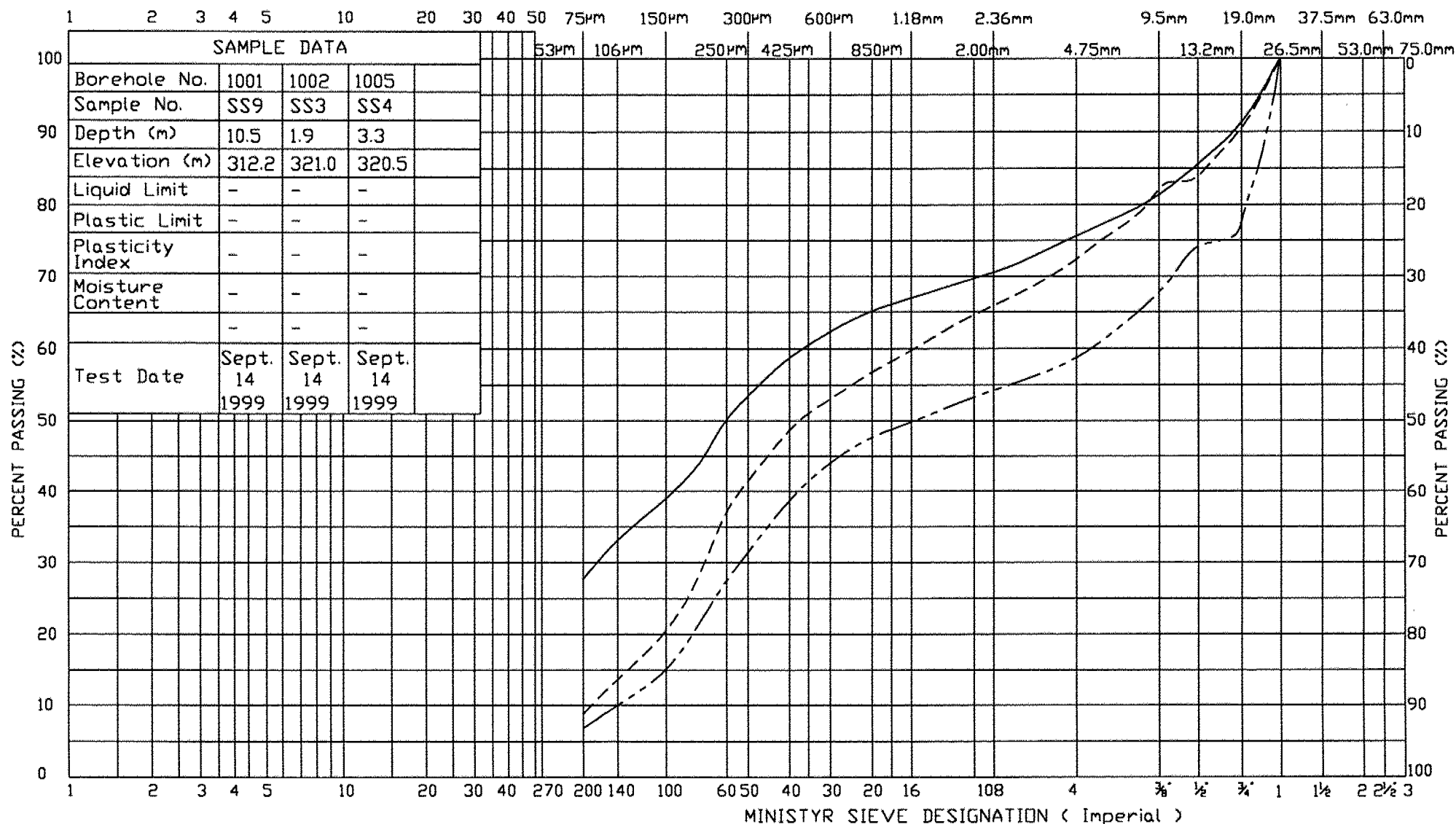
CLIENT:	DELCAN	
JOB NO.:	TT98820	W P 466-93-00
PROJECT:	HWY 11, EMSDALE	
LOCATION:	CULVERT @ STATION 19+984	
DATE:	SEPTEMBER 15, 1999	
	FIGURE: 1	

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



AGRA

ENGINEERING GLOBAL SOLUTIONS

GRAIN SIZE DISTRIBUTION

SAND and GRAVEL
to
GRAVELLY SAND

1001: SS9

1002: SS3

1005: SS4

CLIENT:

DELCAN

JOB NO.:

TT98820

W P 466-93-00

PROJECT:

HWY 11, EMSDALE

LOCATION:

CULVERT @ STATION 19+984

DATE:

SEPTEMBER 15, 1999

FIGURE: 2

ENCLOSURES

DRAFT

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES - METRES.

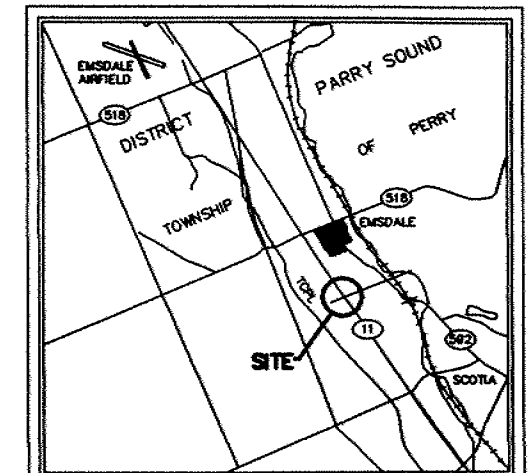
CONT. No.
W.P. No. 473-93-00



**PROPOSED CULVERT AT ST.19+984
& MED. C/L HWY 11
BORE HOLE LOCATIONS & SOIL STRATA**

SHEET

AGRA Earth & Environmental Ltd.



KEY PLAN

1 km 0 1 km 2 km 3 km

LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊙ Bore Hole & Cone
- 'N' Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60' Cone, 475 J/blow)
- WL at time of investigation - Aug. 1999
- WL in Piezometer
- Piezometer

No	ELEVATION	CO-ORDINATES STATION	OFFSET
1002	-	19+945	38 Rt Med C/L
1001	-	19+945	60 Rt Med C/L
G1	-	19+950	30 Rt Med C/L
1003	-	19+975	Med C/L
1004	-	19+990	32 Lt Med C/L
SB5	-	20+000	19 Lt Med C/L
1005	-	20+010	60 Lt Med C/L

-NOTE-

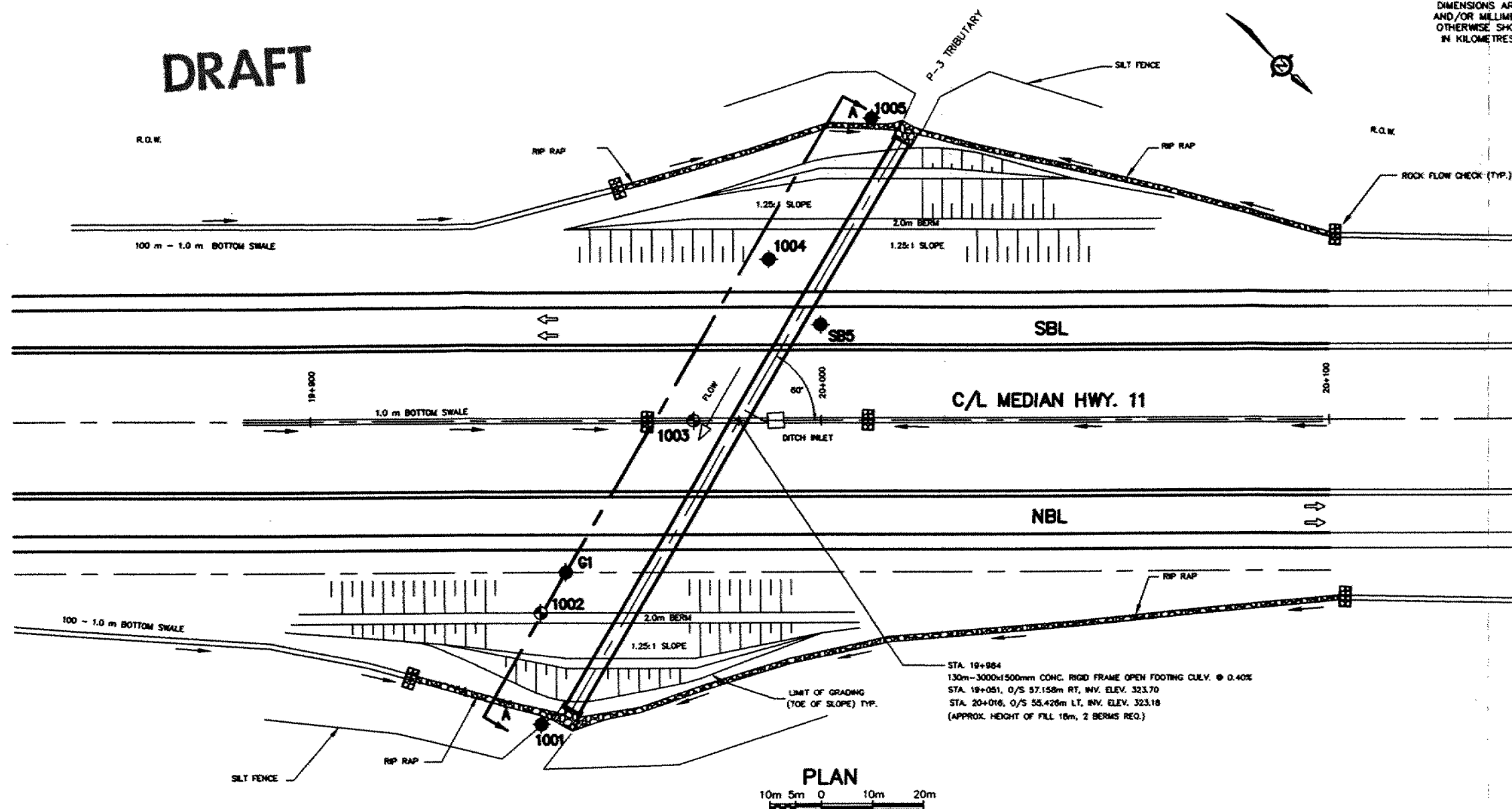
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen. Cond.

DATE	BY	DESCRIPTION
------	----	-------------

HWY No 11	SUBM'D SP	CHECKED SP	DATE Sept. 1999	SITE
	DRAWN MA	CHECKED		DWG 1

REF. Hwy 11 Site Plan
Dwg. by MTO; Aug. 16, 1999

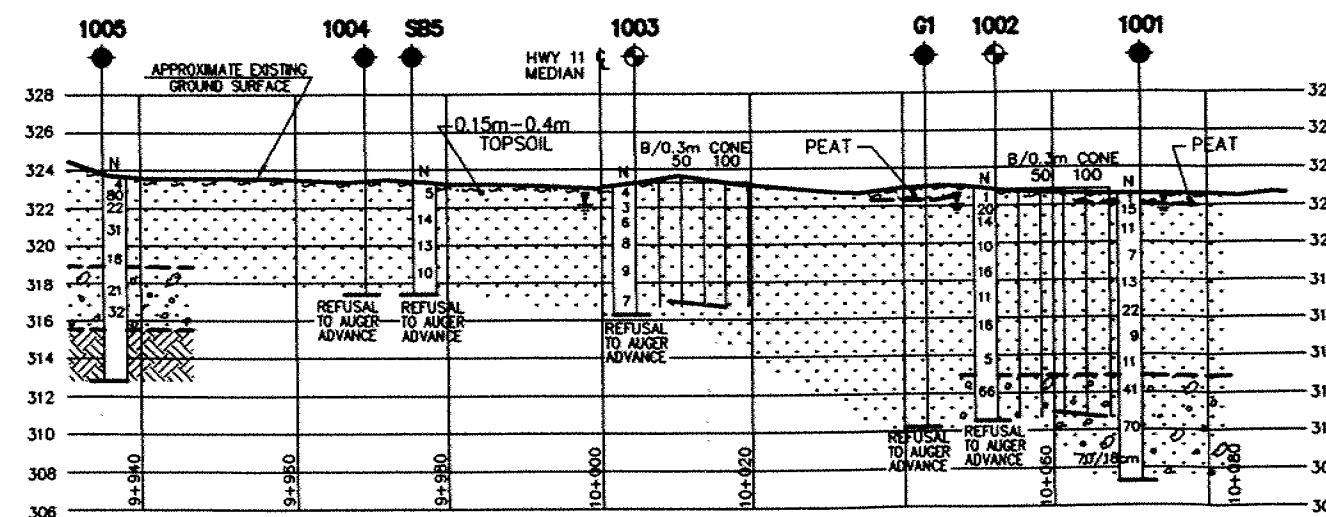


PLAN

10m 5m 0 10m 20m

SOIL STRATIGRAPHY LEGEND

- SAND
SOME GRAVEL
TRACE SILT
Loose to Compact
- SAND & GRAVEL TO GRAVELLY SAND
WITH FREQUENT
COBBLES & BOULDERS
Compact to Very Dense
- GNEISS
BEDROCK



SECTION A-A

10m 5m 0 10m 20m HOR
4m 2m 0 4m 8m VER

RECORD OF BOREHOLE No 1001

1 OF 2

METRIC

W.P. 466-93-00 LOCATION Station 19+945 60m Rt Med C/L ORIGINATED BY MA
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering / Wash Boring / Casing COMPILED BY AD
 DATUM Assumed DATE 20 August 1999 CHECKED BY SP

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	20	40
322.7 0.0	PEAT		1	SS	1														
321.9 0.8	light brown SAND some Gravel, some Silt occasional Cobbles, some oxidized stains compact wet		2	SS	15														
			3	SS	11														
			4	SS	7														
			5	SS	13														
			6	SS	22														
			7	SS	9														
			8	SS	11														
312.9 9.8	grey SAND and GRAVEL to GRAVELLY SAND trace to some Silt frequent Cobbles and Boulders dense to very dense wet		9	SS	41														
			10	SS	70														
			11	SS	70/18														

Continued Next Page

+ 3 x 3 Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 1001

2 OF 2

METRIC

W.P. 466-93-00 LOCATION Station 19+945 60m Rt Med C/L ORIGINATED BY MA
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering / Wash Boring / Casing COMPILED BY AD
 DATUM Assumed DATE 20 August 1999 CHECKED BY SP

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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE 20 40 60 80 100					WATER CONTENT (%)					
											10	20	30			
307.5																
15.3	END OF BOREHOLE WL in open borehole on completion: 0.8m					307										

RECORD OF BOREHOLE No 1002

1 OF 1

METRIC

W.P. 466-93-00 LOCATION Station 19+945 38m Rt Med C/L ORIGINATED BY MA
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering COMPILED BY AD
 DATUM Assumed DATE 19 August 1999 CHECKED BY SP

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								
322.9	0.15m TOPSOIL		1	SS	1								
0.0			2	SS	20								
	Sand and Gravel		3	SS	14								41 52 7 0
			4	SS	10								
	light brown SAND trace to some Gravel, some Silt compact wet		5	SS	16								
			6	SS	11								
	grey brown Gravelly Sand		7	SS	16								
			8	SS	5								
	fine Sand		9	SS	66								
312.4	gray SAND and GRAVEL to GRAVELLY SAND trace to some Silt frequent Cobbles and Boulders very dense wet												
10.5													
311.0	END OF BOREHOLE												
11.9	REFUSAL TO AUGER ADVANCE												
310.6	END OF DCPT												
12.3	REFUSAL TO CONE ADVANCE												
	WL in open borehole on completion: 0.8m												

RECORD OF BOREHOLE No 1003

1 OF 1

METRIC

W.P. 466-93-00 LOCATION Station 19+975 Med C/L ORIGINATED BY MA
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering COMPILED BY AD
 DATUM Assumed DATE 19 August 1999 CHECKED BY SP

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								
323.2	0.2m TOPSOIL		1	SS	4		323						
			2	SS	3		322						
			3	SS	6		321						
			4	SS	8		320						
			5	SS	9		319						
			6	SS	7		318						
							317						
316.2	END OF BOREHOLE REFUSAL TO AUGER ADVANCE DCPT attempts at between 6.1m and 6.7m WL in open borehole on completion: 0.9m												

RECORD OF BOREHOLE No 1004

1 OF 1

METRIC

W.P. 466-93-00 LOCATION Station 19+990 32m Lt Med C/L ORIGINATED BY MA
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering COMPILED BY AD
DATUM Assumed DATE 24 August 1999 CHECKED BY SP

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			20	40						60	80
323.5 0.0	0.15m TOPSOIL		1	AS												
	brown SAND some Gravel trace Silt occasional Cobbles wet		2	AS												
			3	AS												
317.5 6.0	END OF BOREHOLE REFUSAL TO AUGER ADVANCE															

RECORD OF BOREHOLE No 1005

1 OF 1

METRIC

W.P. 466-93-00 LOCATION Station 20+010 60m Lt Med C/L ORIGINATED BY MA
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem Augering / Wash Boring / Casing COMPILED BY AD
DATUM Assumed DATE 23 August 1999 CHECKED BY SP

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						WATER CONTENT (%)
								UNCONFINED		FIELD VANE				
323.8							20	40	60	80	100			
0.0	0.15m TOPSOIL dark brown SAND, trace Silt and organics		1	SS	4									
323.1			2	SS	80									Probable cobble
0.7	brown SAND some Gravel trace Silt compact to dense wet		3	SS	22									
			4	SS	31									28 64 8 0
319.0	grey		5	SS	18									Auger refusal @ 5m depth, started using NW casing @ 5m depth.
4.9	grey SAND and GRAVEL to GRAVELLY SAND trace Silt frequent Cobbles and Boulders		6	SS	21									
			7	SS	32									
315.6			8	RC										Coring commenced @ 7.9m depth
8.2	GNEISS BEDROCK massive, moderately closely to closely jointed, occasional micaceous layer		9	RC										RC 8 REC=83% RQD=42%
			10	RC										RC 9 REC=100% RQD=57%
			11	RC										RC 10 REC=78% RQD=72% RC 11 REC=100% RQD=76%
312.8														
11.0	END OF BOREHOLE WL on completion: Not stabilized due to water used for corng, but likely at about 0.8m depth													

RECORD OF BOREHOLE No G1

1 OF 1

METRIC

W.P. 466-93-00 LOCATION Station 19+950 30m Rt Med C/L ORIGINATED BY MA
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem Augering COMPILED BY AD
DATUM Assumed DATE 20 January 1999 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100	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								
323.2 0.0	PEAT						323						
322.1 1.1							322						
	some Gravel						321						
							320						
							319						
							318						
							317						
	grey SAND trace Gravel wet						316						
							315						
							314						
	occasional cobbles						313						
							312						
							311						
310.4 12.8	END OF BOREHOLE REFUSAL TO AUGER ADVANCE												

RECORD OF BOREHOLE No SB5

1 OF 1

METRIC

W.P. 466-93-00 LOCATION Station 20+000 19m Lt Med C/L ORIGINATED BY MA
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem Augering COMPILED BY AD
DATUM Assumed DATE 20 January 1999 CHECKED BY SP

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					
323.5 0.0	0.4m TOPSOIL		1	SS	5									
			2	SS	14									
			3	SS	13									
			4	SS	10									
317.5 6.0	END OF BOREHOLE REFUSAL TO AUGER ADVANCE													

+ 3, X 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE



Engineering Office
Structural Section
447 McKeown Ave., Suite 301
North Bay, ON P1B 9S9
Tel (705) 497-5529
Fax (705) 497-5426
email:Mike.McCormick@mto.gov.on.ca

To: I. Husain, P. Eng.,
Head Design Engineer
Bridge Office

Date: March 30, 2000

T. Sangiuliano, P. Eng.,
Foundation Engineer
Pavements and Foundations Section

Re : GWP 466-93-00
Hwy 11, District 52-Huntsville

Attached please find a copy for your files of the Foundation Design and Investigation reports for the following structure for the above noted project:

Culvert P3 at Sta. 19+984, Site 44-304

Sincerely,

M. S. McCormick, P. Eng.,
Senior Structural Engineer

MEMORANDUM



To: V. Minassian
Project Manager
Highway Engineering
Northern Region

Date: April 6, 2000

From: Pavements and Foundations Section
Room 223, Central Building
1201 Wilson Avenue
Downsview, Ontario

Tel: (416) 235-3731
Fax: (416) 235-5240

Re: Evaluation of Performance of Foundation Engineering Consultant
AGRA Earth and Environmental
For
Draft Foundation Investigation and Design Report (dated September 1999)
Culvert at Station 19+984
WP 466-93-00
Hwy 11, Huntsville

*Final Report reviewed all
and in general have been
concerns met. The only
outstanding item
is the
culvert
sent out
suggested
outlet
by*

We have reviewed the above-noted report to determine the Consultant's performance in providing the deliverables as specified in the Terms of Reference for this assignment. The accuracy of the subsurface information and the adequacy of the technical aspects of the recommendations have not been reviewed and remain the responsibility and liability of the Consultant. The Ministry assumes no responsibility or liability for these aspects of the report.

Based on our review, the report does not conform to the requirements of the Terms of Reference per the following comments:

- 1) Site number is not indicated ✓
- 2) Borehole elevations are to be references to appropriate Geodetic datum ✓
- 3) Recommendations for construction joints should be provided. Presently, this is discussed in the first paragraph under the Section – Discussion and Recommendations. However, a direct recommendation is needed. ✓
- 4) Bearing resistance for lesser settlements should also be included. (25 mm) (125 kPa) ✓
- 5) In predicting the magnitude of settlement induced by "up to 18 m of embankment fill", the type of fill shall be identified (rock fill) ✓
- 6) Recommendations and a NSSP for unwatering with responsibility assigned to the Contractor. ✓
- 7) Recommendations for pile foundations are considered excessive. ✓
- 8) Recommendations for culvert camber should be provided. ✓

- 9) The size of rock backfill to the culvert should be limited and specified.
- 10) Recommendations for a clay seal alternative to head and wing walls should be provided.
- 11) Consideration should be given to specifying rock fill, which is machine placed, instead of rip rap, which is hand placed.
- 12) The size and plan extent of rock fill erosion protection at both the inlet and the outlet should be provided. *lateral extent of clay not specified.*
- 13) Specific recommendations should be provided for the material, thickness and extent of the outlet filter.
- 14) Recommendations should be provided for the extent of rockfill inlet and outlet channel blankets.
- 15) The foundation elevation for which the bearing resistances for the open footing culvert should be provided.
- 16) Consideration should be given to the box culvert alternative.
- 17) Foundation Investigation (portion of) Report not signed and sealed separately as required.

It is recommended that the consultant should be requested to acknowledge and address our comments.

If there are any questions, please call.

Dave Dundas, P.Eng.
Senior Foundation Engineer

MEMORANDUM



To: V. Minassian
Project Manager
Highway Engineering
Northern Region

Date: November 5, 1999

From: Pavements and Foundations Section
Room 223, Central Building
1201 Wilson Avenue
Downsview, Ontario

Tel: (416) 235-3731
Fax: (416) 235-5240

Re: Evaluation of Performance of Foundation Engineering Consultant
AGRA Earth and Environmental
For
Draft Foundation Investigation and Design Report (dated September 1999)
Culvert at Station 19+984
WP 466-93-00
Hwy 11, Huntsville

We have reviewed the above-noted report to determine the Consultant's performance in providing the deliverables as specified in the Terms of Reference for this assignment. The accuracy of the subsurface information and the adequacy of the technical aspects of the recommendations have not been reviewed and remain the responsibility and liability of the Consultant. The Ministry assumes no responsibility or liability for these aspects of the report.

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- 1) Site number is not indicated
- 2) Borehole elevations are to be references to appropriate Geodetic datum
- 3) Recommendations for construction joints should be provided. Presently, this is discussed in the first paragraph under the Section – Discussion and Recommendations. However, a direct recommendation is needed.
- 4) Bearing resistance for lesser settlements should also be included.(25 mm)
- 5) In predicting the magnitude of settlement induced by “up to 18 m of embankment fill”, the type of fill shall be identified(rock fill)
- 6) Recommendations and a NSSP for unwatering with responsibility assigned to the Contractor.
- 7) Recommendations for pile foundations are considered excessive.

- 8) Recommendations for culvert camber should be provided.
- 9) The size of rock backfill to the culvert should be limited and specified.
- 10) Recommendations for a clay seal alternative to head and wing walls should be provided.
- 11) Consideration should be given to specifying rock fill, which is machine placed, instead of rip rap, which is hand placed.
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- 16) Consideration should be given to the box culvert alternative.
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It is recommended that the consultant should be requested to acknowledge and address our comments.

If there are any questions, please call.

Dave Dundas, P.Eng.
Senior Foundation Engineer

DOCUMENT MICROFILMING IDENTIFICATION

G.I.-30 SEPT. 1976

GEOCRES No. _____

DIST. 52 REGION _____

W.P. No. 466-93-00

CONT. No. _____

W. O. No. _____

STR. SITE No. 44-393

HWY. No. 518

LOCATION Hwy 518 (DEER LAKE Rd)

UNDERPASS

No of PAGES -

=====

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. _____

REMARKS: _____

**FOUNDATION INVESTIGATION AND DESIGN REPORT
FOR
PROPOSED HIGHWAY 11
HIGHWAY 518 (DEER LAKE ROAD) UNDERPASS
STRUCTURE SITE NO. 44-393
DISTRICT 52, HUNTSVILLE
W.P. 471-93-01**

Submitted To:

**Delcan Corporation
133 Wynford Drive
North York, Ontario, M3C 1K1
Canada**

Submitted By:

**AGRA
104 Crockford Blvd.
Scarborough, Ontario, M1R 3C6
Canada**

**September 1999
TT98820A**

September 30, 1999.

Ref. No.: TT98820A

Delcan Corporation
133 Wynford Drive
North York, Ontario, M3C 1K1
Canada

Attention: Mr. Khaled El-Dalati, P. Eng.

Dear Sir:

**Re: FOUNDATION INVESTIGATION AND DESIGN REPORT
FOR
PROPOSED HIGHWAY 11
HIGHWAY 518 (DEER LAKE ROAD) UNDERPASS
STRUCTURE SITE NO. 44-393
DISTRICT 52, HUNTSVILLE
W.P. 471-93-01**

We take pleasure in enclosing seven (7) copies of our Foundation Investigation and Design Report carried out for the above mentioned project and we will be glad to discuss any questions arising from this work.

Soil samples will be retained for a period of one year, and will thereafter be disposed of unless we are otherwise instructed.

We thank you for giving us this opportunity to be of service to you.

Sincerely,



George S.W. Chow, P. Eng.,
Designated MTO Contact.
GSWC/dee

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1.0 INTRODUCTION

AGRA, Consulting Geotechnical Engineers, was retained by Delcan Corporation to conduct a foundation investigation at the site of a proposed bridge that will carry Deer Lake Road and Highway 518 West over the realigned northbound and southbound lanes of proposed Highway 11 and associated interchange ramps. The site is located north of the Village of Emsdale, at the existing Deer Lake Road/Highway 518 West intersection with Highway 11, in the Township of Perry, Lot 14, Concession 12, MTO District 52 - Huntsville (see Key Plan, Drawing No. 1). The proposed bridge will be an approximately 69 m long, two span, 2-lane, structure.

The purpose of the investigation has been to obtain information about the subsurface conditions at the site of the proposed bridge and approach embankments by means of exploratory boreholes, and based on the findings, to provide recommendations for the foundation design of the proposed structure and approach fills.

2.0 SITE DESCRIPTION AND PHYSIOGRAPHY

The site is located at the intersection of Highway 11 and Deer Lake Road/Highway 518 West, north of the Village of Emsdale south of Burk's Falls and north of Huntsville. The ground elevation in the general area of the proposed bridge site is generally level, ranging in Elevation from about 335 to 334 m. The ground rises west of the proposed bridge site, and drops to the east. The existing bridge alignment is located along existing Deer Lake Road and Highway 518 West. The surrounding area is heavily wooded with commercial buildings located along Highway 11.

Based on available geologic information, the site is in an area of glaciolacustrine sediments. Generally after the last glacial withdrawal, ice-contact sediments (sands and gravels) followed by glaciofluvial sediments (ranging from deltaic and nearshore sands and gravels to prodeltaic and lake bottom silts and clays) were deposited on top of the existing sandy glacial till or Precambrian bedrock. The area was then inundated by glacial Lake Algonquin depositing sands, silts and clays in low lying areas.

Published information shows that the bedrock can be expected to be composed of strongly foliated, gneissic to migmatic rocks which form part of the Central Gneiss Belt of the Grenville Province (a structural subdivision of the Canadian Shield).

3.0 INVESTIGATION PROCEDURES

The field work for this project was performed during the period of January 11 to 22, February 16 to 18 and March 5 to 6, 1999, and consisted of drilling and sampling ten boreholes and conducting seven dynamic cone penetration tests. The plan locations of the boreholes, along with stratigraphic sections, are shown on Drawing No. 2. The boreholes could not be drilled at the exact proposed abutment and pier locations, due to the presence of overhead cables and underground Bell Telephone cables, but were drilled as close as possible to the actual locations.

The boreholes were advanced using solid and hollow stem continuous flight augers with a track-mounted power auger drilling rig (CME 75) owned and operated by Canadian Soil Drilling Inc. and a track-mounted power auger drilling rig (BOA 6M) owned and operated by Groundworks Drilling Inc., under the full-time supervision of an engineer from AGRA.

Sampling in the boreholes was effected at frequent intervals of depth by the Standard Penetration Test Method (SPT), as specified in ASTM Method D 1586. This consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm diameter o.d. split barrel (split-spoon) sampler into the ground. The number of blows of the hammer to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.30 m is recorded as the Standard Penetration Resistance or the 'N'-value of the soil and this gives an indication of the consistency or the compactness condition of the soil deposit.

In addition, dynamic cone penetration tests were performed at seven borehole locations. This test consists of driving a 60° point, 50 mm diameter cone attached to the drill rod continuously, into the undisturbed ground with a driving energy of 475 J (63.5 kg hammer falling freely a distance of 76 cm) per blow. The number of blows for each 0.30 m of penetration is recorded and this provides an indication of the relative changes in the soil density with depth.

Due to difficult drilling conditions at Borehole DL1, the borehole was advanced below a 9 m depth by washboring methods, employing NW size casing.

The borehole locations were established in the field by our engineering staff, in relation to the already staked out centre-line of Highway 518/Deer Lake Road (by Dearden and Stanton Limited). The borehole geodetic elevations and co-ordinates were later taken and provided to us by surveyors from Dearden and Stanton Limited.

The soil samples were shipped in sealed containers to our geotechnical laboratory in Toronto (Scarborough) for further examination and classification. A laboratory testing programme, consisting of natural moisture content and bulk unit weight determinations, Atterberg limits tests and grain-size analyses, was performed on selected representative soil samples. The results of the laboratory tests are presented on the appropriate Borehole Log Sheets and also on Figure Nos. 1 to 9.

The boreholes were left open until the end of each work day to enable us to take additional water level readings. Standpipe piezometers were installed in Boreholes DL1, 2, 3, 5, 7, and 8, although the piezometer installed in Borehole DL1 did not function properly. All boreholes were backfilled on completion.

4.0 SUBSURFACE CONDITIONS

The subsurface conditions were explored at ten borehole locations (Borehole Nos. DL1 to DL9 and DL5A), and were inferred at the locations of seven dynamic cone penetration tests. The locations of the boreholes and dynamic cone penetration tests are shown on the Plan and Profile Drawing No. 2 and are also indicated on the individual Borehole Log Sheets. Cross sections of inferred subsurface stratigraphy are also given on Drawing No. 2.

The ground surface at the proposed site is generally level. The ground elevation at the proposed bridge location generally ranges from about 335 to 334 m.

In general, the boreholes contacted, below a surficial granular pavement fill, granular soil deposits ranging from relatively finer materials (silty fine sand to sand) near the surface, becoming somewhat coarser with depth (i.e. sand to sand & gravel). In Boreholes DL2, 5 and 5A, drilled at the east abutment location, the sand has frequent clayey silt and silt interbeds below a depth of about 8.5 m or Elevation 326 m and extending to about 13 to 16 m. At the time of the investigation the groundwater table was recorded at depths of about 11 to 15 m below the existing grade or at Elevations generally ranging between about 323 and 319 m. A perched watertable was also contacted at depths of about 5 to 7 m below existing grade (approximately Elevations 329 to 327 m) overlying the clayey silt and silt interbeds in the sand at the proposed east abutment location.

Details of the subsurface conditions encountered in the boreholes are presented on the Borehole Log Sheets. The following paragraphs are only meant to complement and summarize these data.

4.1 IRREGULAR MIXTURE OF SAND, GRAVEL AND SILT (FILL MATERIAL)

The boreholes encountered 0.6 to 2.3 m of pavement fill. The pavement structure generally consists of 0.15 m of hot mix underlain by about 0.15 m of granular base course which is in turn underlain by sub-base granular materials. The base course material generally consists of sandy gravel, whereas the sub-base material ranges in composition from sand and gravel to silty sand. Six grain size analyses were conducted on the sub-base granular material resulting in the following grain size measurements, which are presented in envelope form in Figure No. 1:

Gravel:	1 - 42%
Sand:	47 - 74%
Silt:	18 - 33%
Clay:	0 - 2%

.../...

M:\reports\1999\deerlakea.wpd

Measured 'N'-values within this fill range from 3 to 60 blows/0.3 m, but were generally in excess of 15 blows/0.3 m indicating that the majority of the fill has received compactive effort. Measured natural moisture contents range from 5 to 34%.

4.2 SILTY FINE SAND TO SAND

Below the surficial pavement fill, Boreholes DL2, 3, 4, 5, 6, 7 and 8 encountered a granular deposit consisting of silty fine sand to sand. This deposit extends to depths ranging between 2.3 m or Elevation 332.7 m (Borehole DL7) and 12.0 m or Elevation 322.6 m (Borehole DL8). Grain size distribution analyses were conducted on thirteen samples from the material and the range of particle sizes are presented as a curve envelope in Figure No. 2. The analyses indicate:

Gravel:	0 - 17%
Sand:	51 - 95%
Silt and Clay:	5 - 49%

The deposit also contains some sandy silt to silt lenses. The grain size distribution of two samples from such lenses/seams is shown in Figure No. 3.

Measured 'N'-values in this deposit range from 10 to in excess of 50 blows/0.3 m indicating a compact to very dense condition, but generally compact to dense. Measured natural moisture contents range from 1 to 34%, but generally 3 to 19%.

In this deposit dynamic cone penetration tests yielded values ranging from 19 to in excess of 100 blows/0.3 m.

4.3 SAND WITH CLAYEY SILT TO SILT INTERBEDS

At the east abutment location (Boreholes DL2, 5 and 5A), below the surficial silty sand to sand deposit (at depths ranging from 8.4 to 8.6 m below the ground surface or below about Elevation 326 m), a sand deposit was encountered that contains frequent clayey silt to silt zones (or interbeds). The deposit consists of cohesionless sand with interbedded cohesive clayey silt and silt. It extends to about 13 and 16 m (Elevation 321.8 and 318.4 m) in Boreholes DL2 and 5, respectively. Two grain size distribution analyses were conducted on samples from the cohesionless sand from Borehole DL5 and the range of particle sizes is presented in Figure No. 4. The analyses indicate:

Gravel:	0 - 30%
Sand:	62 - 96%
Silt and Clay:	4 - 8%

Measured natural moisture contents range from 3 to 9%.

The thickness of the cohesive interbeds generally range from about 50 to 400 mm.

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Five grain size distribution analyses were conducted on samples from the cohesive interbeds. The results are presented in Figure No. 5. The analyses indicate:

Gravel:	0 - 25%
Sand:	19 - 40%
Silt:	29 - 71%
Clay:	6 - 15%

Atterberg Limits were conducted on five cohesive samples, indicating the following results (presented in Figure No. 9):

	Average	Range
Plastic Limit (%)	15.9	13 - 19
Liquid Limit (%)	20.8	16 - 26
Plasticity Index (%)	4.9	3 - 9
Moisture Content (%)	18	9 - 29
Unit Weight (kN/m ³)	-	20.1 - 21.4

The above results indicate that the interbedded layers generally consist of clayey silt to silt with minor zones of sandy silt and clay of low plasticity. The natural moisture content in the upper 1± m of the cohesive layers are generally higher than the material's measured liquid limit, whereas below this level the natural moisture content is below the liquid limit (in some cases below the plastic limit). Pocket penetrometer tests performed on the recovered split-spoon samples gave undrained shear strength values ranging from 120 to in excess of 220 kPa, indicating a very stiff to hard consistency.

The recorded 'N'-values in this deposit range from 15 to in excess of 50 blows/0.3 m. These results indicate a compact to very dense compactness condition (i.e. relative density) or very stiff to hard consistency.

The presence of occasional cobbles within the deposit was inferred while drilling and sampling.

4.4 SAND TO SAND & GRAVEL

Underlying the pavement fill and/or sand or clayey silt deposits described in the preceding sections, an ice-contact sand to sand & gravel deposit was encountered at Boreholes DL1, 2, 3, 5, 7 and 9. In Boreholes DL1, 7 and 9 (located near the central pier area), this deposit was contacted at surficial depths (i.e. 2.3 to 2.5 m below the ground surface) while in the remaining borehole areas it was encountered below depths ranging from 11.2 to 16.2 m. In all cases the deposit extended to the remaining depth of the boreholes. This is a cohesionless (granular) deposit and contains lenses of silty sand and occasional cobbles and boulders. Nineteen grain size distribution analyses were conducted and the range of particle sizes is presented as a curve envelope in Figure No. 6. The analyses indicate:

Gravel: 0 - 56%
Sand: 26 - 88%
Silt and Clay: 3 - 21%

Figure 7 shows the grain size distribution of two relatively finer layers in the deposit.

Measured 'N'-values in this deposit range from 17 to in excess of 50 blows/0.3 m indicating a compact to very dense condition. Measured natural moisture contents range from 3 to 19%.

Dynamic cone penetration results range from 24 to greater than 100 blows/0.3 m. In order to advance the boreholes at the central pier location (Borehole DL1), the borehole was cased and advanced using a tricone due to the presence of cobbles and boulders. Auger refusal on a boulder was encountered in Borehole DL9 at a depth of 9.8 m. In order to advance the borehole, a dynamic cone penetration test was conducted at the bottom of the borehole to a depth of 25.3 m where refusal was encountered.

These observations also show that the presence of cobbles and boulders can be expected in this deposit.

4.5 SANDY SILT

In Borehole DL8 sandy silt was encountered at 12.0 m (Elevation 222.6 m). This deposit was penetrated for 0.3 m where the borehole was terminated. The grain size distribution of the deposit is presented in Figure No. 8.

Based on an 'N'-value of 50 blows/0.10 m the deposit is described as very dense.

4.6 GROUNDWATER CONDITIONS

Groundwater levels in the open boreholes were observed during the drilling and at the completion of each borehole. The water levels in the open boreholes were checked prior to removing the augers or casing. To enable us to measure water levels for a prolonged period of time without interference from surface water, standpipe piezometers were installed in Boreholes DL1, 2, 3, 5, 7 and 8. The standpipe piezometer in Borehole DL1 did not function and is believed to have been plugged at depth as it retained drilling mud from the drilling process.

The recorded values, shown on the individual Borehole Log Sheets, indicate that the groundwater levels at the time of the investigation generally ranged from 11 to about 15 m below the ground surface (Elevations 323 to 319 m).

A perched watertable was encountered during drilling in Boreholes DL2 and 5, at a depth of about 5.0 to 7.0 m below existing grade, or at Elevations 329.4 to 327.6 m, respectively. This perched watertable lies within the surficial silty fine sand to sand deposit which overlies the sand with cohesive interbeds at the proposed east abutment location.

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It should, be pointed out that the groundwater at the site would fluctuate seasonally and can be expected to be somewhat higher during the spring months and in response to heavy rains.

5.0 DISCUSSION AND RECOMMENDATIONS

The proposed Highway 11 realignment will consist of a four lane divided highway with an approximately 30 m wide median. The proposed bridge will carry Deer Lake Road and Highway 518 West over the realigned northbound and southbound lanes of Highway 11 (the southbound lane will follow the alignment of the existing Highway 11) and the associated interchange ramps. It will be an approximately 69 m long, 2-lane (11 m wide), two span structure. The finished grade for the bridge will range from 338.0 m at the west abutment to 337.0 m at the east abutment location. The existing ground elevation at the bridge location is generally level (335 to 334 m), while the grade beyond the bridge location rises to the west and drops to the east. The proposed grade of Highway 11 under the bridge is approximately Elevation 330 m (approximately 329 m along the median). The existing grades under the bridge will therefore be lowered to build the highway, while the existing grades at the abutments will be raised by about 2.4 to 3.0 m.

Due to the presence of overhead utility cables and underground Bell Telephone cables, the exact proposed pier and abutment locations were not accessible. The boreholes were therefore drilled offset from the actual foundation elements, as close as practicable.

In general, the boreholes have shown beneath the pavement fill the presence of compact to very dense silty sand to sand & gravel deposits to the full extent of this investigation. These granular deposits are generally finer near the surface becoming coarser with increasing depth. In Boreholes DL2, 5 and 5A drilled in the proposed east abutment area the sand is interbedded with clayey silt and silt between about 8.5 and 13 to 16 m in Boreholes DL2 and DL5, respectively, below the ground surface.

The groundwater table at the time of our investigation was about 11 to 15 m below the ground surface, with a perched water table (perched in the sand overlying the less pervious clayey silt & silt interbeds) encountered at the proposed east abutment location at depths of 5 to 7 m.

5.1 FOUNDATIONS

As the proposed Highway 11 is in a cut (proposed highway grade of approximately 329 m vs existing grades of 335 to 334 m), the founding levels for a closed end abutment type bridge can be expected to be about 327 to 326 m. For a perched abutment type structure however, higher foundation elevations would be feasible.

The boreholes show that the use of conventional shallow spread footing foundations is feasible for both a closed end and a perched abutment type structure. Due to variable soil strength and compressibility (as evidenced by variable 'N'-values) and time rate of settlement (due to the clayey silt seams in the east abutment areas) however, we recommend that the soil immediately

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underneath the footings foot-print be removed to a depth of about 1.5 m and replaced with compacted Granular 'A' material.

Due to variable 'N'-values recorded throughout the depths explored and the presence of cobbles and boulders within the overburden, the use of deep foundations is not considered to be a viable option.

These options are discussed in more detail in the following sections.

5.1.1 Spread Footing Foundations

5.1.1.1 Spread Footing Foundations on Native Subgrade

For footings founded on competent, undisturbed, native compact to dense cohesionless sands or gravels, or on the very stiff to hard clayey silt interlayers, the following bearing capacities can be used:

FOUNDATION ELEMENT	APPROXIMATE DEPTH TO FOUNDING ELEVATION BELOW EXISTING GRADE (m)	BOTTOM OF FOOTING ELEVATION (m)	FACTORED BEARING RESISTANCE AT U.L.S. * (kPa)	BEARING RESISTANCE AT S.L.S. (kPa)
West Abutment Boreholes DL3 and DL8	5.5 - 8.0	329.0 - 326.5	650	250
Central Pier Boreholes DL1, DL7 and DL9	8.0 - 9.0	327.0 - 326.0	750	300
East Abutment Boreholes DL2, DL5 and DL5A	2.5 - 8.0	332.0 - 326.5	600	250

* Incorporating a resistance factor of 0.5 as per Ontario Highway Bridge Design Code (O.H.B.D.C.), 3rd Edition.

These values are applicable to both closed end and perched type abutments.

The serviceability condition is based on the premise that total and differential settlements will not exceed 25 mm and 20 mm, respectively.

As mentioned before, at the east abutment location a perched water level was recorded at Elevations 329.4 and 327.6 m in Boreholes DL2 and DL5, respectively. Depending on the water level at the time of construction and footing elevations chosen, dewatering will be required if the excavations extend below the perched water table. For this reason there may be an advantage to keep the footing level at this abutment location at or above Elevation 330.5 m (i.e. perched abutments). In any event, the dewatering will need to be capable of drawing the water level to no

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less than 0.7 m below the bottom of the excavation. When preparing the dewatering scheme, the presence of the impervious cohesive interbeds should be considered. If the water table is not properly lowered, the soil at the bottom of the excavation can lose its load carrying capacity (in addition to the instability of the side slopes). If this happens and the footing is placed on disturbed and loosened subgrade, excessive settlements can ensue after the application of the structural loads. For this reason we recommend that the contractor investigate the position of the water table before starting the excavation to assess required dewatering. Therefore we would recommend that a NSSP be included in the contract documents that alerts the Contractor of the subsurface and groundwater conditions and the potential for soil sloughing (as discussed above). Dewatering is not expected to be a major problem.

For frost protection, the footings should have a permanent earth cover of not less than 1.8 m.

Under inclined loading conditions the Factored Bearing Resistance at U.L.S. should be reduced in accordance with Clause 6-8.4.2 of O.H.B.D.C., 3rd Edition.

The unfactored horizontal resistance against sliding between concrete and approved sand to sand and gravel surface can be calculated using a friction angle of 32 degrees, whereas a friction angle of 29 degrees could be used for footings placed on silty sand to fine sand and 22 degrees on clayey silt interlayers.

5.1.1.2 Spread Footing Foundations on Improved Subgrade

In order to provide more uniform foundation subgrade conditions and to improve the load carrying capability of the upper zones of the foundation soils, it is recommended that conventional spread footing foundations be used after the improvement of the founding soil to a sufficient depth below the founding level at the proposed footing locations. In-situ soil improvement methods such as vibro-compaction, dynamic-compaction and grouting to densify the soil will likely be impractical due to the presence of frequent cohesive interbeds below the east abutment, as well as being expensive. It is therefore recommended that spread footings be used after the improvement of the overburden immediately beneath the footings by removing the soil to a sufficient depth and replacing it with compacted granular fill.

For this purpose the following approach is recommended. The soil beneath the proposed footing should be removed to a depth of not less than 1.5 m below the bottom elevation of the footing within an area at least 1.5 m beyond the perimeter of the proposed footing (for example for a footing measuring 12.0 x 5.0 m in plan, the size of the excavation at the bottom would be $5 + 1.5 + 1.5 = 8$ m by $12 + 1.5 + 1.5 = 15$ m in plan). The sides of the excavation would be sloped not steeper than 1H:1V (flatter if necessary) and therefore at the proposed founding level, the size of the excavation would be 11 m by 18 m or greater. The sides of excavations above the water table should not be sloped steeper than 1H:1V but may require flatter side slopes, especially since vibrations induced during compaction may create instability. If excavations extend below the water table (or perched water table) or come close to it, dewatering will be required.

When the excavation reaches the required depth, the subgrade should be evaluated and approved by a geotechnical engineer familiar with the findings of this report and appointed by the Contract Administrator. If necessary, the excavation may need to be deepened to the surface of a sufficiently competent soil. After its approval, the exposed subgrade at the base of the excavation may need to be compacted, if requested by the geotechnical engineer appointed by the Contract Administrator, to achieve a density of not less than about 98% of the material's Standard Proctor Maximum Dry Density (SPMDD). The fill used to raise the grade inside the excavation should consist of Granular 'A' material placed when its moisture content is within $\pm 2\%$ of its optimum moisture content. It should be placed in layers not exceeding 200 mm in thickness and should be uniformly compacted to not less than 100% of its SPMDD.

A factored bearing resistance at U.L.S. of 750 kPa and a bearing resistance at S.L.S. equal to 300 kPa can be assigned to soil prepared in this manner, except for the central pier where these values can be increased to 850 kPa and 350 kPa, respectively. The serviceability condition is based on the premise that total and differential settlements will not exceed 25 mm and 20 mm, respectively.

For frost protection, the footings should have a permanent earth cover of not less than 1.8 m.

A possible problem with this approach, as mentioned before, is the position of the perched water table at the east abutment. This has already been discussed previously in Section 5.1.1.1 and will not be further discussed here.

The recommended resistance values at U.L.S. and S.L.S. can be increased to 900 kPa and 350 kPa, respectively, if abutments (perched) are founded on an additional 1.2 m thick engineered fill consisting of Granular 'A' type material (as per MTO standards) placed on top of the subgrade prepared as discussed above. In this case the total thickness of the Granular 'A' pad (compacted in thin layers to at least 100% of the material's SPMDD) supporting the spread footing foundations would be at least 2.7 m (i.e. 1.5 m + 1.2 m) and the footing would be at a higher elevation (i.e. perched abutments). The construction of the Granular 'A' pad and of the earth fill should meet the minimum requirements as per Ontario Ministry of Transportation, as shown in Appendix B.

In any event as mentioned before, for frost protection, the footings should have a permanent earth cover of at least 1.8 m.

Under inclined loading conditions the Factored Bearing Resistance at U.L.S. should be reduced in accordance with Clause 6-8.4.2 of O.H.B.D.C., 3rd Edition.

The unfactored horizontal resistance against sliding between concrete and Granular 'A' type material can be calculated using a friction angle of 32.

5.1.2 Deep Foundations

Due to the presence of cobbles and boulders and variable 'N'-values recorded in the boreholes, pile lengths for driven piles can not be predicted with any certainty. This will lead to considerably different pile lengths and/or pile capacities and can be expected to create problems during the driving of the piles. For these reasons, the use of driven pile foundations is not recommended. If, however, further consideration of this aspect is required, we recommend drilling deeper boreholes at the site.

We were however asked to comment on the available pile resistances for steel H-piles within the depths drilled, in order to permit the consideration of an integral abutment type bridge. Based on the borehole data, the following table summarizes the estimated approximate pile tip elevations and pile resistances for HP 310x110 size driven steel H-piles, assuming that the pile top (head) elevations will not be lower than about Elevation 332 m at the abutment locations (for an integral abutment type bridge) and 326.5 m at the central pier.

TABLE

FOUNDATION LOCATION	REFERENCE BOREHOLES(S)	RECOMMENDED PILE TIP ELEVATION (m)	RECOMMENDED FACTORED RESISTANCE AT U.L.S.	RECOMMENDED RESISTANCE AT S.L.S.
West Abutment	DL3 and DL8	320	725kN	500kN
Central Pier	DL1, DL7 and DL9	316	560kN	400kN
East Abutment	DL2 and DL5	318	640kN	440kN

The recommended horizontal resistances for HP 310x110 steel H-piles are as follows:

$$\begin{aligned}\text{Factored Horizontal Resistance at U.L.S.} &= 100\text{kN/pile} \\ \text{Horizontal Resistance at S.L.S.} &= 50\text{ kN/pile}\end{aligned}$$

Unbalanced horizontal forces at the pier support could also be resisted by battered piles.

Due to the presence of cobbles and boulders, the piles should be equipped with reinforced tips.

The piles should be driven using a suitably heavy hammer capable of delivering a rated capacity of between 40 and 50 kJ/blow. An NSSP should be inserted in the contract documents specifying the restriction on the energy rating. When the driving of the piles reaches to about 0.6 m above the quoted elevations, the driving should be controlled by the Hiley Formula, assuming an ultimate capacity of about 1500kN.

The base of the pile caps (e.g. at the pier location) should be provided with a minimum earth cover of 1.8 m for frost protection.

At least two piles should be re-tapped at each support location (i.e. foundation element) to check that relaxation has not occurred. Re-tapping should take place about 24 to 48 hours after driving. If relaxation is deemed to have occurred, then all piles should be re-tapped.

Possible variable pile lengths should be taken into consideration when ordering the piles.

In order to minimize the effect of any downdrag, we recommend that the approach embankment fills be placed to their final grade elevation at least three weeks prior to driving the piles.

We recommend that dynamic pile testing (e.g. CAPWAP) be conducted during the installation of the piles.

As mentioned earlier, however, piled foundations are not recommended from a geotechnical engineering point of view.

5.2 LATERAL EARTH PRESSURES

Backfill behind abutments and retaining walls should consist of non-frost susceptible, free draining granular materials in accordance with the Ontario Ministry of Transportation Standards.

Free-draining backfill materials (i.e. Granular 'B' or Granular 'A') and the provision of drain pipes and weep holes, etc., should prevent hydrostatic pressure build-up. Computation of earth pressures should be in accordance with O.H.B.D.C., 3rd Edition. For design purposes, the following parameters (unfactored) can be used.

Compacted Granular 'A'

Unit Weight = 22 kN/m^3

Coefficient of Lateral Earth Pressures:

$$K_a = 0.27$$

$$K_o = 0.43$$

Compacted Granular 'B'

Unit Weight = 21 kN/m^3

Coefficient of Lateral Earth Pressures:

$$K_a = 0.31$$

$$K_o = 0.47$$

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or movements can be allowed such that the active state of earth pressure can develop. If the abutment is restrained and does not allow lateral yielding, then at-rest pressures should be used as per Clause C6-7.1 of the O.H.B.D.C., 3rd Edition. The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients in accordance with

Clause 6-7.4.3 of the O.H.B.D.C., 3rd Edition.

Vibratory equipment for use behind abutments and retaining walls should be restricted in size as per current MTO practice.

As an alternative to conventional retaining walls, MTO's Retained Soil System may be used. The following should be included in the Contract Documents:

- identify longitudinal extent in plan of the Retained Soil System
- identify in plan transverse space constraints (top of wall and bottom of wall)
- identify elevation of top of wall and bottom of wall
- include NSSP for Retained Soil Systems in Contract Documents

The Retained Soil System should be of high performance and moderate to high appearance.

5.3 APPROACH EMBANKMENTS

As the proposed finished bridge deck level is about Elevation 338.0 to 337.0 m, and the existing grade elevations at the immediate approaches are approximately 335 to 334 m, up to about 3.0 m and 2.4 m embankments will have to be built on the west and east sides, respectively. Based on the borehole results, the strength of the foundation materials is such that deep-seated failures are not anticipated, provided all organic soils, weak or otherwise unsuitable materials are removed as per MTO Standards before placing the fill.

All organic and other unsuitable soils should be removed within an envelope given by an imaginary slope not steeper than 1:1 from the toe of the proposed embankment as shown by the sketch in Appendix C. After stripping, the exposed subgrade should be inspected, approved and properly compacted from the surface under the supervision of qualified personnel.

Provided that all organic and otherwise unsuitable materials are removed and the subgrade is properly compacted from the surface as detailed above, the settlement of the foundation materials (i.e. not including the settlement of the embankment material under its own weight) should not exceed 20 mm and should be substantially completed within three to four weeks of placing the embankment fill to its full height. Such settlements are considered acceptable and will not necessitate preloading or surcharging.

Water level measurements in the boreholes indicate water levels between Elevations 323 and 319 m. Therefore we do not anticipate major problems due to groundwater seepage during stripping of the subgrade and backfilling for the construction of the embankments.

The materials used for the construction of the embankment fills should consist of approved, clean earth fill (e.g. Select Subgrade Materials - OPSS 1010). The fill should be placed in lifts not exceeding 300 mm before compaction and each lift should be uniformly compacted to at least 95% of the Material's Standard Proctor Maximum Dry Density. The degree of compaction within the top 0.5 m of the fill (i.e. the subgrade immediately beneath the granular sub-base) should be increased

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to 98%. The selection, placement and compaction of the fill should be carried out under a geotechnical engineer familiar with the findings of this report and appointed by the Contract Administrator. The settlement of the embankment fills prepared as described above should not exceed 20 mm.

Permanent cut slopes above the groundwater table will be stable at 2H:1V. For slope heights of more than 6 m, however, a 2 m wide mid-height bench (berm) is recommended. Cut slopes should be inspected after construction and where deemed necessary, measures such as granular blanket (sheeting) should be provided.

For both fill embankments and cut slopes proper erosion measures should be implemented both during the construction and permanently. This can be achieved by immediate seeding or sodding (OPSS 572).

5.4 CONSTRUCTION

Excavations should be carried out in accordance with the latest edition of the Ontario Occupational Health and Safety Act and its regulations.

To reduce the excavation depths and to facilitate the construction of spread footing foundations it would be advisable to carry out the cut for Highway 11 first.

In general the sides of temporary excavations above the water table will be stable at 1.5H:1V side slopes. This to a certain extent depends on the height of excavation, the length of time they will remain open, protective measures, etc. and as such somewhat steeper slide slopes may be feasible (i.e. 1H:1V) for shallow, short duration excavations while elsewhere some local flattening may be required.

Potential problems, depending on the groundwater table level at the time of construction and depth of excavations for footings on native soil or to prepare the engineered fill to support footings, due to groundwater, were discussed in Section 5.1.1.1 of this report and will not be repeated here. In general, however, no major problems due to groundwater are envisaged for excavations extending to about Elevation 324 m (i.e. to below the anticipated foundation excavation elevations), except for the perched water table at the east abutment location. Any surface water seepage, if necessary, can easily be handled by gravity drainage and pumping from open sumps.

All foundation excavations and bearing surfaces should be inspected and approved by a geotechnical engineer familiar with the findings of this report and appointed by the Contract Administrator. We recommend that following construction of the footing, backfill be placed to a height of at least 1.8 m above the footing to prevent disturbance and frost penetration.

It should be pointed out that, as discussed in Section 4.3 of this report, the measured natural moisture contents of the upper 1± m of the cohesive interbeds in the sand deposit, encountered in the boreholes drilled in the proposed east abutment area, are generally higher than the measured liquid limits and as such can be expected to be weak and compressible. This aspect

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should be kept in mind when inspecting and evaluating the foundation excavations as such interbeds may need to be subexcavated.

Allowance should be made to place an approximately 150 mm thick layer of lean concrete on the bearing surface to receive the foundations within four hours of preparation and acceptance of the bearing soil. It should be pointed out that if the foundation soil is disturbed, excessive settlements can occur after structural loads are applied.

6.0 CLOSURE

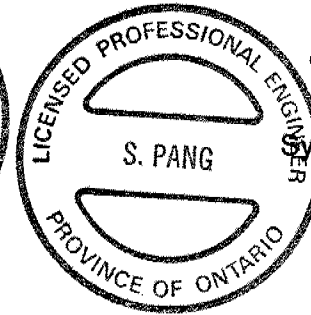
We recommend that once the details of the structure are finalized, our recommendations should be reviewed for their specific applicability.

The Limitations of Report, as quoted in Appendix A, is an integral part of this report.

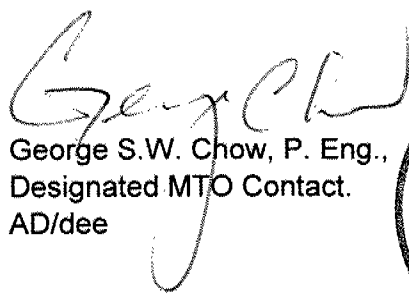
Sincerely,



Andrew Drevininkas, P. Eng.



Sydney Pang, P. Eng.



George S.W. Chow, P. Eng.,
Designated MTO Contact.
AD/dee



APPENDIX A

AGRA

LIMITATIONS OF REPORT

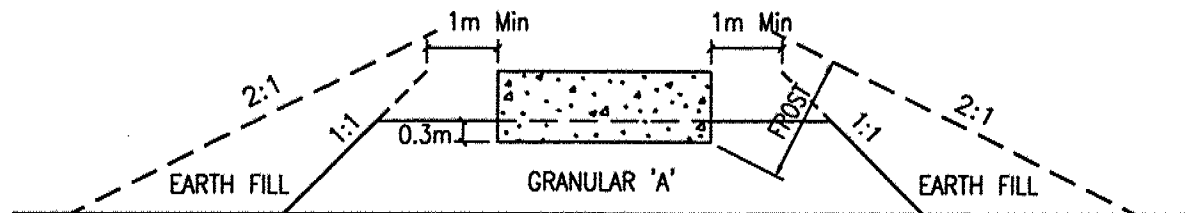
The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environmental aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. It is recommended practice that the Geotechnical Engineer be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in testholes. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report. Since all details of the design may not be known, we recommend that we be retained during the final design stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.

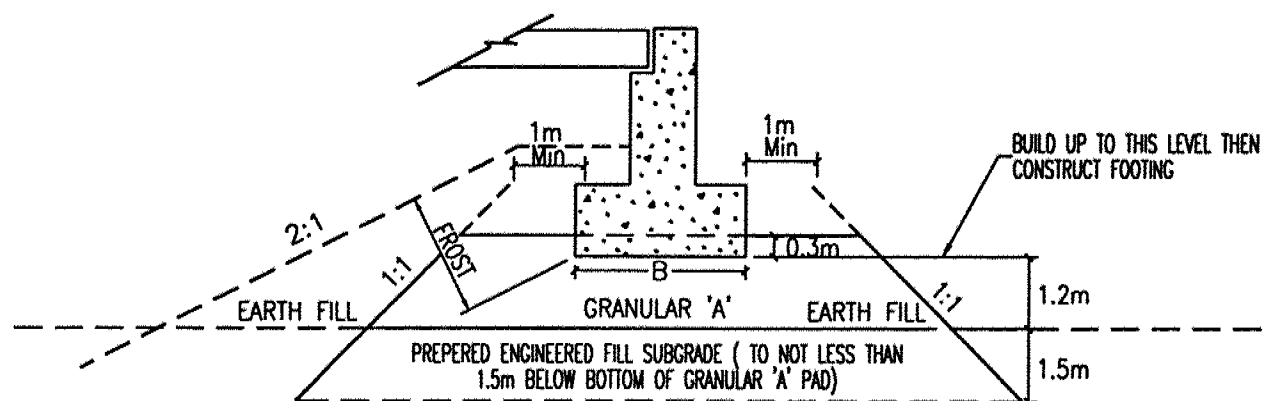
The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices. No other warranty is expressed or implied.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. AGRA accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

APPENDIX B



X SECTION



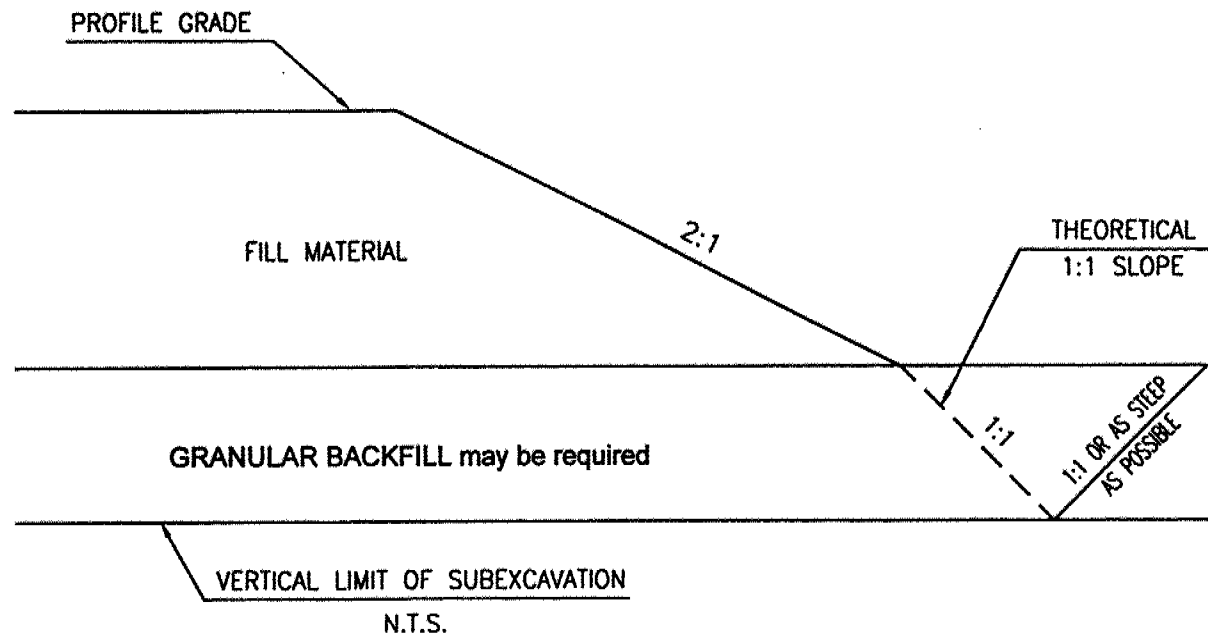
LONGITUDINAL SECTION

NOTES:

1. PREPARE SUBGRADE AS DISCUSSED IN REPORT.
2. PLACE GRANULAR 'A' & EARTH FILL TO BOTTOM OF FOOTING LEVEL COMPACTED ACCORDING TO CURRENT M.T.O. STANDARDS.
3. CONSTRUCT CONCRETE FOOTING.
4. PLACE REMAINDER OF GRANULAR 'A' & EARTH FILL AS REQUIRED.

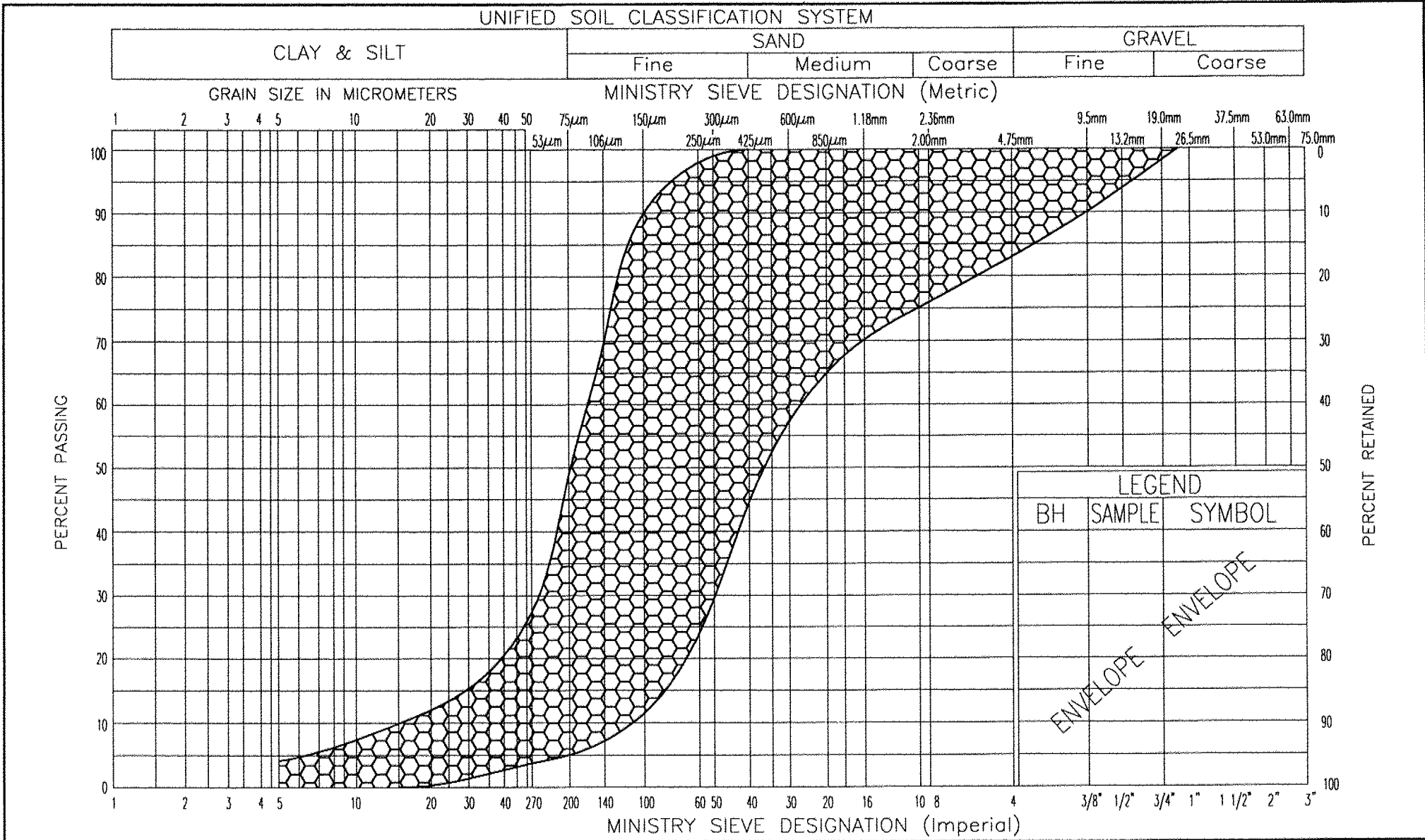
ABUTMENT ON COMPACTED FILL
SHOWING GRANULAR 'A' CORE
 N.T.S.

APPENDIX C



REMOVAL OF UNSUITABLE SOILS
FROM BENEATH APPROACH FILLS
N.T.S.

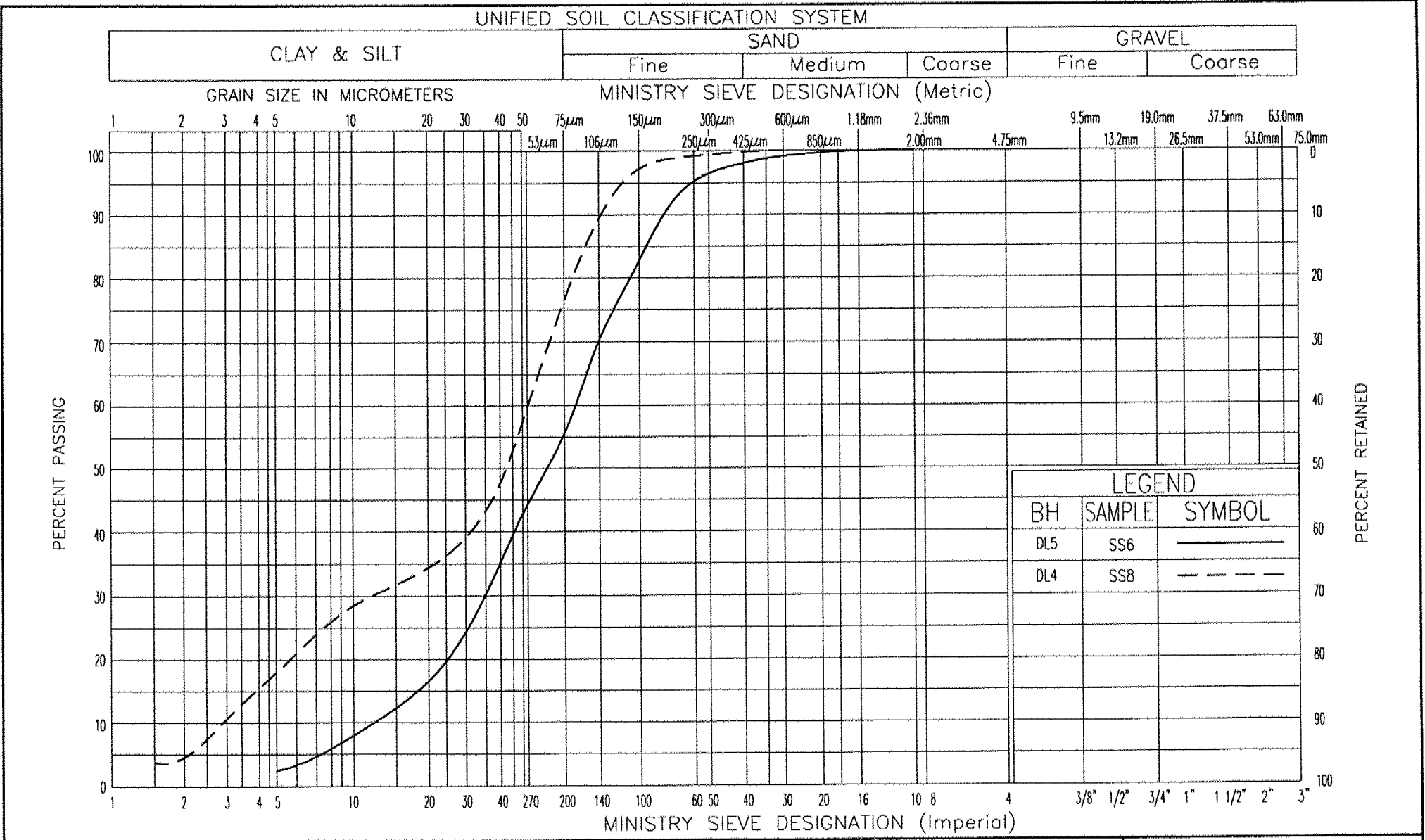
FIGURES



AGRA

GRAIN SIZE DISTRIBUTION
SILTY FINE SAND TO SAND

FIG No 2
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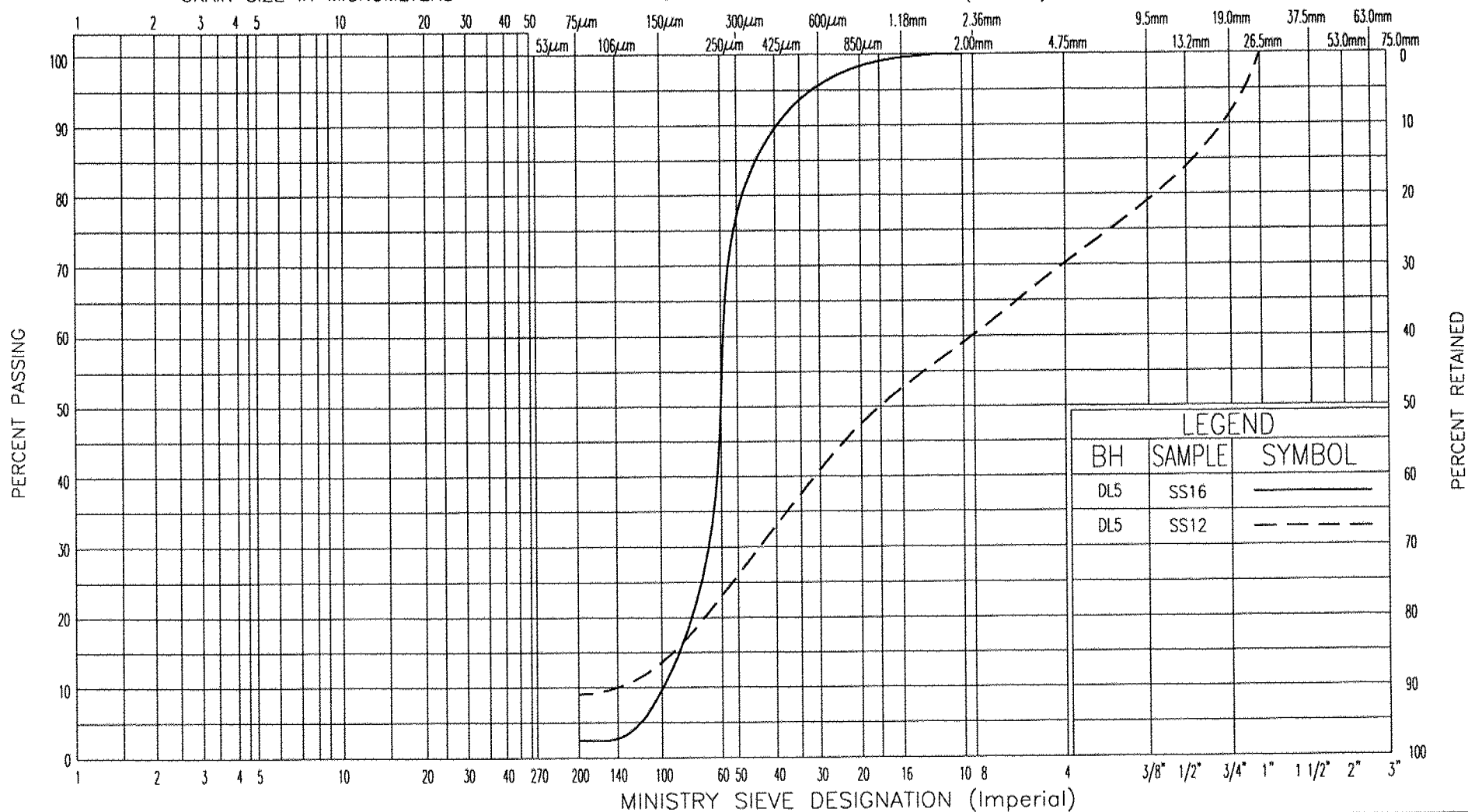


UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)

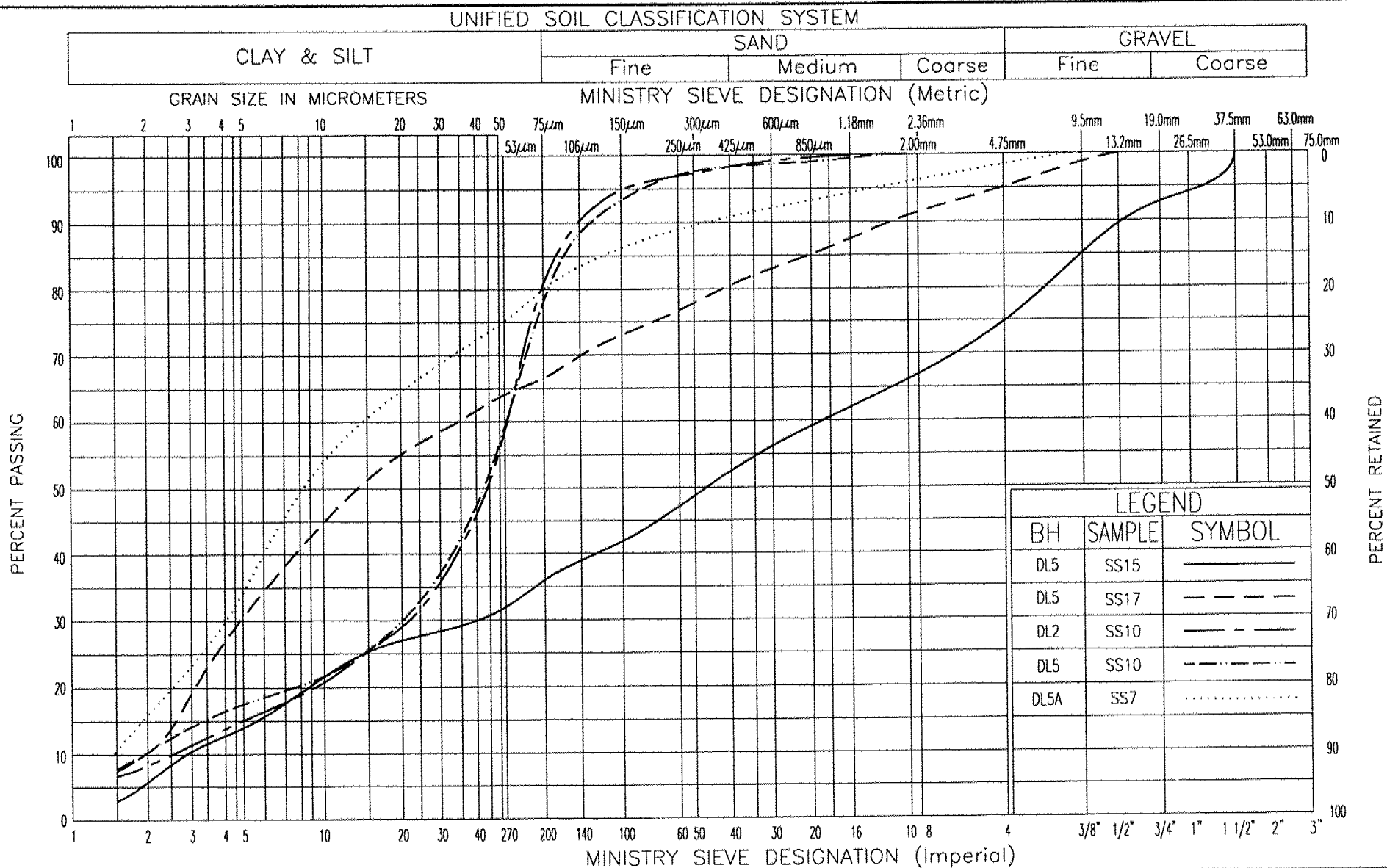


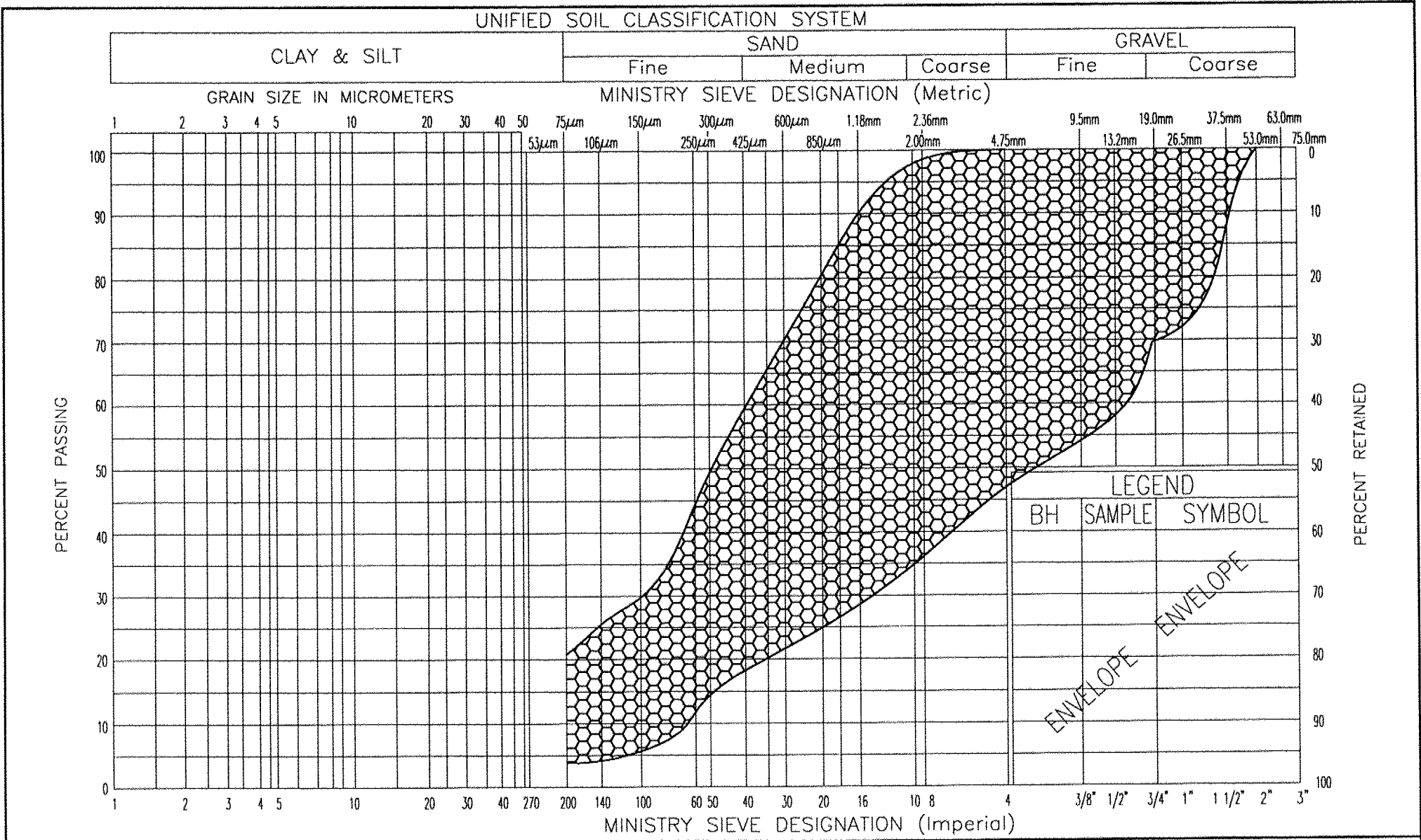
LEGEND		
BH	SAMPLE	SYMBOL
DL5	SS16	————
DL5	SS12	- - - - -



GRAIN SIZE DISTRIBUTION
SAND

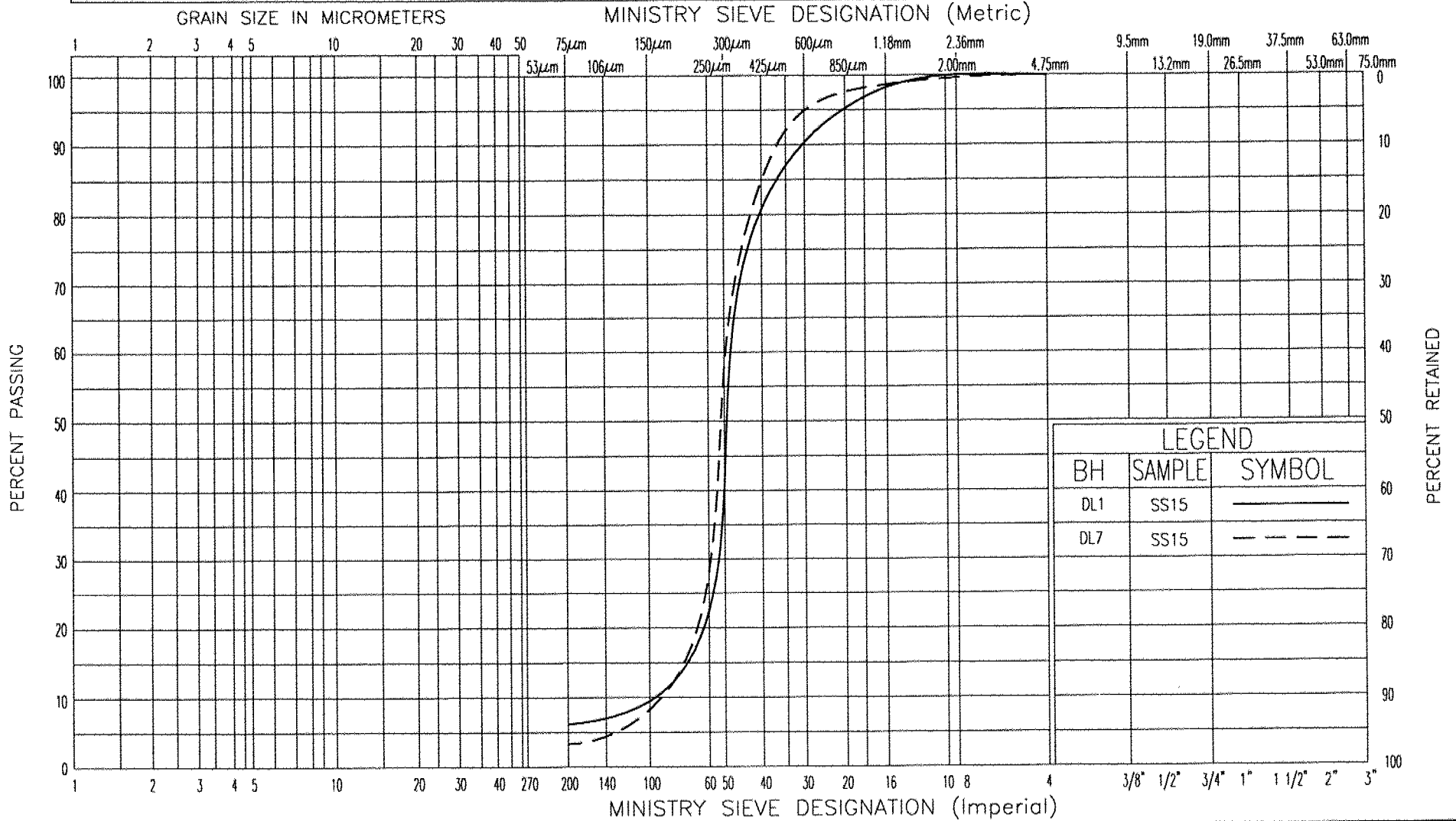
FIG No 4
W P 471-93-01





UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

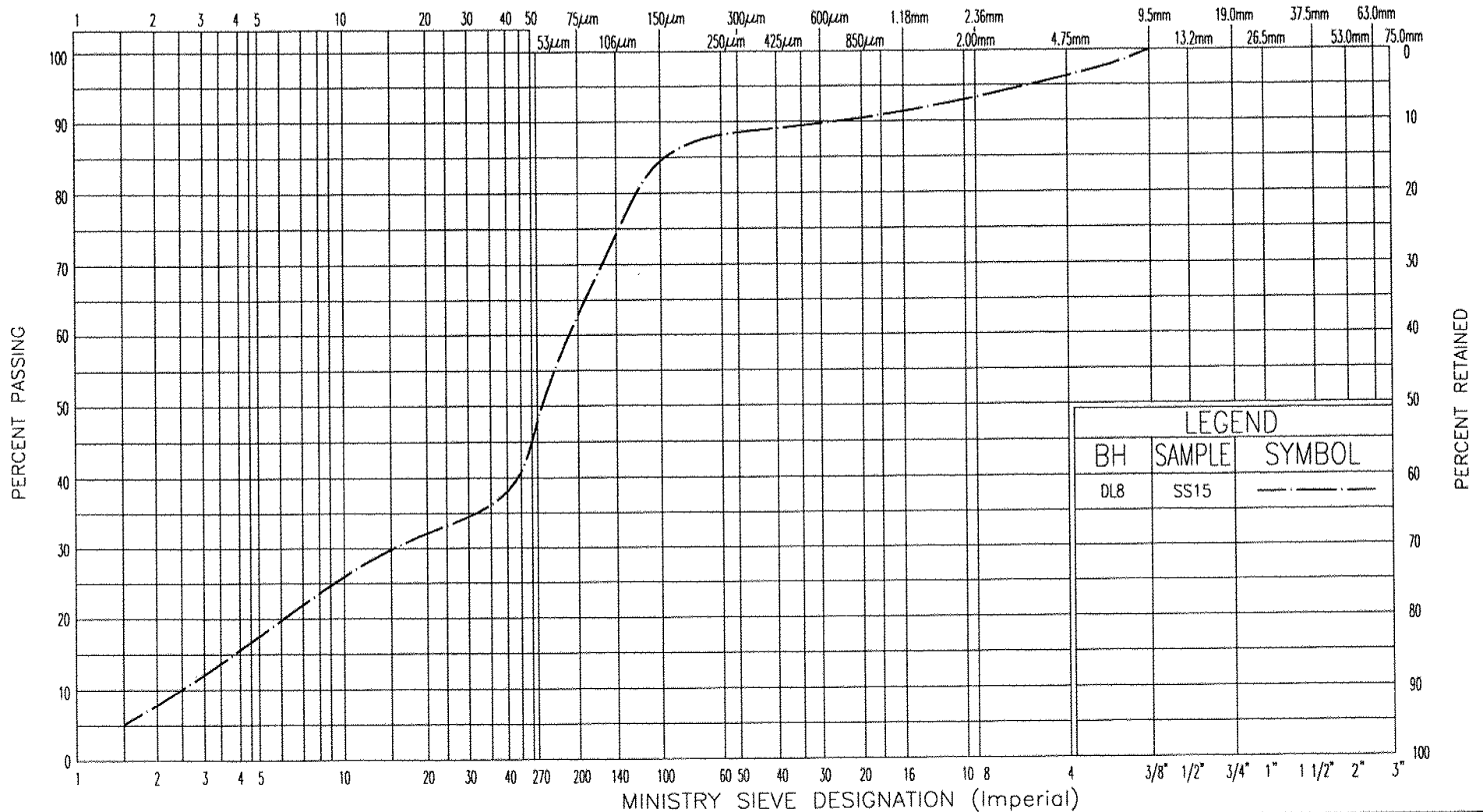


UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)

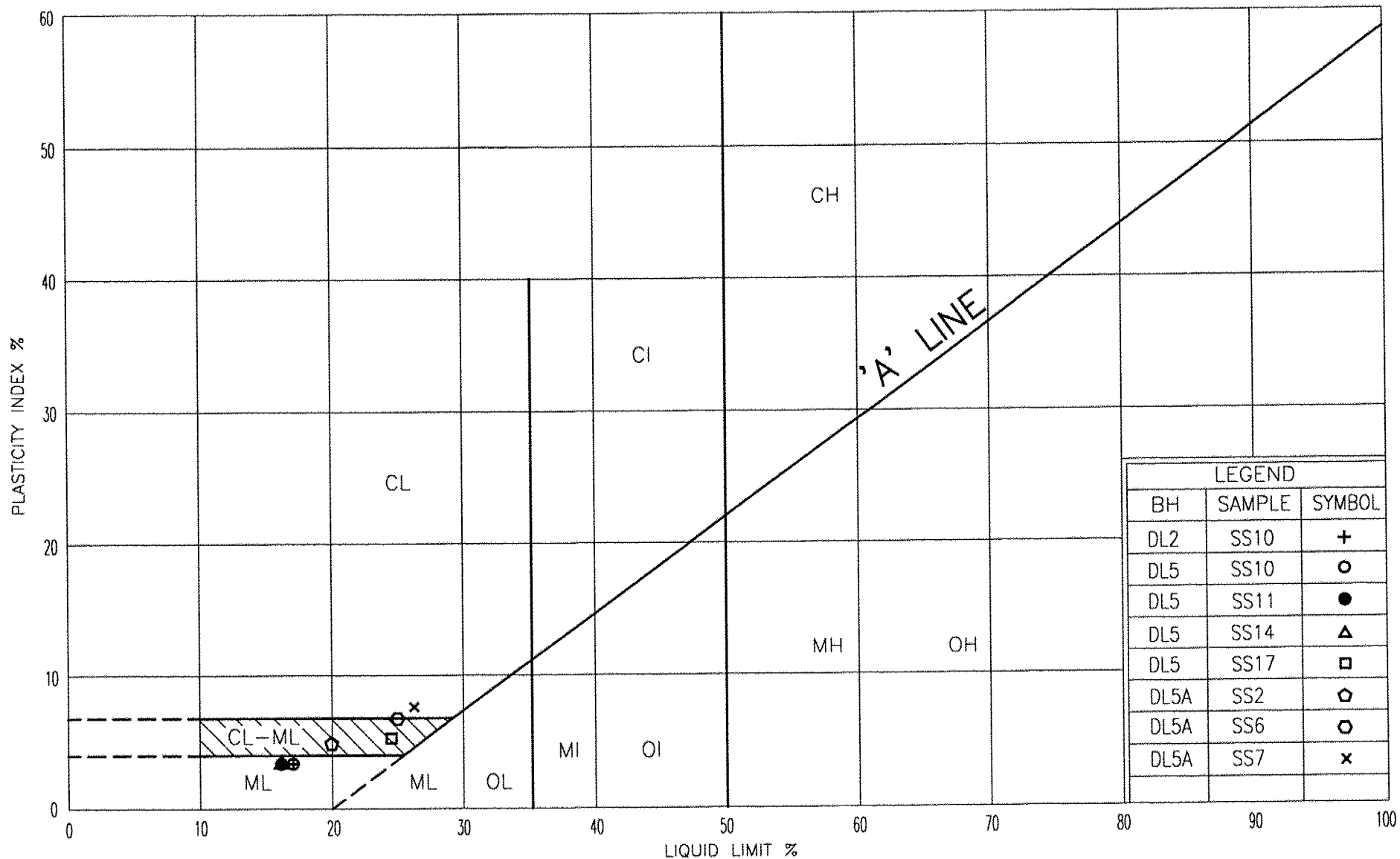


GRAIN SIZE DISTRIBUTION
SANDY SILT

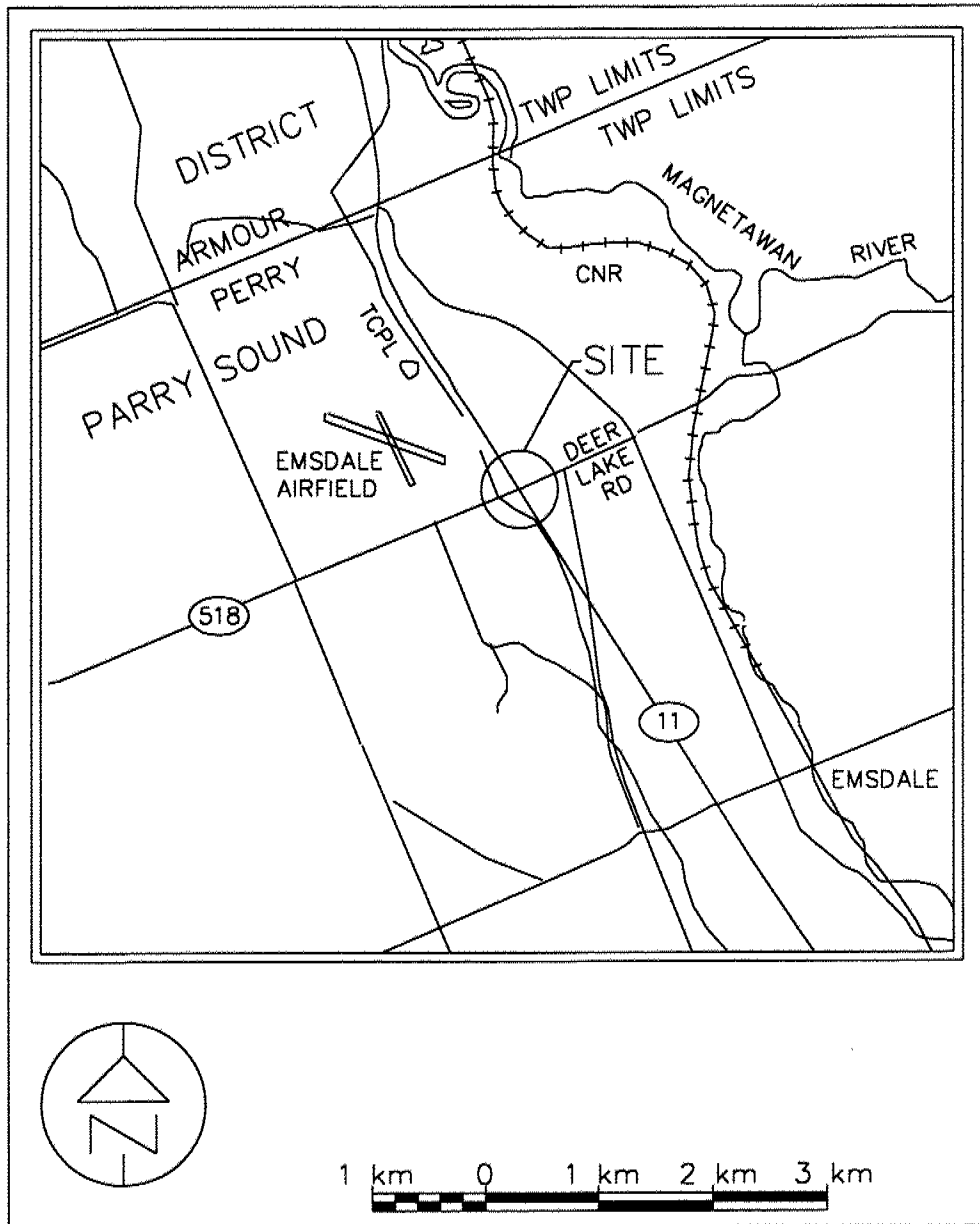
FIG No 8

W P 471-93-01





ENCLOSURES



HWY 518 (DEER LAKE ROAD) UNDERPASS
KEY PLAN

Dwg. No 1



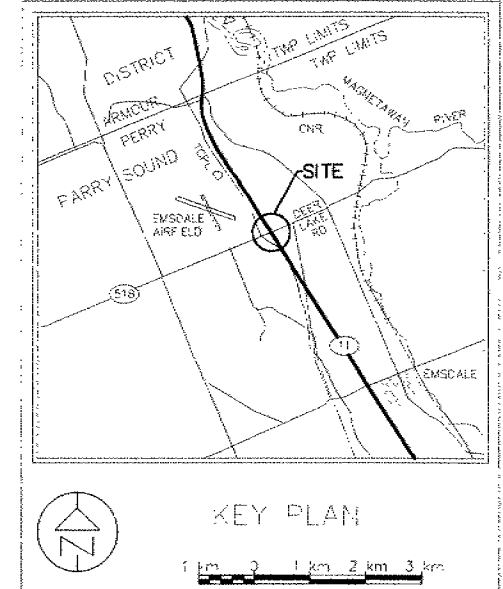
CONT. No.
W.P. No. 471-93-01

HWY 518 (DEER LAKE ROAD) UNDERPASS
BORE HOLE LOCATIONS & SOIL STRATA



SHEET

AGRA Earth & Environmental Ltd.



LEGEND

- Bore hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊕ Bore hole & Cone
- 'N' Blows/0.3m (Std Pen Test, 475 g/blow)
- CONE Blows/0.3m (60' Cone, 475 g/blow)
- WL at time of investigation Feb. 99
- WL in Piezometer Feb 15, 99
- Piezometer

No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
DL1	335.0	5 044 915	317 752
DL2	334.5	5 044 932	317 776
DL3	334.5	5 044 897	317 710
DL4	334.4	5 044 889	317 693
DL5	334.6	5 044 927	317 780
DL5A	334.6	5 044 928	317 781
DL6	334.2	5 044 934	317 797
DL7	335.0	5 044 921	317 750
DL8	334.6	5 044 904	317 756
DL9	335.0	5 044 910	317 744

NOTE

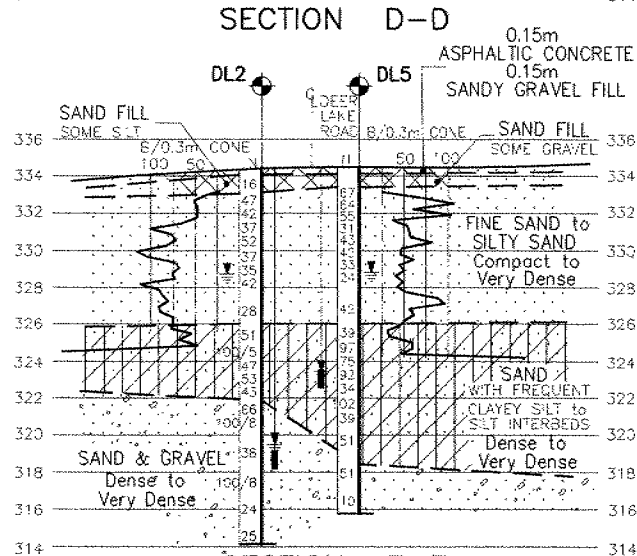
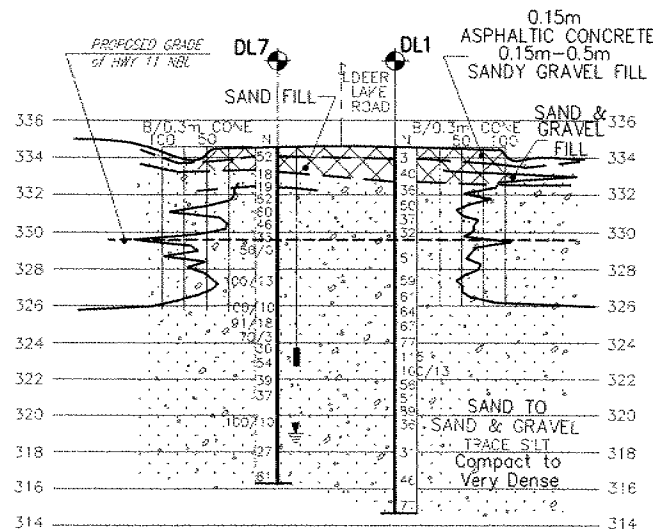
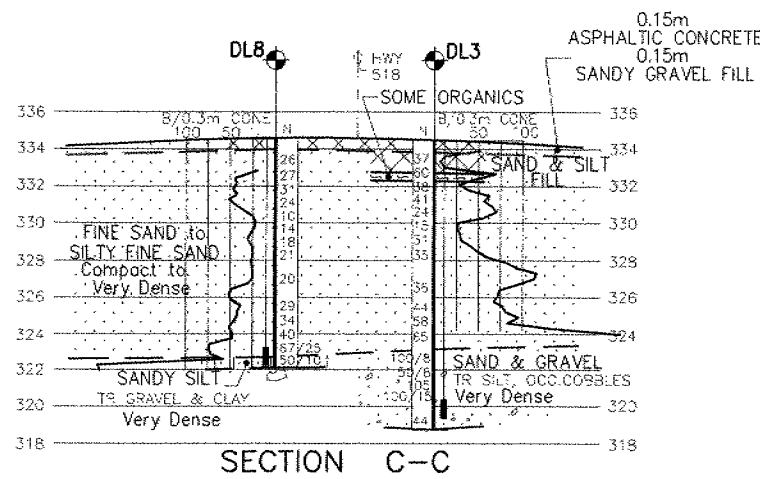
The boundaries between soil strata have been established only at Bore hole locations. Between Bore holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 2.3.1 of OPS Gen.Cond.

REV	DATE	BY	DESCRIPTION
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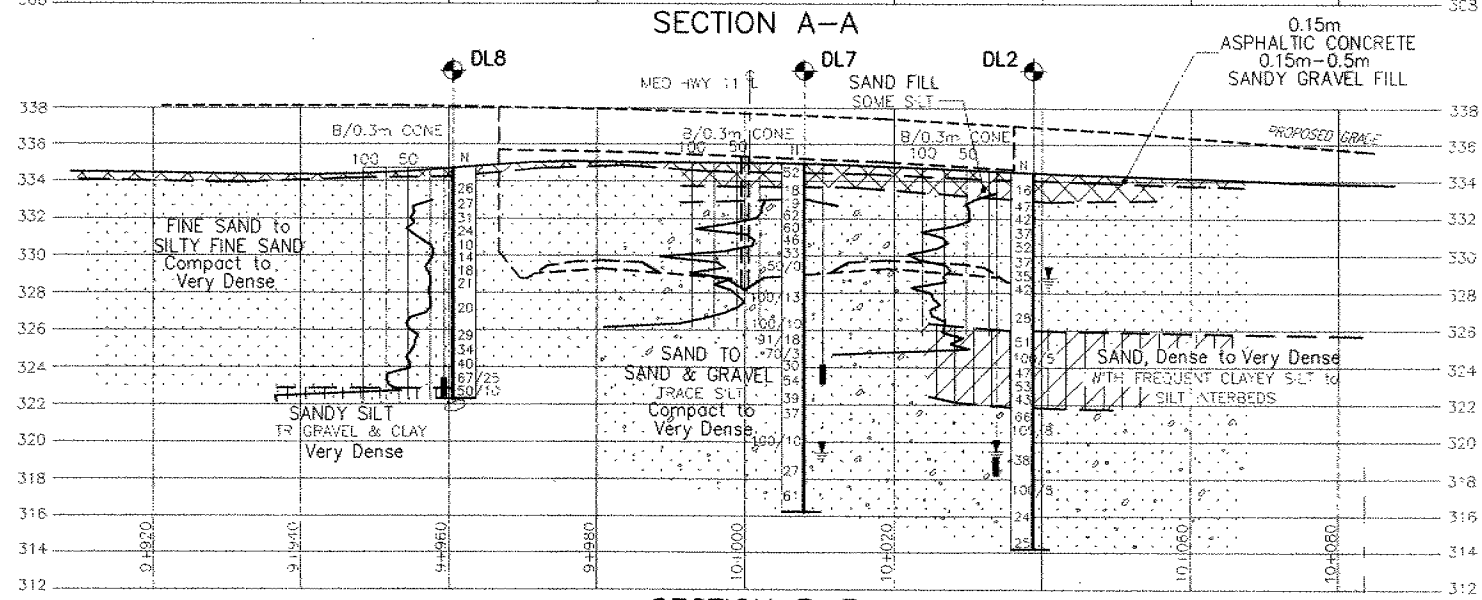
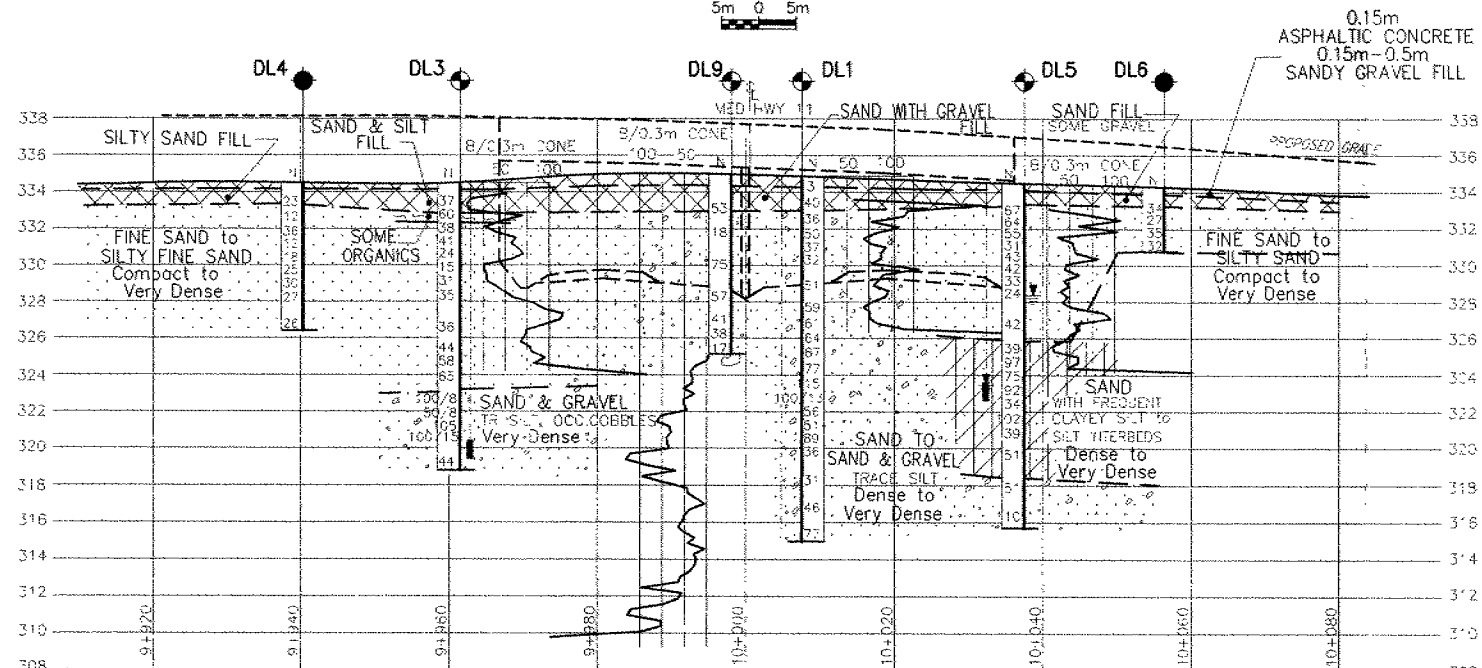
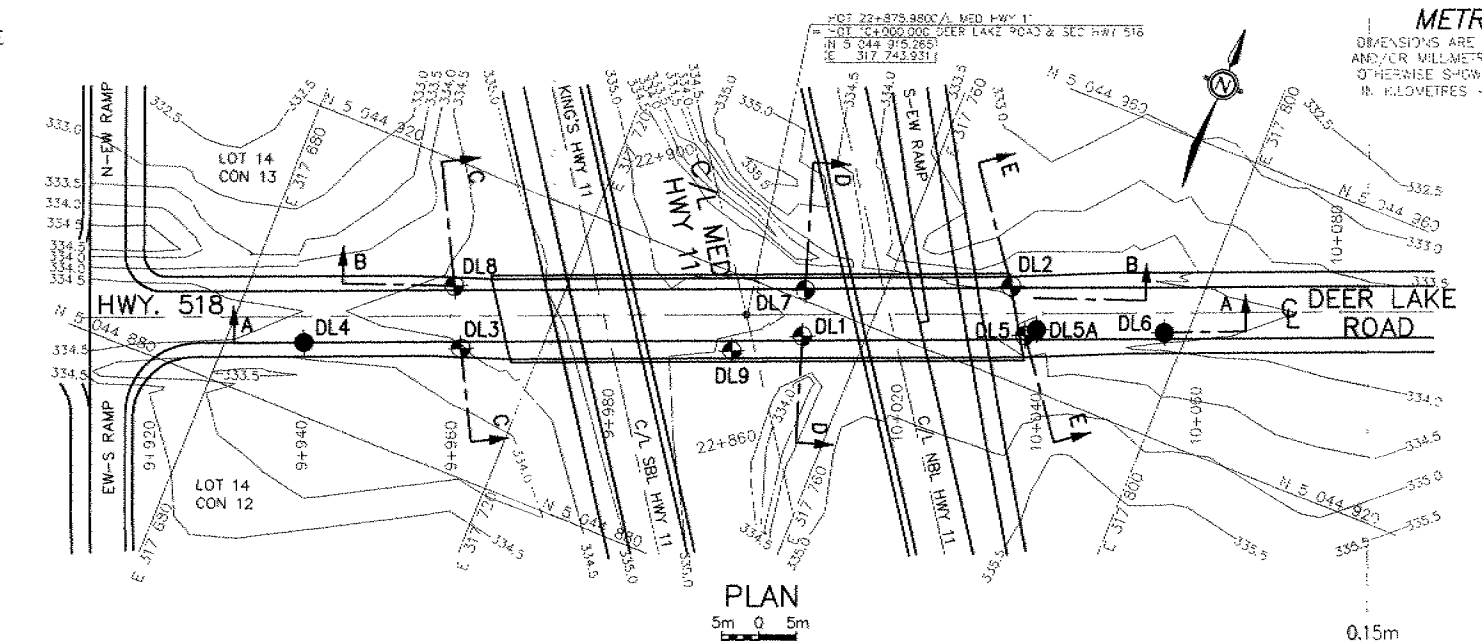
HWY No. 11	CHECKED SP	DATE Sep. 30, 1999	DIST 52-HUNTSVILLE
SUBMITTAL	CHECKED AD	DATE	STC 44-393
DRAWN MA	CHECKED AD	DATE	DWG 2

REF: Hwy 11 Bridge Site Plan
Dwg. by ELCAN, Jan 1999



SOIL STRATIGRAPHY LEGEND

- FILL
- SAND TO SAND & GRAVEL
TRACE SILT
Compact to Very Dense
- SANDY SILT
TRACE GRAVEL & CLAY
Very Dense
- FINE SAND to SILTY SAND
Compact to Very Dense
- SAND
WITH FREQUENT CLAYEY SILT
to SILT INTERBEDS
Compact to Very Dense



SECTION B-B

5m 0 5m HOR
2m 0 2m VER

RECORD OF BOREHOLE No DL1

1 OF 2

METRIC

W.P. 471-93-01 LOCATION N 5044915.4 E 317752.0 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem/Hollow Stem COMPILED BY CK
DATUM Geodetic DATE 21 January 1999 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE	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								
335.0	0.15m ASPHALTIC CONCRETE 0.5m Sandy Gravel FILL Sand & Gravel FILL trace silt damp		1	SS	3		334						Station 10 + 008 1.9 Rt
332.7			2	SS	40		333						42 47 (11)
2.3	SAND to SAND & GRAVEL trace silt with occasional cobbles and boulders dense to very dense damp		3	SS	36		332						24 73 (3)
			4	SS	50		331						39 54 (7)
			5	SS	37		330						Solid Stem Hollow Stem
			6	SS	32		329						
			7	SS	51		328						
			8	SS	59		327						43 52 (5)
			9	SS	61		326						Auger refusal on boulder at 9.1m
			10	SS	64		325						Tricone with NW casing
			11	SS	67		324						
			12	SS	77		323						
			13	SS	115		322						1 93 (6)
			14	SS	100/13		321						
			15	SS	56		320						
			16	SS	51		319						
			17	SS	89		318						
			18	SS	36		317						SS20: No Recovery
			19	SS	31								
			20	SS	46								

Continued Next Page

+ 3, X 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No DL1

2 OF 2

METRIC

W.P. 471-93-01 LOCATION N 5044915.4 E 317752.0 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem/Hollow Stem COMPILED BY CK
DATUM Geodetic DATE 21 January 1999 CHECKED BY ZSO

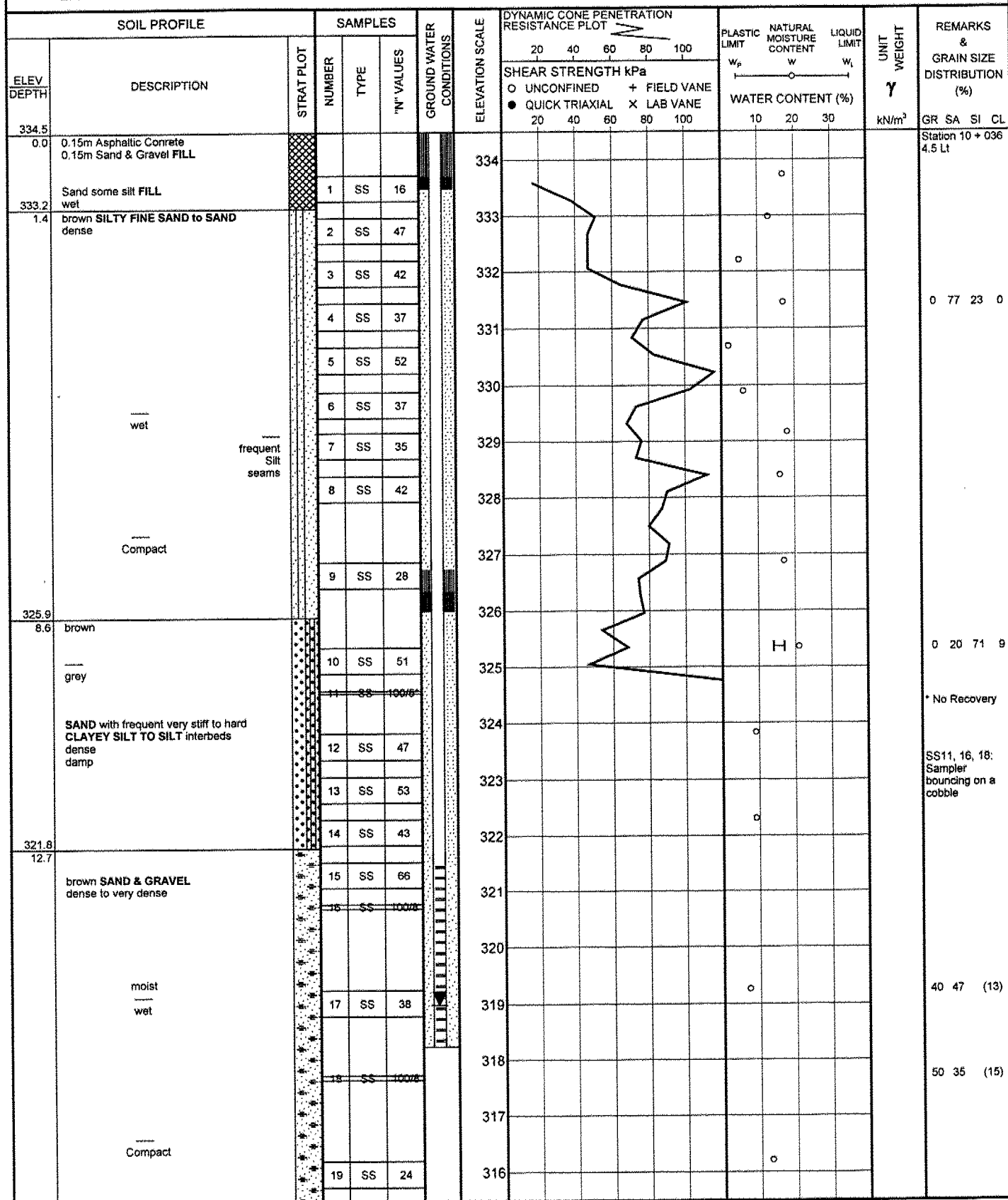
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w_p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w_L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
314.7			21	SS	77		315									15 79 (6)	
20.3	End of Borehole at 20.3 WL in Piezometer on completion: 7.1m (probably drilling mud) Feb.15/99: 7.9m (probably drilling mud)																

RECORD OF BOREHOLE No DL2

1 OF 2

METRIC

W.P. 471-93-01 LOCATION N 5044931.9 E 317776.1 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem COMPILED BY CK
DATUM Geodetic DATE 25 January 1999 CHECKED BY ZSO



RECORD OF BOREHOLE No DL2

2 OF 2

METRIC

W.P. 471-93-01 LOCATION N 5044931.9 E 317776.1 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem COMPILED BY CK
DATUM Geodetic DATE 25 January 1999 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
							20	40	60	80	100	W _p	W	W _L			
314.2			20	SS	25		315									30 57 (13)	
20.3	End of Borehole at 20.3m WL in Piezometer Feb. 8/99: 15.0m Feb. 15/99: 15.5m																

RECORD OF BOREHOLE No DL3

1 OF 1

METRIC

W.P. 471-93-01 LOCATION N 5044896.6 E317709.9 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem COMPILED BY CK
DATUM Geodetic DATE 26 January 1999 CHECKED BY ZSO

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 (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE x LAB VANE						
334.5	0.15m Asphaltic Concrete 0.15m Sandy Gravel FILL						20 40 60 80 100	20 40 60 80 100	10 20 30						
0.0	Fine Sand with silt FILL damp		1	SS	37									Station 9 + 961 3.7 Rt	
332.7	Some Organics, moist		2	SS	60									1 74 25 0	
1.8			3	SS	38										
			4	SS	41									0 88 (12)	
			5	SS	24										
	brown SILTY FINE SAND to SAND some silt, occasional silty layers compact to dense dry to damp		6	SS	15									0 85 (15)	
			7	SS	31										
			8	SS	35									0 83 (17)	
	SILTY		9	SS	36										
			10	SS	44										
	very dense		11	SS	58									0 58 42 0	
	Silt seams		12	SS	65									53 26 (21)	
323.3			13	SS	100/8										
11.2	SILTY		14	SS	50/8										
	SAND & GRAVEL trace silt occasional cobbles and boulders very dense		15	SS	105									40 53 (7)	
			16	SS	100/15										
	dense		17	SS	44										
318.8	End of Borehole at 15.7m WL in Piezometer on completion: none Feb. 8/99: none Feb. 15/99: piezometer plugged														

RECORD OF BOREHOLE No DL4

1 OF 1

METRIC

W.P. 471-93-01 LOCATION N 5044889.5 E 317690.0 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem COMPILED BY CK
DATUM Geodetic DATE 27 January 1999 CHECKED BY ZSO

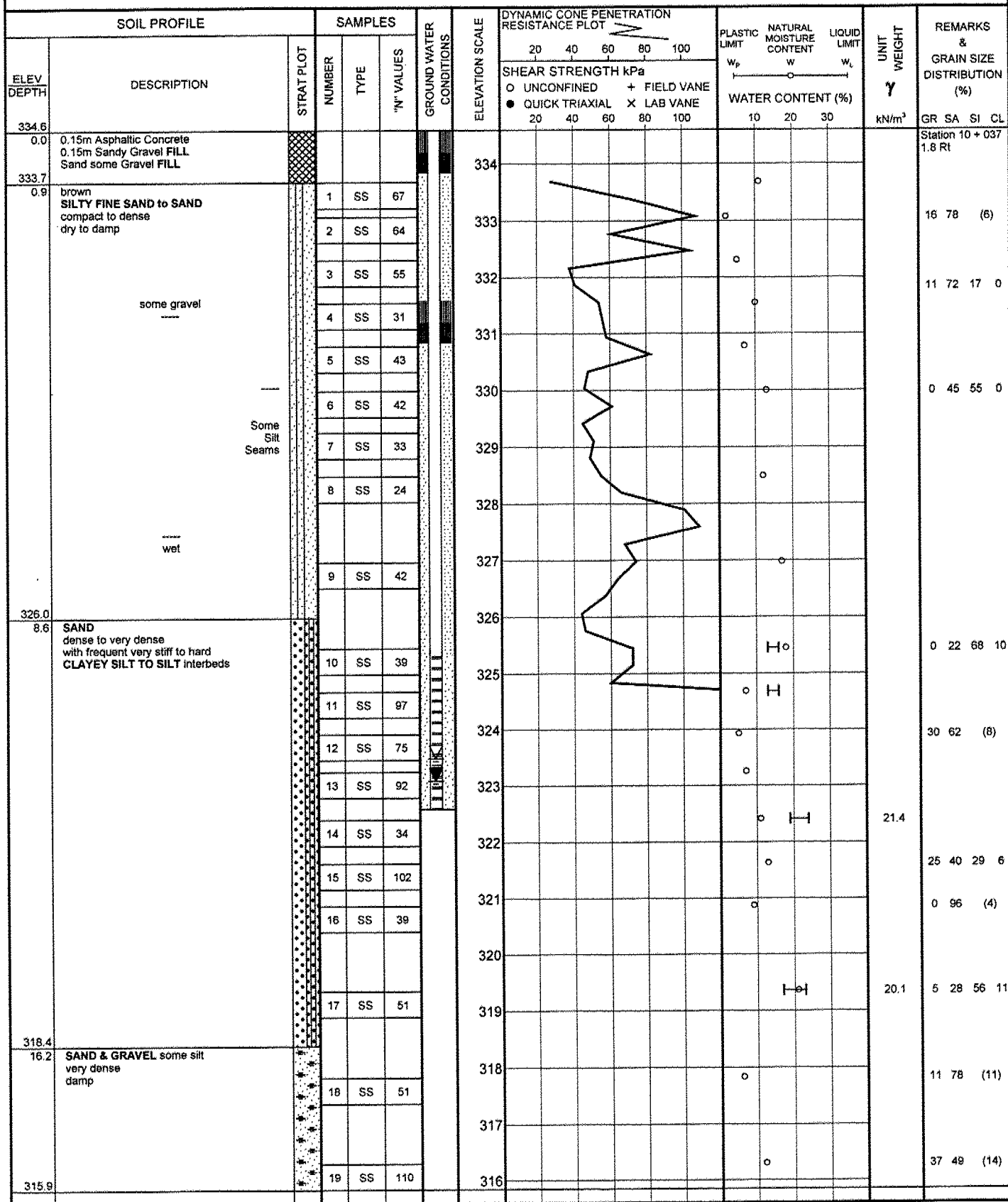
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W_p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W_L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40						60	80	100	20	40	60
334.4	0.15m Asphaltic Concrete 0.15m Sandy Gravel FILL						334													
333.4	Silty Sand FILL, moist		1	SS	23															
1.0	brown SILTY FINE SAND to SAND with some Sandy Silt zones compact to dense damp		2	SS	12		333													
			3	SS	36		332													
			4	SS	12		331													
			5	SS	18		330													
			6	SS	25		329													
			7	SS	30		328													
			8	SS	27		327													
326.3	End of Borehole at 8.1m WL at completion: none		9	SS	26															

RECORD OF BOREHOLE No DL5

1 OF 2

METRIC

W.P. 471-93-01 LOCATION N 5044926.7 E 317779.8 ORIGINATED BY AD
 DIST 52 HWY 11 BOREHOLE TYPE Solid Stem COMPILED BY CK
 DATUM Geodetic DATE 2 February 1999 CHECKED BY ZSO



Continued Next Page

+ 3, x 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No DL5

2 OF 2

METRIC

W.P. 471-93-01 LOCATION N 5044926.7 E 317779.8 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem COMPILED BY CK
DATUM Geodetic DATE 2 February 1999 CHECKED BY ZSO

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					
18.8	End of Borehole at 18.7m WL in piezometer on completion: 7.0m Feb. 8/99: 11.2m Feb. 15/99: 11.6m																

RECORD OF BOREHOLE No DL5A

1 OF 1

METRIC

W.P. 471-93-01 LOCATION N 5044928 E 317781 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem COMPILED BY CK
DATUM Geodetic DATE 6 March 1999 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE 20 40 60 80 100	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								
334.6 0.0													
	Auger to 8.4m												
325.2 8.4	SAND compact to very dense with frequent very stiff to hard CLAYEY SILT TO SILT interbeds damp		1	SS	22								
			2	SS	15								
			3	SS	25								
			4	SS	50/1*								
			5	SS	96								
			6	SS	50								
			7	SS	34								
			8	SS	40								
			9	SS	103*								
319.5 15.1	End of Borehole at 15.1m												

RECORD OF BOREHOLE No DL6

1 OF 1

METRIC

W.P. 471-93-01 LOCATION N 5044934.1 E 317797.1 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem COMPILED BY CK
DATUM Geodetic DATE 25 January 1999 CHECKED BY ZSO

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			20	40	60	80	100					
334.2	0.15m Asphaltic Concrete 0.15m Sandy Gravel FILL																
333.3	Sand FILL																
0.9	SILTY FINE SAND to SAND some gravel compact to dense dry to damp		1	SS	34												
			2	SS	27												
			3	SS	35												
			4	SS	32												
330.7	End of Borehole at 3.5m Water Level on completion: none																

RECORD OF BOREHOLE No DL7

1 OF 2

METRIC

W.P. 471-93-01 LOCATION N 5044921.4 E317750.0 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem/Hollow Stem COMPILED BY CK
DATUM Geodetic DATE 2 February 1999 CHECKED BY ZSO

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 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE						
335.0								20 40 60 80 100						GR SA SI CL
0.0	0.15m Asphaltic Concrete 0.15m Sandy Gravel FILL													Station 10 + 008 4.4 Lt
333.6	Sand FILL		1	SS	52									10 72 18 0
1.4	brown SILTY FINE SAND to SAND compact, damp		2	SS	18									
332.7														
2.3	Compact		3	SS	19									20 77 (3)
			4	SS	62									
			5	SS	60									
			6	SS	46									36 55 (9)
			7	SS	33									
			8	SS	50/6**									Solid Stem Hollow Stem ** SS8, SS12: No Recovery
	SAND to SAND & GRAVEL trace silt with frequent cobbles dense to very dense damp		9	SS	100/13									
		boulder	10	SS	100/10									Auger refusal on boulder @ 9.1m (Feb. 2/99)
			11	SS	91/18									Feb. 7/99 *
			12	SS	76/8**									
			13	SS	30									35 56 (9)
			14	SS	54									56 37 (7)
			15	SS	39									37 58 (5)
		fine Sand layers	16	SS	37									1 95 (4) * Refused to advance with tricone. Move borehole 0.6m North and advance with Hollow Stem Augers
			17	SS	100/10									47 46 (7)
		wet												
			18	SS	27									
	COMPACT													
			19	SS	61									0 88 (12)
316.3														
18.7	End of Borehole at 18.7m.													

Continued Next Page

+ 3, x 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No DL7

2 OF 2

METRIC

W.P. 471-93-01 LOCATION N 5044921.4 E317750.0 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem/Hollow Stem COMPILED BY CK
DATUM Geodetic DATE 2 February 1999 CHECKED BY ZSO

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 Y kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa												
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE					WATER CONTENT (%)									
						20	40	60	80	100	20	40	60	80	100	10	20	30		
	WL in hollow stem augers on completion: 16m WL in piezometer on completion: none Feb.8/99: none Feb.15/99: none																			

RECORD OF BOREHOLE No DL8

1 OF 1

METRIC

W.P. 471-93-01 LOCATION N 5044904.2 E 317705.9 ORIGINATED BY AD
 DIST 52 HWY 11 BOREHOLE TYPE Solid Stem COMPILED BY CK
 DATUM Geodetic DATE 5 February 1999 CHECKED BY ZSO

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								
334.6	0.15m Asphaltic Concrete		1	AS								Station 9 + 960 4.8 Lt
334.0	0.15m Sandy Gravel FILL											12 55 31 2
0.6	0.25m Silty Sand FILL											
	brown SILTY FINE SAND to SAND compact to dense damp to dry		2	SS	26							
			3	SS	27							
			4	SS	31							
	V. Silty		5	SS	24							0 51 47 2
			6	SS	10							
			7	SS	14							
			8	SS	18							0 95 (5)
			9	SS	21							
	frequent Silt seams		10	SS	20							0 52 44 4
			11	SS	29							
	moist		12	SS	34							0 67 33 0
			13	SS	40							
			14	SS	67/25							
322.6	brown, v. dense SANDY SILT, trace gravel & clay		15	SS	50/10							5 35 53 7
12.3	End of Borehole at 12.3m. Auger Refusal on Probable Boulder WL in piezometer on completion: none Feb. 8/99: none Feb. 15/99: none											

RECORD OF BOREHOLE No DL9

1 OF 2

METRIC

W.P. 471-93-01 LOCATION N 5044910 E 317744 ORIGINATED BY AD
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem COMPILED BY CK
 DATUM Geodetic DATE 5 March 1999 CHECKED BY ZSO

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						
							20 40 60 80 100	20 40 60 80 100	10 20 30					
335.0	0.15m ASPHALTIC CONCRETE		1	AS										Station 9 + 996 4.0 Rt 25 51 24 0
	Sand FILL with Gravel, Silt damp		2	SS	53									
332.5	SAND to SAND & GRAVEL trace silt		3	SS	18									
	Compact ----- dense to very dense		4	SS	75									
			5	SS	57									40 53 (7)
			6	SS	41									
			7	SS	38									
	Compact		8	SS	17									
325.2	End of Borehole at 9.8m													
9.8	Auger Refusal on Boulder													
	WL on completion: none													

Continued Next Page

+ 3, X 3. Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No DL9

2 OF 2

METRIC

W.P. 471-93-01 LOCATION N 5044910 E 317744 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem COMPILED BY CK
DATUM Geodetic DATE 5 March 1999 CHECKED BY ZSO

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 Y 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 (%)					
309.7													
25.3	End of DCPT at 25.3m												

+ 3, X 3. Numbers refer to
Sensitivity

O 3% STRAIN AT FAILURE

GEOCRES No. _____

DIST. 52 REGION _____W.P. No. 466-9300

CONT. No. _____

W. O. No. _____

STR. SITE No. _____

HWY. No. 11LOCATION Star Lake Rd.OverpassNo of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. _____

REMARKS: _____

**FOUNDATION INVESTIGATION AND DESIGN REPORT
FOR
PROPOSED STAR LAKE ROAD OVERPASS, NBL
STRUCTURE SITE NO. 44-392N
DISTRICT 52, HUNTSVILLE
W.P. 468-93-01**

Submitted To:

**DELCAN Corporation
133 Wynford Drive
North York, Ontario, M3C 1K1
Canada**

Submitted By:

**AGRA
104 Crockford Blvd.
Scarborough, Ontario, M1R 3C6
Canada**

**August 1999
TT98820B**

August 31, 1999.
Ref. No.: TT98820B

Delcan Corporation
133 Wynford Drive
North York, Ontario, M3C 1K1
Canada

Attention: Mr. Khaled El-Dalati, P. Eng.

Dear Sir:

**Re: FOUNDATION INVESTIGATION AND DESIGN REPORT
FOR
PROPOSED STAR LAKE ROAD OVERPASS, NBL
STRUCTURE SITE NO. 44-392N
DISTRICT 52, HUNTSVILLE
W.P. 468-93-01**

We take pleasure in enclosing six (6) copies of our Foundation Investigation and Design Report carried out for the above mentioned project and we will be glad to discuss any questions arising from this work.

Soil samples will be retained for a period of one year, and will thereafter be disposed of unless we are otherwise instructed.

We thank you for giving us this opportunity to be of service to you.

Sincerely,

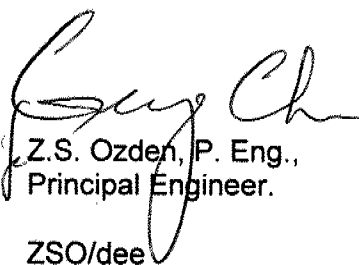

Z.S. Ozden, P. Eng.,
Principal Engineer.
ZSO/dee

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1.0 INTRODUCTION

AGRA, Consulting Geotechnical Engineers, was retained by Delcan Corporation to conduct a foundation investigation at the site of a proposed bridge that will carry the proposed realigned northbound lane of Highway 11 over the existing Star Lake Road. The site is located in the Village of Emsdale, about 0.3 km west of the intersection of Star Lake Road and present Highway 11, in the Township of Perry, Lot 14, Concession 11 in MTO District 52-Huntsville (see Key Plan, Drawing No. 1). The proposed bridge will be an approximately 21 m long, single span, 2-lane structure.

The purpose of the investigation has been to obtain information about the subsurface conditions at the site of the proposed bridge and approach embankments by means of exploratory boreholes, and based on the findings, to provide recommendations for the foundation design of the proposed structure and the approach fills.

2.0 SITE DESCRIPTION AND PHYSIOGRAPHY

The site is located about 0.3 km west of the intersection of Star Lake Road and the present Highway 11, in the Village of Emsdale. The ground elevation in the general area of the proposed bridge site falls to the north and the east, ranging in Elevation from about 345 to 332 m. The surrounding area is wooded with residential properties about 100± m to the west and TransCanada PipeLine about 150± m further to the west.

Based on available geologic information, the site is in an area of ice-contact sediments. Generally after the last glacial withdrawal, ice-contact sediments (sands and gravels) followed by glaciofluvial sediments (ranging from deltaic and nearshore sands and gravels to prodeltaic and lake bottom silts and clays) were deposited on top of the existing sandy glacial till or Precambrian bedrock. The area was then inundated by glacial Lake Algonquin, depositing sands, silts and clays in low lying areas.

The bedrock generally consists of strongly foliated gneissic to migmatic rocks of the Central Gneiss Belt, which is part of the Grenville Province (a structural subdivision of the Canadian Shield).

3.0 INVESTIGATION PROCEDURE

The fieldwork for this project was performed during the periods of January 28 to 29, and February 18 to 25, 1999, and consisted of drilling and sampling seven boreholes (Borehole Nos. SL1 through 7) and performing four dynamic cone penetration tests. The plan locations of the boreholes, along with stratigraphic sections are shown on Drawing No. 2.

The boreholes were advanced using solid and hollow stem continuous flight augers with a track-mounted power auger drilling rig (CME 75) owned and operated by Canadian Soil Drilling Inc. and a track-mounted power auger drilling rig (BOA 6M) owned and operated by Groundworks Drilling

.../...

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Inc., under the full-time supervision of a soils engineer from AGRA.

Sampling in the boreholes was effected at frequent intervals of depth by the Standard Penetration Test Method (SPT), as specified in ASTM Method D 1586. This consists of freely dropping a 63.5 kg hammer a vertical distance of 0.76 m to drive a 51 mm diameter o.d. split barrel (split-spoon) sampler into the ground. The number of blows of the hammer to drive the sampler into the relatively undisturbed ground by a vertical distance of 0.30 m is recorded as the Standard Penetration Resistance or the 'N'-value of the soil and this gives an indication of the consistency or the compactness condition of the soil deposit.

In addition, dynamic cone penetration tests were performed adjacent to four of the boreholes. This test consists of driving a 60° point, 50 mm diameter cone attached to the drill rod continuously, into the undisturbed ground with a driving energy of 475 J (63.5 kg hammer falling freely a distance of 76 cm) per blow. The number of blows for each 30 cm of penetration is recorded and this provides an indication of the relative changes in the soil density with depth.

Due to the presence of boulders above the bedrock surface, Boreholes SL2 and 4 were cored through boulders in the overburden utilizing NW size casing to the bedrock surface and the bedrock was subsequently cored using a NXL size core-barrel.

The borehole locations were established in the field by our engineering staff, in relation to the already staked out centre-line of Highway 11 (by Dearden and Stanton Limited). The borehole geodetic elevations and co-ordinates were later taken by surveyors from Dearden and Stanton Limited.

The soil samples were shipped in sealed containers to our geotechnical laboratory in Toronto (Scarborough) for further examination and classification. A laboratory testing programme, consisting of natural moisture content and grain-size analyses, was performed on selected representative soil samples. The results of the laboratory tests are presented on the appropriate Borehole Log Sheets and also in Figure Nos. 1 to 5.

The boreholes were left open until the end of each work day to enable us to take additional water level readings. Standpipe piezometers were installed in Boreholes SL1, 2 and 5 to monitor the groundwater level over a prolonged period of time without interference from surface water. The remaining boreholes were grouted on completion of the fieldwork, while the piezometer tubes were grouted on February 29, 1999.

4.0 SUBSURFACE CONDITIONS

The subsurface conditions were explored at seven borehole locations (Borehole Nos. SL1, 2, 3, 4, 5, 6, and 7), and were inferred at the locations of four dynamic cone penetration tests. The locations of the boreholes and cone penetration tests are shown on the Plan and Profile Drawing No. 2 and are also indicated on the individual Borehole Log Sheets. Cross sections of inferred subsurface stratigraphy are given on Drawing No. 2.

The ground surface at the proposed site falls to the north and east. The ground elevation at the proposed bridge location generally ranges from about 338 to 334 m.

In general, the boreholes have shown beneath a surficial topsoil the presence of cohesionless (i.e. granular) sand overburden to a depth of about 21 m. The overburden consists of generally fine or fine to medium sand with some (coarser) sand to sand & gravel zones. A bouldery sand & gravel layer was also encountered immediately overlying the bedrock. The Precambrian diorite bedrock was encountered at a depth of about $21 \pm$ m (approximately Elevation 317 to 312 m). The groundwater table at the time of our investigation was encountered at depths of about 15 to 19 m below existing grade.

Details of the subsurface conditions encountered in the boreholes are presented on the Borehole Log Sheets. The following paragraphs are only meant to complement and summarize these data.

4.1 TOPSOIL

Topsoil was encountered at the majority of boreholes (except Boreholes SL3, 4 and 7), ranging in thickness from 0.05 to 0.2 m.

In our experience the thickness of topsoil frequently varies in between and beyond the borehole locations. In addition, at the time of our investigation the ground near the surface was frozen; therefore the soil conditions within the upper several decimeters could not be accurately determined and the descriptions given for this upper zone should be considered approximate only.

4.2 SILTY SAND

Below the surficial topsoil at Borehole SL5 and at the surface at Borehole SL7, a silty sand deposit was encountered to depths of 1.5 and 3.3 m (or Elevation 333.3 and 329.5 m), respectively. This cohesionless deposit contains occasional clayey silt seams (in Borehole SL7) and traces of gravel (in Borehole SL5). Measured 'N'-values within this deposit range from 15 to 58 blows/0.3 m, indicating a compact to very dense condition. Measured natural moisture contents range from 11 to 17%.

Results of the dynamic cone penetration tests range from 6 to 27 blows/0.3 m.

4.3 UPPER SAND TO GRAVEL & SAND

Underlying the surficial topsoil in Boreholes SL1 and SL2 (i.e. near the south abutment location) and below 1.4 and 3.3 m depths at Boreholes SL6 and SL7, a sand to sand & gravel deposit was contacted. This unit was found to be 3.6 to 5.6 m thick and extended to depths of between 3.8 m (Borehole SL2) and 8.9 m (Borehole SL7) below the ground surface or to Elevations 334.4 and 323.9 m, respectively. Eight grain size analyses were conducted on samples from this granular (cohesionless) deposit, resulting in the following grain size measurements.

Gravel:	17 - 55%
Sand:	42 - 81%
Silt and Clay:	0 - 7%

The grain size analyses results are presented in envelope form in Figure No. 1.

With the exception of two low values of 4 and 10 blows/0.3 m within 0.6 m of the ground surface, the measured 'N'-values within this deposit generally range from 27 to 76 blows/0.3 m, indicating a compact to very dense condition. The results of dynamic cone penetration tests in this deposit range from 19 to in excess of 100 blows/0.3 m, with lower values (3 to 5 blows/0.3 m) within the top 1 m \pm .

Measured natural moisture contents range from 2 to 9%.

4.4 SAND

Underlying the surficial soils described in the preceding sections, all boreholes contacted a major deposit of fine or fine to medium sand to depths of 1.4 (Borehole SL6) to 19.3 m (Borehole SL4) below existing grade or Elevation 339.5 m to 314.3 m. The grain size distribution of seventeen samples from this cohesionless (granular) deposit are presented in envelope form in Figure No. 2. These indicate 0-5% gravel, 84-99% sand and 1-16% soil fines (i.e. silt & clay) size particles.

Measured 'N'-values within the deposit range from 8 to 61 blows/0.3 m indicating loose to very dense conditions, but generally compact to dense. The results of dynamic cone penetration tests in this deposit range from 17 to in excess of 100 blows/0.3 m.

Measured natural moisture contents range from 1 to 10%.

4.5 GRAVELLY SAND

Interbedded within the fine sand deposit in Borehole SL2, is a gravelly sand layer extending from about 7.6 m (Elevation 330.6 m) to about 12.2 m (Elevation 326.0 m) below existing grade. A grain size distribution analysis was conducted on a sample from this granular deposit and the resulting curve is presented in Figure No. 3. The results indicate 34% gravel, 62% sand and 4% silt & clay size particles. Measured 'N'-values within this deposit range from 25 to 39 blows/0.3 m, indicating

a compact to dense condition. The measured natural moisture contents range from 1 to 2%.

4.6 LOWER SILTY SAND TO SANDY SILT

Near the south abutment location Boreholes SL1 and 2, contacted, underlying the fine to medium sand, a somewhat finer silty fine sand to fine sandy silt deposit at depths of 16.6 m (Elevation 321.1 m) and 16.2 m (Elevation 322.0 m), respectively. This basically cohesionless deposit was found to be 1.4 to 3.4 m thick and extended to depths of 18.0 m (Elevation 319.7 m) and 19.6 m (Elevation 318.6 m), respectively. Grain size distribution analyses were conducted on two samples and the range of particle sizes are presented in Figure No. 4. The analyses indicate the following particle distribution range.

Gravel:	0 %
Sand:	44 - 67%
Silt & Clay:	33 - 56%

The measured natural moisture contents range from 8 to 24%.

'N'-values recorded in this deposit range from 17 to 61 blows/0.3 m indicating a compact to very dense condition.

4.7 LOWER SAND & GRAVEL

Boreholes SL1 and SL2, drilled near the south abutment location, encountered, immediately above the bedrock, a sand & gravel deposit containing frequent cobbles and boulders. This bouldery deposit was contacted below depths of 18.0 m (Elevation 319.7 m) and 19.6 m (Elevation 318.6 m) at Boreholes SL1 and SL2, respectively and extended to the surface of the bedrock at depths of 21.3 m (Elevation 316.4 m) and 21.2 m (Elevation 317.0 m), respectively. The presence of the deposit was also inferred at the north abutment location at Borehole SL4, below a depth of 19.3 m or Elevation 314.3 m.

A grain size distribution analysis of a sample recovered from Borehole SL1 was carried out and, as shown in Figure No. 5, this indicated 42% gravel, 43% sand and 15% silt & clay size particles. It should however be pointed out that in Boreholes SL2 and SL4, coring had to be resorted in order to advance the boreholes through frequent cobbles and boulders.

Measured 'N'-values in this deposit range from 75 to in excess of 100 blows/0.3 m indicating a generally very dense condition although some of the measured values may be unreliable due to the presence of oversized materials.

Measured natural moisture contents range from 10 to 16%.

4.8 BEDROCK

Bedrock was encountered and cored in Boreholes SL1, 2 and 4 at depths of 21.3 m (Elevation 316.4 m), 21.2 m (Elevation 317.0 m) and 21.0 m (Elevation 312.6 m) below existing ground surface, respectively. These boreholes were advanced 3.1 to 4.8 m into the rock. The recovered core samples show that the Precambrian bedrock consists of a massive, moderately closely jointed, slightly metamorphosed diorite. In Boreholes SL1 and SL4 the percentage of core recovery was 97 to 100%, while in Borehole SL2 it was 45 to 100%. Rock quality designation (R.Q.D.) values of 80 to 100% were measured in Boreholes SL1 and SL4, with lower values in Borehole SL2 (0 to 83%). Based on these values together with a visual examination of the rock cores, the rock is considered to be of excellent quality in Boreholes SL1 and SL4, and generally of poor to good quality in Borehole SL2.

From the results of Boreholes SL1, 2 and 4, drilled for this investigation, and Boreholes SL12 and 14 for the proposed south-bound twin bridge about 30 m to the west, it can be surmized that the bedrock surface generally dips in a north easterly direction (i.e. from a high Elevation of 322.1 m at Borehole SL 14 to a low of 312.6 m at SL 4), more or less following the existing ground surface contours.

4.9 GROUNDWATER CONDITIONS

Groundwater levels in the open boreholes were observed during the drilling and at the completion of each borehole. To enable us to measure water levels at the site over a prolonged period of time without interference from surface water, standpipe piezometers were installed in Boreholes SL1, 2 and 5.

The recorded values, are shown on the individual Borehole Log Sheets. Based on the recorded values in the piezometers installed in Boreholes SL2 and 5 and moisture contents of samples in Boreholes SL1 and 4, the groundwater levels at the time of the investigation generally ranged from 15 to 19 ± m below the ground surface (Elevation 321 to 318 m). It should, however, be pointed out that the groundwater at the site would fluctuate seasonally and can be expected to be somewhat higher during the spring months and in response to heavy rains.

5.0 DISCUSSION AND RECOMMENDATIONS

The proposed Highway 11 realignment will consist of a four lane divided highway with an approximately 30 m wide median. The proposed bridge will carry the proposed northbound lane of Highway 11 over Star Lake Road and will be an approximately 21 m long, single span, 2-lane (13 m wide) structure. The grade at the bridge site falls to the north and to the east. In general, the existing ground elevation along the bridge alignment is 338 to 334 m. The proposed grade of Highway 11 at the bridge site is approximately Elevation 345 to 345.5 m. The existing grades at the bridge will therefore be raised by about 8 to 11 m at the south and north abutment locations, respectively.

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Due to the presence of overhead utility cables along the north side of Star Lake Road, and uneven sloping ground along the road cut on the south side of Star Lake Road, the exact proposed abutment locations were not accessible. The boreholes were therefore drilled somewhat offset from the actual proposed foundation elements, as close as practicable.

In general, the boreholes have shown beneath a surficial topsoil the presence of cohesionless, granular deposits of compact to very dense sand. These sand deposits generally consist of fine or fine to medium sand with some coarser sand to sand & gravel zones and occasional silty fine sand to sandy silt layers. A sand & gravel layer, containing frequent cobbles and boulders, immediately overlies the bedrock. The Precambrian diorite bedrock was encountered at a depth of about $21 \pm$ m (approximately Elevations 317 to 312 m). The bedrock was cored in Boreholes SL1, 2 and 4 to a depth of 3.1 to 4.8 m and within this zone it was found to be of excellent quality in Boreholes SL1 and 4, with a relatively less competent condition at Borehole SL2. The groundwater table at the time of our investigation was encountered at depths of 15 to 19 m below existing grades.

5.1 FOUNDATIONS

It is our understanding that the preferred foundations for the abutments of the proposed bridge are of the "integral" type and will be supported on driven steel H-piles, if possible. In our opinion, the subsurface conditions are suitable for the use of integral abutments.

The boreholes show that for the prevailing subsurface conditions the use of a low displacement pile, such as a steel H-pile with a heavy section, such as HP310X110 with reinforced tips as per MTO Specifications, would be better suited than other pile types (i.e. steel tube piles, steel H-piles with a light section).

The piles would preferably be driven to the surface of the bedrock where uniformly high resistances can be utilized. In Boreholes SL1 and SL2, drilled near the proposed south abutment location, the surface of the bedrock was contacted at Elevations 316.4 and 317.0 m, respectively (i.e. fairly level) and in Borehole SL4, drilled near the north abutment location, at Elevation 312.6 m. Experience in the general area however shows that in many cases the surface of the bedrock can frequently be uneven and unpredictable. In addition, in two of these three boreholes (i.e. SL2 and SL4) frequent cobbles and boulders were encountered in the sand & gravel deposit immediately overlying the bedrock, and the boreholes had to be extended by rock coring methods to reach the bedrock. Because of this it is likely that many of the piles may not reach the surface of the bedrock and will likely terminate in the bouldery sand & gravel overburden above the bedrock surface.

The following table summarizes, based on the borehole results, the estimated approximate average pile tip elevations that may be assumed for design purposes.

TABLE 1

SUPPORT LOCATION	REFERENCE BOREHOLE	BEDROCK SURFACE ELEVATION (m)	ESTIMATED PILE TIP ELEVATION (m)	ESTIMATED APPROXIMATE AVERAGE PILE TIP ELEVATION (m)
South Abutment	SL1 SL2	316.4 317.0	317.5 ± 318 ±	317.5±
North Abutment	SL4 SL5*	312.6	313.5 ± 314 ±	313.5±

* based on dynamic cone penetration test conducted adjacent to borehole

It should also be pointed out that in general the pile tip elevations can be expected somewhat higher on the west side of each abutment and somewhat lower towards the east, following the inferred bedrock surface contours.

For piles driven to practical refusal within the very dense overburden at or below elevations shown in the above table the following axial resistances may be utilized for HP310X110 steel H-piles.

Factored axial resistance at Ultimate Limit States (U.L.S.) = 1,600 kN
Geotechnical resistance at Serviceability Limit States (S.L.S.) = 1,100 kN

These values were conservatively selected in view of the fact that some premature refusals may be encountered due to the presence of cobbles and boulders. It is also possible that due to undulations in the surface of the bedrock and the overburden soils immediately overlying it, which are not uncommon in Northern Ontario, the piles may drive several meters below the tip elevations given above. We recommend that this aspect be taken into consideration when ordering the piles.

The piles should be driven with a suitably heavy hammer capable of delivering a rated capacity of at least 50 kJ/blow. The energy should however be restricted to not more than 60 kJ/blow.

The driving of the piles should be controlled by a recognized pile driving formula, such as the Hiley Formula. The estimated ultimate resistance of the piles (driven to practical refusal in the overburden) by the Hiley Formula is approximately 3200 kN. This value was arrived utilizing a resistance factor of 0.5 (i.e. ULS value of 1600 kN divided by 0.5 = 3200 kN) as per MTO current practice. Because of the presence of occasional cobbles or boulders in the overburden, the bouldery layer immediately above the surface of the bedrock, and the anticipated hard driving conditions, as mentioned before, the piles should be equipped with reinforced tips as per MTO Standards (OPSD 3301.00). In this respect, it may be worth mentioning that premature refusal to augering was encountered (probably due to a boulder) in one of the boreholes drilled for the twin, south bound bridge (i.e. Borehole SL13).

Oversize materials (e.g. greater than 75 mm nominal diameter) should not be used in the fills through which piles would be driven, or within the area of the proposed future widening at the north and south abutments.

In cohesionless soils the coefficient of horizontal subgrade reaction may be estimated from;

$$k_s = n_h z/d$$

where k_s = coefficient of horizontal subgrade reaction
 z = depth
 d = pile width
 n_h = coefficient related to soil density as given in the table below

Also, presented in the same table are the estimated values for angle of internal friction and bulk unit weights.

TABLE 2

AREA/REFERENCE BOREHOLE NO.	APPLICABLE DEPTH FROM EXISTING GROUND SURFACE	SOIL TYPE	BULK UNIT WEIGHT (kN/m ³)	ANGLE OF INTERNAL FRICTION (ϕ) DEGREES	RECOMMENDED n_h VALUE (MN/m ³)
North Abutment					
SL3	0 - 1 m	loose to compact sand	18	29	3.0
	1 - 12 m	compact to dense sand	20	32	9.0
SL4	0 - 11 m	compact sand	20	32	7.0
	11 - 13.5 m	dense sand	20	33	9.0
	13.5 - 15.5 m	v. dense sand	20	33	15.0
	15.5 - 19 m	compact sand	20	30	7.0
SL5	0 - 1.5 m	compact silty sand	19	29	7.0
	1.5 - 13 m	compact to dense sand	20	32	9.0

TABLE 2 (continued)

AREA/REFERENCE BOREHOLE NO.	APPLICABLE DEPTH FROM EXISTING GROUND SURFACE	SOIL TYPE	BULK UNIT WEIGHT (kN/m ³)	ANGLE OF INTERNAL FRICTION (ϕ) DEGREES	RECOMMENDED n_h VALUE (MN/m ³)
South Abutment					
SL1	0 - 1 m	v.loose sand	18	28	2.0
	1 - 4.5 m	dense to v.dense sand to sand & gravel	21	34	18.0
	4.5 - 16.5 m	compact to dense sand	20	32	9.0
	16.5 - 18 m	v.dense silty sand /sandy silt	20	31	11.0
SL2	0 - 1 m	loose sand	18	28	2.5
	1 - 4 m	compact to v.dense sand to sand & gravel	20	32	15.0
	4 - 8 m	compact sand	20	30	7.0
	8 - 12 m	compact to dense gravelly sand	20	32	9.0
	12 - 16 m	compact to dense sand	20	32	9.0
	16 - 19 m	compact to dense silty sand/sandy silt	20	30	9.0

The recommended horizontal resistances for the HP310X110 steel H-piles are as follows:

$$\begin{aligned} \text{Factored Horizontal Resistance at U.L.S.} &= 130\text{kN} \\ \text{Horizontal Resistance at S.L.S} &= 60\text{ kN} \end{aligned}$$

In order to minimize the effect of any downdrag we recommend that the approach embankment fills be placed to their final grade elevation at least three weeks prior to driving the piles.

In accordance with MTO requirements (MTO Structural Office Standard), piles for integral abutments require a 3 m long flex zone. In essence the current MTO standard for the flex zone consists of an annular space in between two consecutive CSP's. One of the CSP's surrounds the H-pile (i.e. has a diameter of about 600 mm surrounding the pile, while the second CSP has a somewhat larger diameter; typically 800 mm for a 310 mm H-pile). The annular space in between the CSP's is the 3 m long flex zone. After the pile is driven, the space between the H-pile and the inner CSP is filled with coarse sand. An NSSP should be included in the contract documents detailing the gradation of the sand backfill as follows.

<u>Sieve Size</u>	<u>Percentage Passing</u>
2 mm	100%
600 μ m	80 - 100%
425 μ m	40 - 80%
250 μ m	4 - 25%
150 μ m	0 - 6%

Depending on the details of the proposed structure (i.e. foundation width, elevation, etc.) spread footing foundations on engineered fill may be feasible at or above Elevation 340 m at the south abutment and 337 m at the north abutment, after stripping all the weak surficial soils. The settlements may however be somewhat in excess of the normally accepted value of 25 mm (e.g. 'N'-values of between 12 and 16 blows/0.3 m were recorded between Elevations 334 and 331 m. in Borehole SL2). In view of this and the requirement of integral abutments, the use of normal spread footing foundations is not recommended. If however, it is necessary to consider normal spread footing foundations we will be pleased to further look into this aspect.

5.2 LATERAL EARTH PRESSURES

Backfill behind abutments and retaining walls should consist of non-frost susceptible, free draining granular materials in accordance with the Ontario Ministry of Transportation Standards.

Free-draining backfill materials (i.e. Granular 'A' or Granular 'B') and the provision of drain pipes and weep holes, etc., should prevent hydrostatic pressure build-up. Computation of earth pressures should be in accordance with O.H.B.D.C. For design purposes, the following parameters (unfactored) can be used.

Compacted Granular 'A'

Unit Weight = 22 kN/m³

Coefficient of Lateral Earth Pressures:

$$K_a = 0.27$$

$$K_o = 0.43$$

Compacted Granular 'B'

Unit Weight = 21 kN/m^3

Coefficient of Lateral Earth Pressures:

$$K_a = 0.31$$

$$K_o = 0.47$$

These values are based on the assumption that the backfill behind the retaining structure is free-draining and adequate drainage is provided. As well, it is assumed that the ground behind the retaining structure is level.

The earth pressure coefficient adopted will depend on whether the retaining structure is restrained or movements can be allowed such that the active state of earth pressure can develop. If the abutment is restrained and does not allow lateral yielding, then at rest pressures should be used as per Clause C6-7.1 of the O.H.B.D.C., 3rd Edition. The effect of compaction should also be taken into account in the selection of the appropriate earth pressure coefficients in accordance with Clause 6-7.4.3 of the O.H.B.D.C., 3rd Edition.

Vibratory equipment for use behind abutments and retaining walls should be restricted in size as per current MTO practice.

As an alternative to conventional retaining walls, MTO's Retained Soil System may be used. The following should be included in the Contract Documents:

- identify longitudinal extent in plan of the Retained Soil System.
- identify in plan transverse space constraints (top of wall and bottom of wall)
- identify elevation of top of wall and bottom of wall
- include NSSP for Retained Soil Systems in Contract Documents

The Retained Soil System should be of high performance and moderate to high appearance.

5.3 APPROACH EMBANKMENTS

At the south abutment location the proposed bridge deck level is about Elevation 345.5 m, and the existing grade elevation is about 337.5 m and therefore the grade will be raised by about 8 m. At the north abutment location, on the other hand, while the proposed grade elevation is about 345.0 m, the existing grade elevation is about 334 m and thus the grade here will be raised by about 11 m at the immediate abutment location, with the height of the approach embankment increasing to a maximum of 12.5 m further north.

Based on the borehole results, the strength of the foundation materials is such that deep-seated failures are not anticipated, provided all organic soils, weak or otherwise unsuitable materials are removed as per MTO Standards before placing the fill.

.../...

M:\reports\1999\starlakenbl.wpd

Assuming properly compacted, acceptable inorganic earth fill material, 2 horizontal in 1 vertical side slopes can be used but for embankment heights of greater than 6 m, a 2 m wide mid-height berm should be provided to satisfy current requirements by MTO. The berm gradient should be sloped (say 1V:20H) to drain away from the embankment. Proper erosion control measures should be implemented both during the construction and permanently. This can be achieved by immediate seeding or sodding (OPSS 572).

All organic and other unsuitable soils should be removed within an envelope given by an imaginary slope not steeper than 1:1 from the toe of the proposed embankment as depicted by the sketch presented in Appendix B. The average thickness of the unsuitable soils to be stripped can be assumed to be about 0.2 m. After stripping, the exposed subgrade should be inspected, approved and properly compacted from the surface under the supervision of a geotechnical engineer who is familiar with the findings of this report and appointed by the Contract Administrator.

Provided that all organic and otherwise unsuitable materials are removed and the subgrade is properly compacted from the surface as detailed above, the settlement of the foundation materials (i.e. not including the settlement of the embankment material under its own weight) should not exceed 40 mm on the south side and 75 mm on the north side and should be substantially completed during the construction and within three weeks of placing the embankment fill to its full height. Such settlements are considered acceptable and will not necessitate preloading or surcharging.

Water level measurements indicate water levels at about 15 to 19 m below existing grade and, therefore, we do not anticipate major problems due to groundwater seepage during stripping of the subgrade and backfilling for the construction of the embankments.

The materials used for the construction of the embankment fills should consist of approved, clean earth fill (e.g. Select Subgrade Materials - OPSS 1010). The majority of the clean, inorganic in-situ materials (i.e. sands) are considered to be suitable for this purpose. The fills should be placed in lifts not exceeding 300 mm before compaction and each lift should be uniformly compacted to at least 95% of the material's Standard Proctor Maximum Dry Density. The degree of compaction within the top 0.5 m of the fill (i.e. the subgrade immediately beneath the granular sub-base) should be increased to 98%. The selection, placement and compaction of the fill should be carried out under the supervision of a geotechnical engineer who is familiar with the findings of this report and appointed by the Contract Administrator. The settlement of the embankment fills prepared as described above should not exceed 50 and 80 mm, respectively for the south and north approach embankments. The time rate of settlement of the fill making up the embankment will depend on the material used for construction and for granular fills it should substantially be completed during the construction and within a few weeks thereafter (i.e. should be essentially elastic). Clayey fills can on the other hand, be expected to consolidate over a longer period of time. It should also be pointed out that these quoted settlements will be in addition to the foundation settlements of 40 and 75 mm, quoted earlier.

For embankment construction rockfill can also be used, if available. Side slopes of 1 1/4H:1V can be maintained for embankments constructed from rockfill. In conformance with MTO Northern Region Practice a 2 m wide mid-height berm should be provided for fill heights greater than 6 m. Rockfill should not be used in the area of driven piles because this will interfere with the installation of piles. This would also apply to the area of the proposed future widening.

5.4 CONSTRUCTION COMMENTS

Water level measurements in the boreholes indicate groundwater levels approximately between Elevations 318 and 321 m, at depths of about 15 to 19 m below existing grade. No major problems due to groundwater seepage are therefore foreseen in excavations. Any surface water seepage, if necessary, can easily be handled by gravity drainage and pumping from open sumps.

5.5 FROST PROTECTION

Design frost penetration for the general area is 1.8 m. Therefore, a permanent soil cover of 1.8 m or its thermal equivalent is required for frost protection of foundations.

6.0 CLOSURE

We recommend that once the details of the structure are finalized, our recommendations should be reviewed for their specific applicability.

The Limitations of Report, as quoted in Appendix A, is an integral part of this report.

Sincerely,



Andrew Drevininkas, P. Eng.

AD/dee



Z. S. Ozden, P. Eng.

APPENDIX A

AGRA

LIMITATIONS OF REPORT

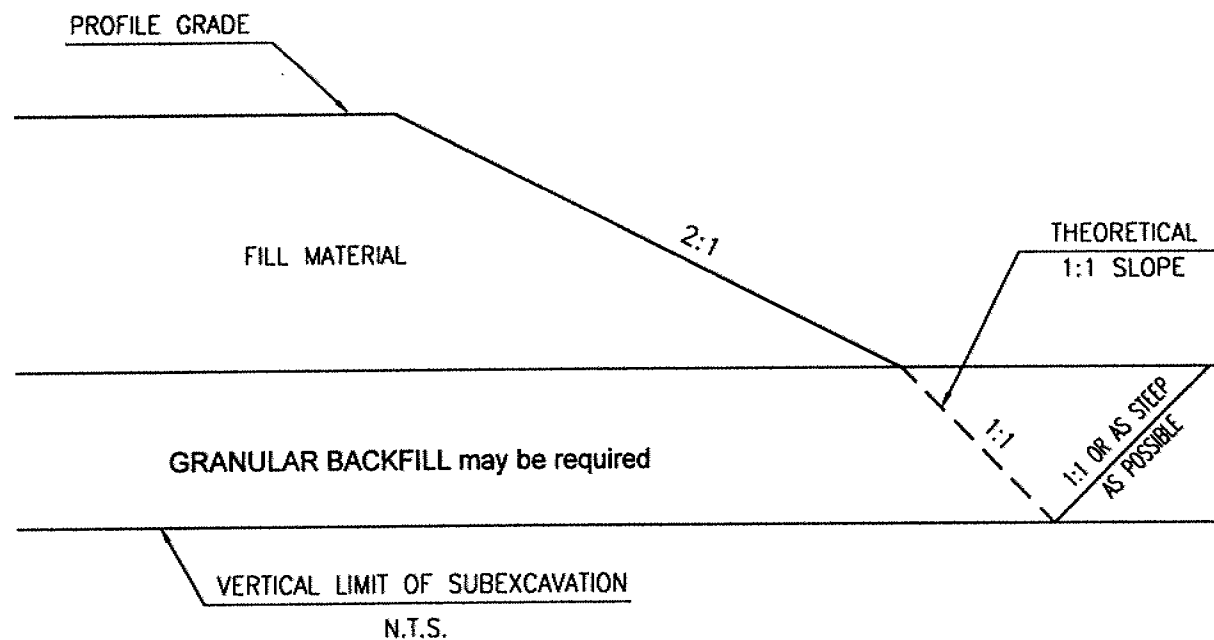
The conclusions and recommendations given in this report are based on information determined at the testhole locations. The information contained herein in no way reflects on the environmental aspects of the project, unless otherwise stated. Subsurface and groundwater conditions between and beyond the testholes may differ from those encountered at the testhole locations, and conditions may become apparent during construction, which could not be detected or anticipated at the time of the site investigation. It is recommended practice that the Geotechnical Engineer be retained during construction to confirm that the subsurface conditions throughout the site do not deviate materially from those encountered in testholes. The benchmark and elevations used in this report are primarily to establish relative elevation differences between the testhole locations and should not be used for other purposes, such as grading, excavating, planning, development, etc.

The design recommendations given in this report are applicable only to the project described in the text and then only if constructed substantially in accordance with the details stated in this report. Since all details of the design may not be known, we recommend that we be retained during the final design stage to verify that the design is consistent with our recommendations, and that assumptions made in our analysis are valid.

The comments made in this report on potential construction problems and possible methods are intended only for the guidance of the designer. The number of testholes may not be sufficient to determine all the factors that may affect construction methods and costs. For example, the thickness of surficial topsoil or fill layers may vary markedly and unpredictably. The contractors bidding on this project or undertaking the construction should, therefore, make their own interpretation of the factual information presented and draw their own conclusions as to how the subsurface conditions may affect their work. This work has been undertaken in accordance with normally accepted geotechnical engineering practices. No other warranty is expressed or implied.

Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. AGRA accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

APPENDIX B



REMOVAL OF UNSUITABLE SOILS
FROM BENEATH APPROACH FILLS
N.T.S.

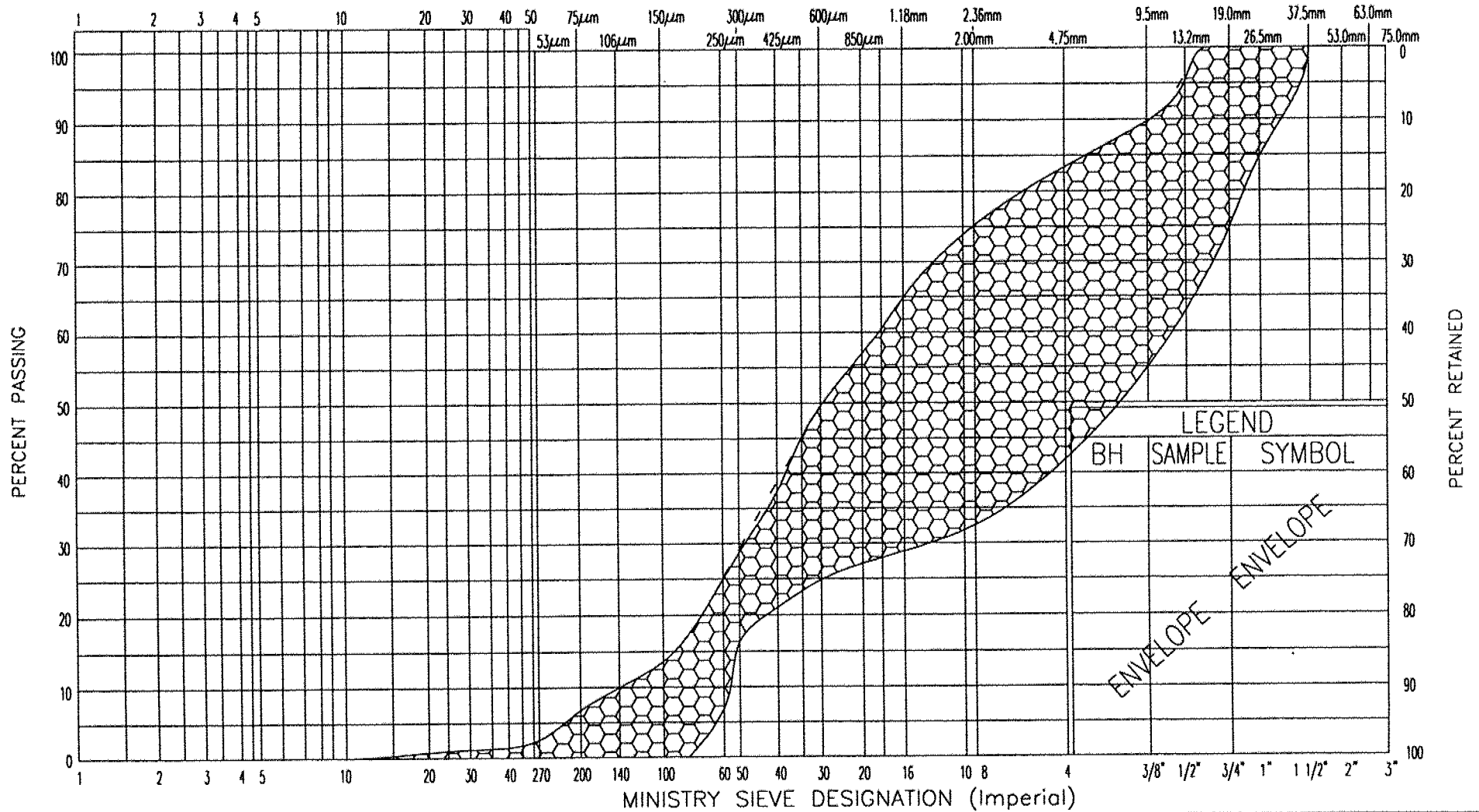
FIGURES

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)

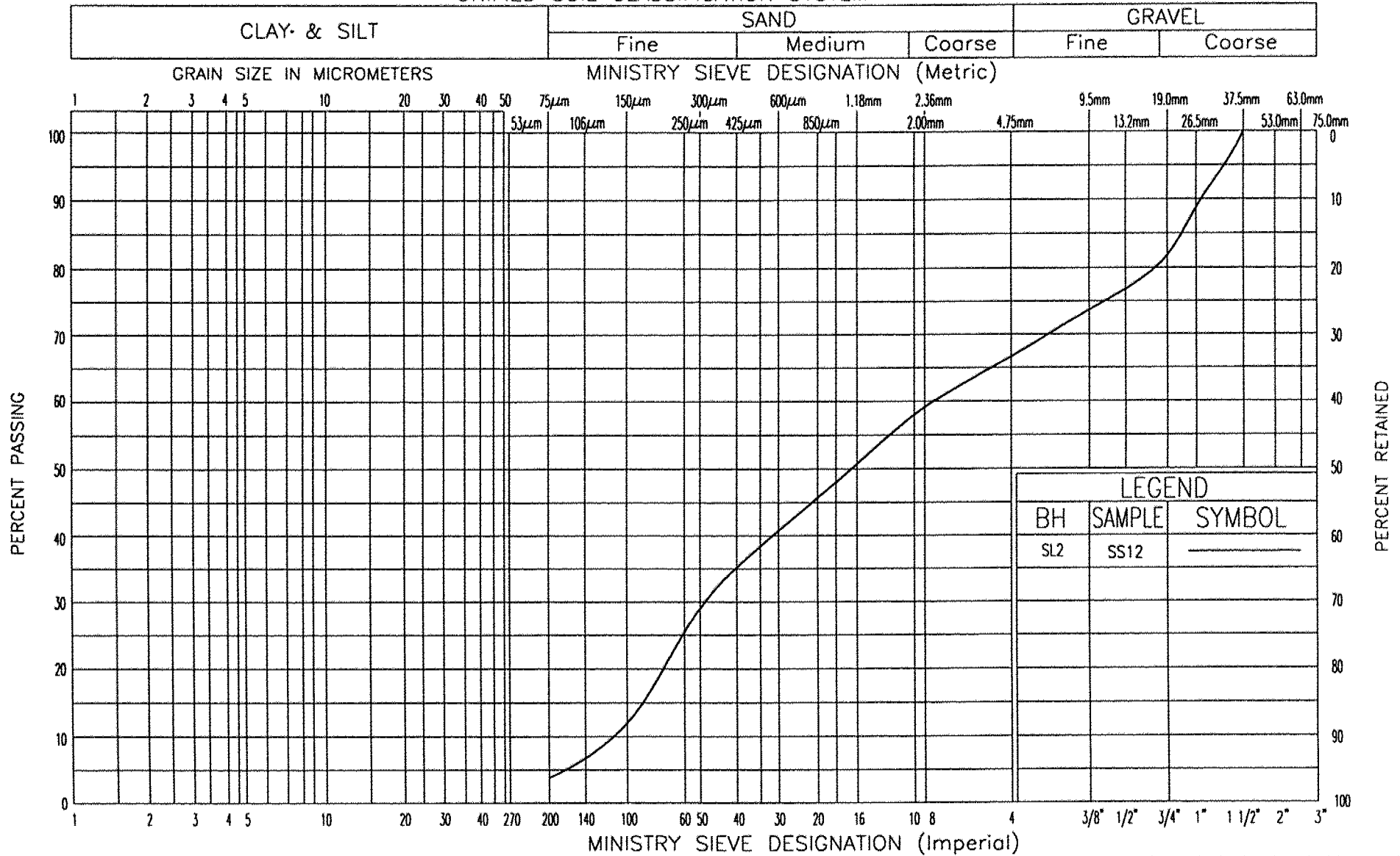


GRAIN SIZE DISTRIBUTION
UPPER SAND TO SAND & GRAVEL

FIG No 1

W P 466-93-00

UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION
GRAVELLY SAND

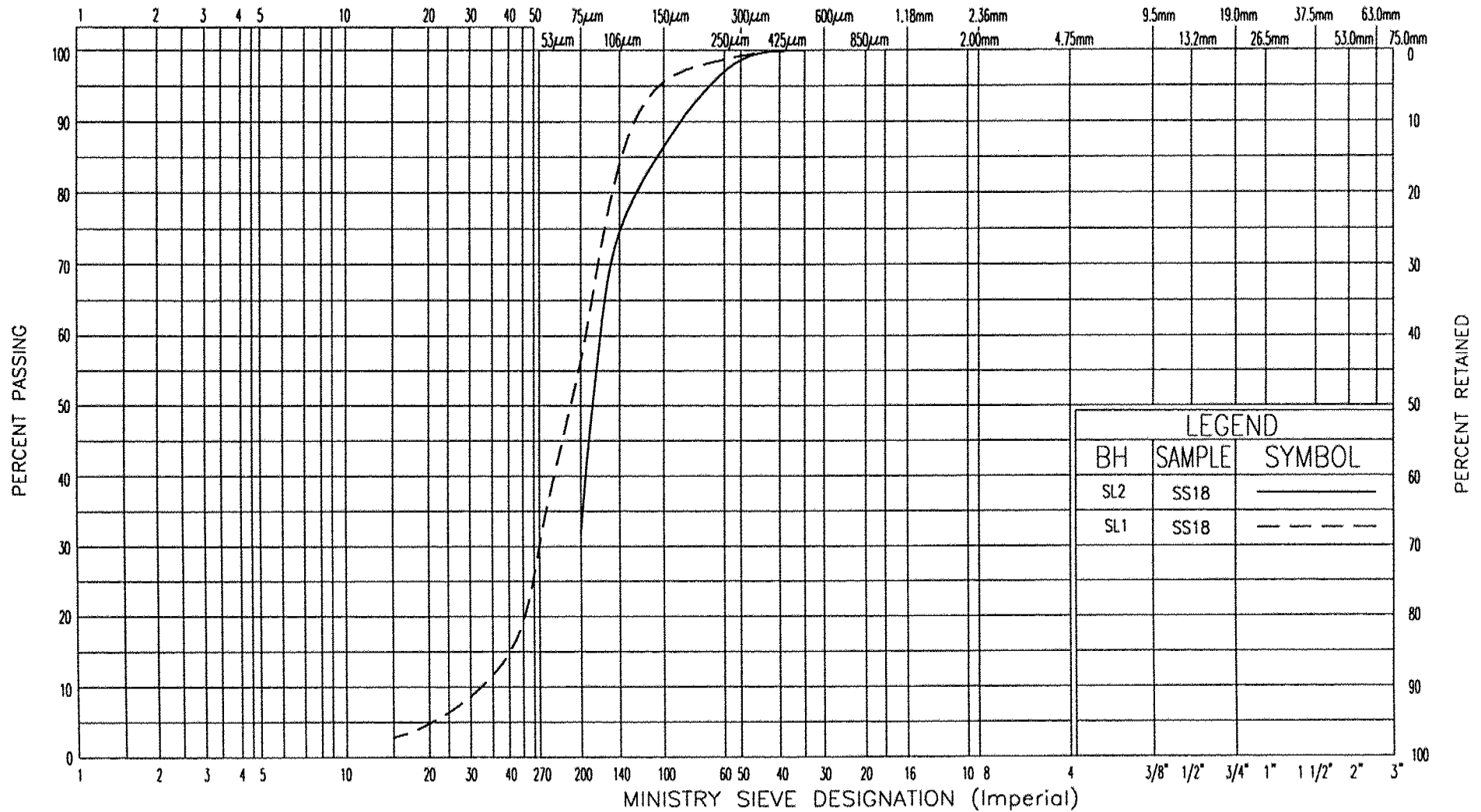
FIG No 3
W P 466-93-00

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

MINISTRY SIEVE DESIGNATION (Metric)



GRAIN SIZE DISTRIBUTION
LOWER SILTY SAND to SANDY SILT

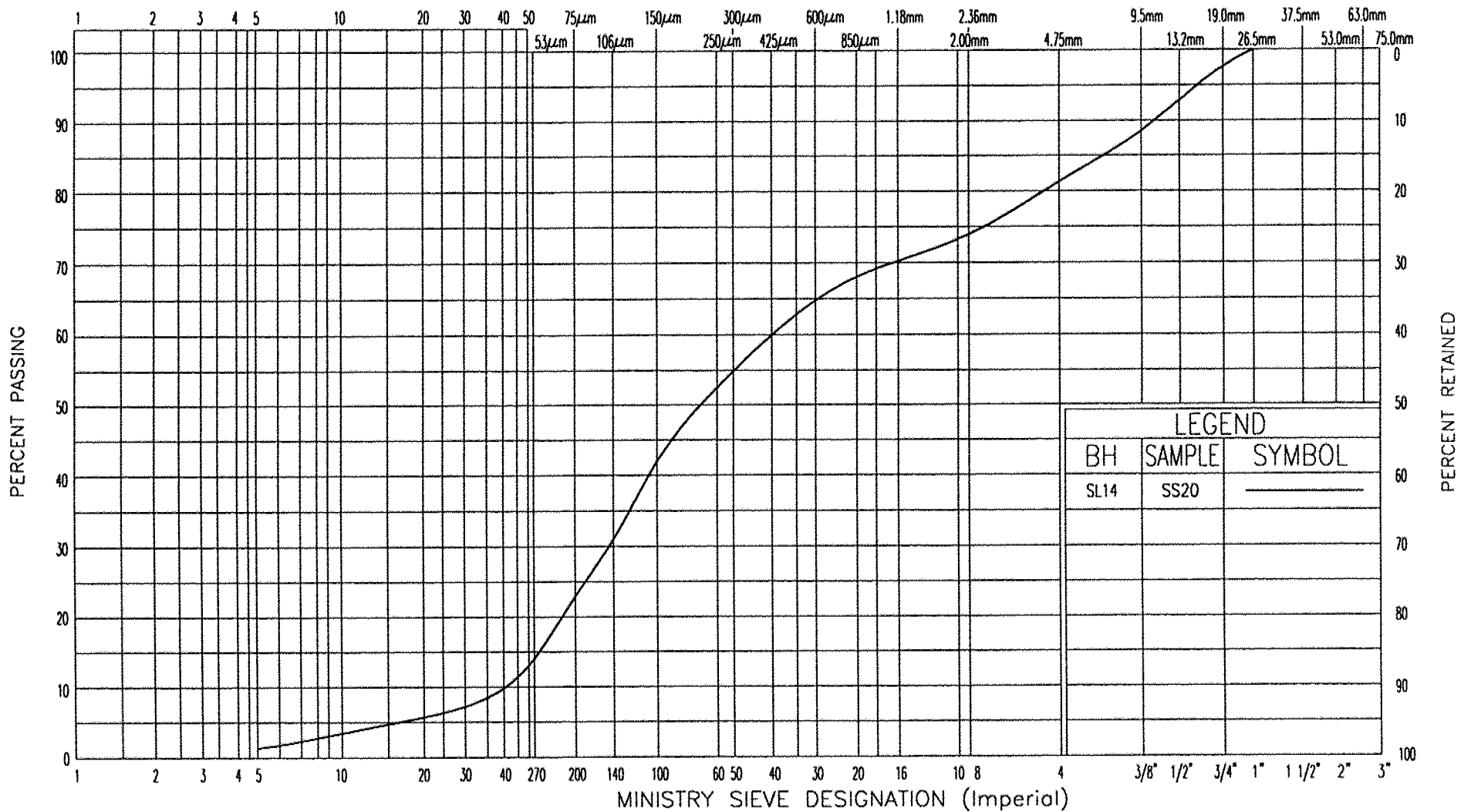
FIG No 4
W P 466-93-00

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse

GRAIN SIZE IN MICROMETERS

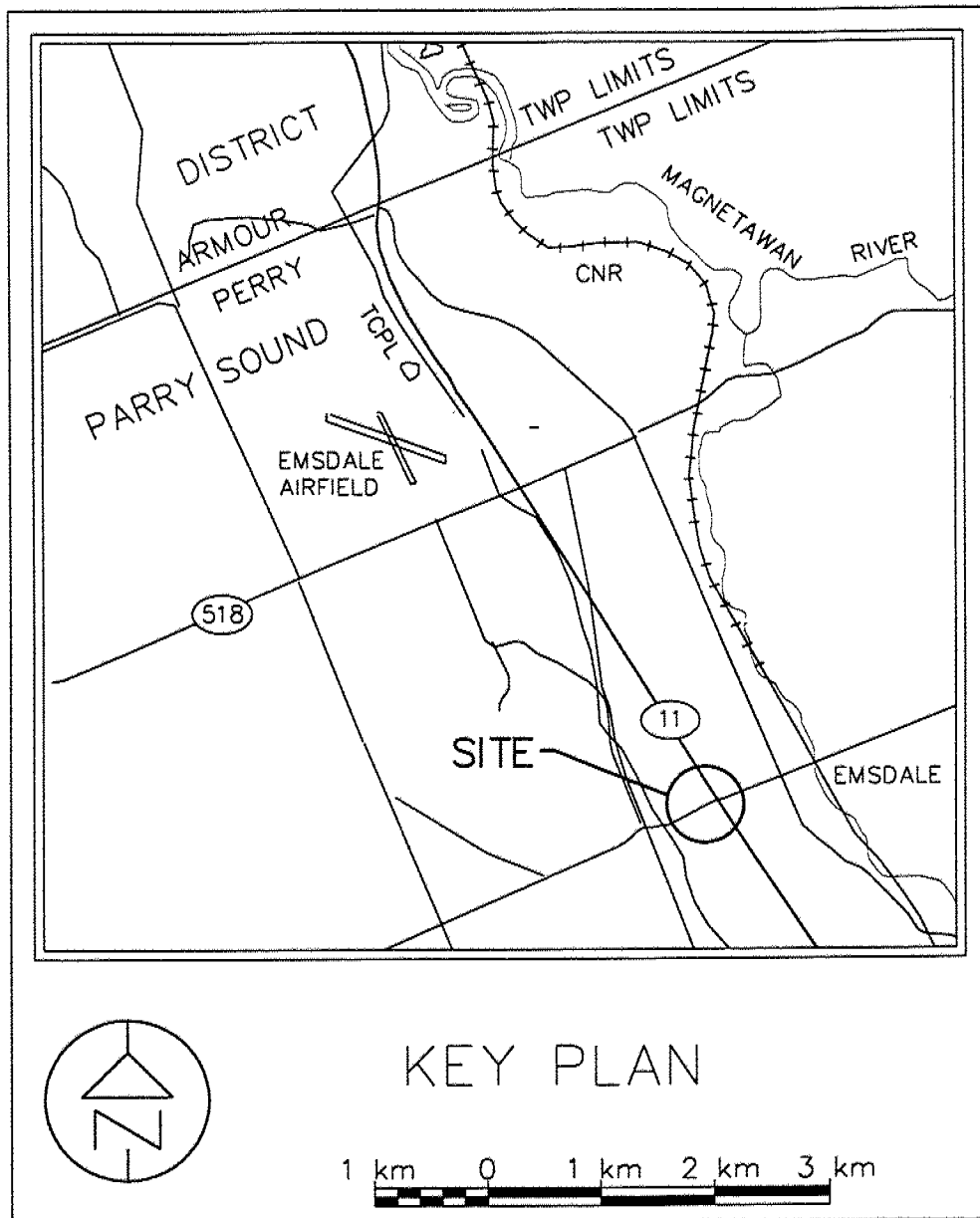
MINISTRY SIEVE DESIGNATION (Metric)



GRAIN SIZE DISTRIBUTION
LOWER SAND & GRAVEL

FIG No 5
W P 466-93-00

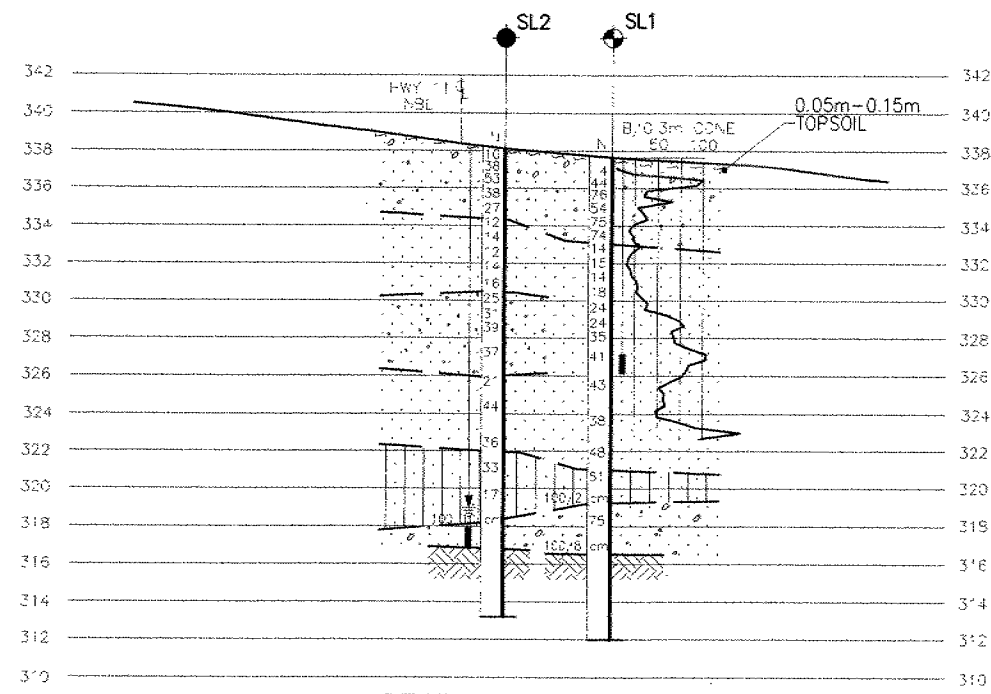
ENCLOSURES



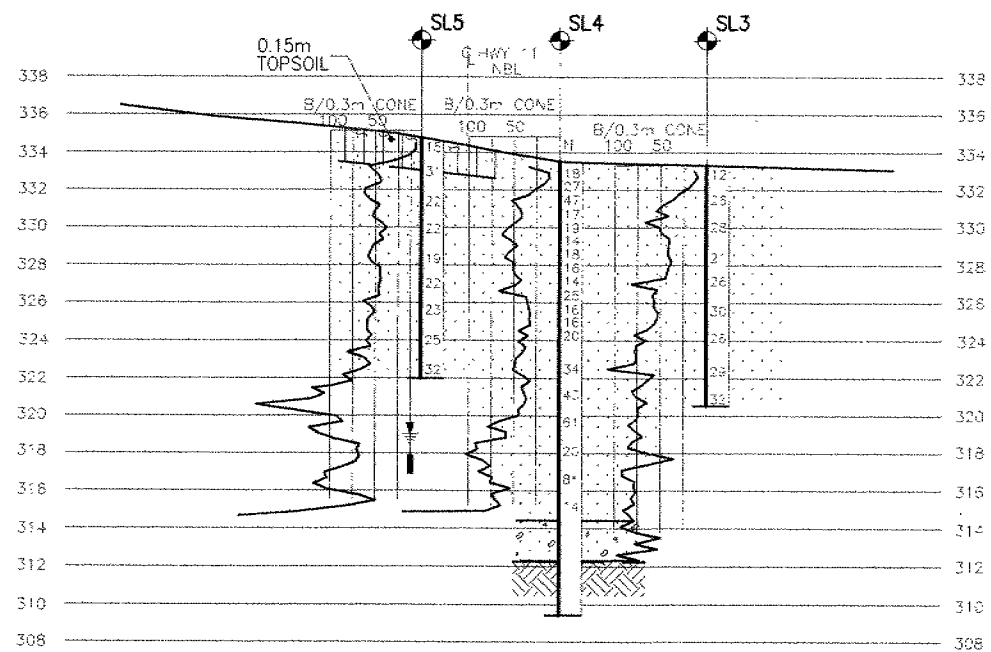
STAR LAKE ROAD OVERPASS (NBL)
KEY PLAN

Dwg. No 1





SECTION B-B

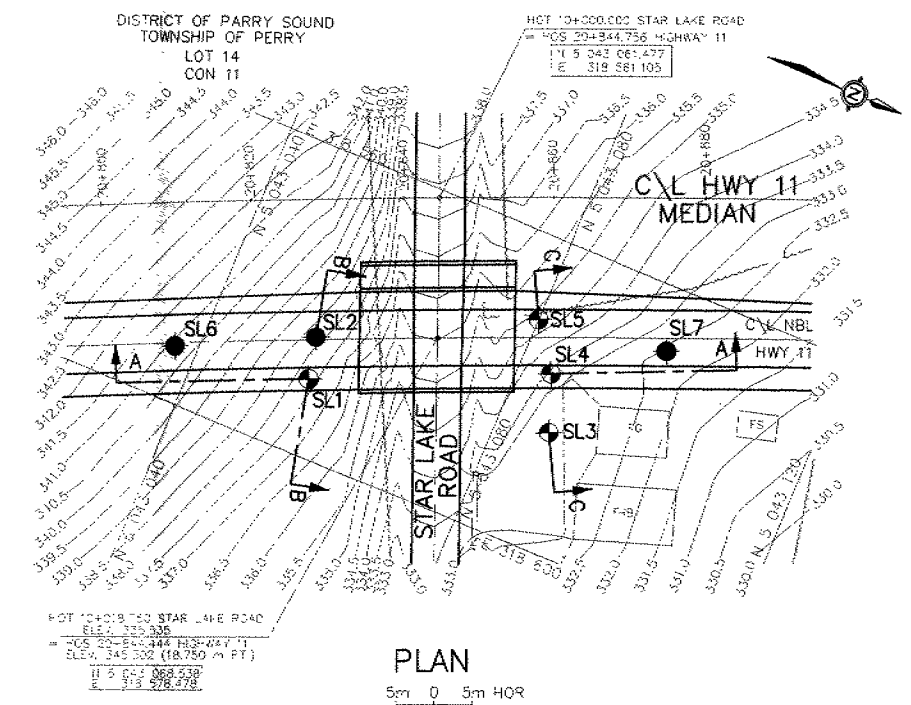


SECTION C-C

2m 0 2m

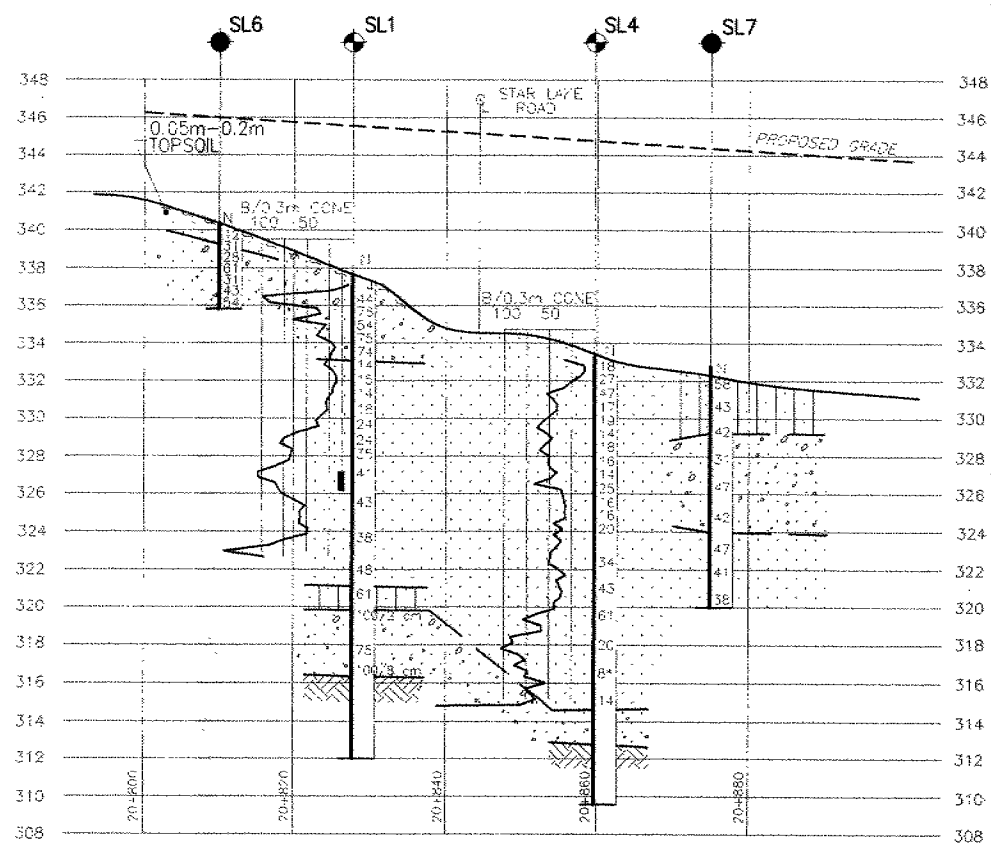
SOIL STRATIGRAPHY LEGEND

- | | | | |
|--|---|--|--|
| | SAND to SAND & GRAVEL
Very Loose to Very Dense | | SAND
Compact to Dense |
| | GRAVELLY SAND
Compact to Dense | | SILTY SAND to
SANDY SILT
Dense to Very Dense |
| | GRANODIORITE
BEDROCK | | |



PLAN

5m 0 5m HQR



SECTION A-A

5m 0 5m HQR
2m 0 2m VER

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES - METRES.

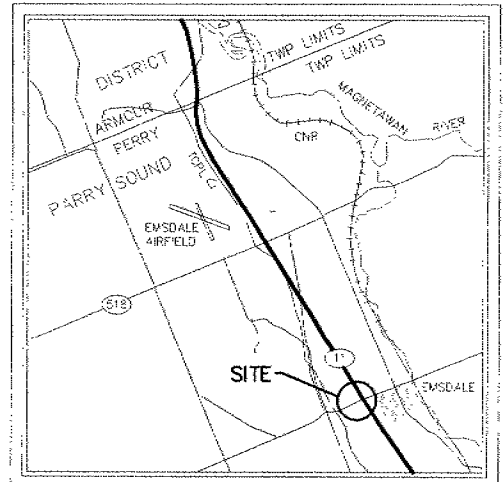


CONT. No.
W.P. No. 468-93-01

STAR LAKE ROAD OVERPASS (NBL)
BORE HOLE LOCATIONS & SOIL STRATA

SHEET

AGRA Earth & Environmental Ltd.



KEY PLAN

1 km 0 1 km 2 km 3 km

LEGEND			
	Bore Hole		
	Dynamic Cone Penetration Test (Cone)		
	Bore Hole & Core		
	Blows/0.3m (Std Pen Test, 475 J/blow)		
	CONE Blows/0.3m (60' Cone, 475 J/blow)		
	WL at time of investigation Feb. 99		
	WL in Piezometer		
	Piezometer		
No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
SL1	337.7	5 043 055	318 590
SL2	338.2	5 043 054	318 584
SL3	333.3	5 043 087	318 584
SL4	333.6	5 043 084	318 577
SL5	334.8	5 043 081	318 571
SL6	340.9	5 043 037	318 593
SL7	332.8	5 043 097	318 568

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

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 20.01 of QPS Gen-Cond.

REV	DATE	BY	DESCRIPTION

REF. Hwy 11 Bridge Site Plan
Dwg. by MTO: Jan. 1999

Hwy No 11	CHECKED AD	DATE June, 1999	DIST 52-HUNTSVILLE
SUBM'D 20	CHECKED	APPROVED	SITE 44-392N
DRAWN MA	CHECKED		DWG 2

RECORD OF BOREHOLE No SL2

1 OF 2

METRIC

W.P. 468-93-01 LOCATION N 5043053.8 E 318584.5 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem COMPILED BY CK
DATUM Geodetic DATE 26 February 1999 - 27 February 1999 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL							x LAB VANE	20
338.2	0.15m TOPSOIL		1	SS	10		338								GR SA SI CL Station 20 + 828 0.2 Lt NBL C/L			
0.0	loose		2	SS	38		337								51 43 (6)			
	brown SAND to SAND & GRAVEL compact to very dense damp		3	SS	53		336								17 81 (2)			
			4	SS	38		335											
			5	SS	27		334											
334.4			6	SS	12		333								0 92 (8)			
3.8	brown SAND fine to medium compact dry		7	SS	14		332											
			8	SS	12		331											
			9	SS	14		330											
			10	SS	16		329											
330.6			11	SS	25		328											
7.6	compact dense		12	SS	31		327								34 62 (4)			
	brown GRAVELLY SAND dry		13	SS	39		326											
			14	SS	37		325											
			15	SS	21		324								0 98 2 0			
326.0	compact		16	SS	44													
12.2	dense																	
	brown SAND fine to medium dry																	

Continued Next Page

+ 3, X 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No SL2										2 OF 2		METRIC		
W.P. 468-93-01		LOCATION N 5043053.8 E 318584.5				ORIGINATED BY AD								
DIST 52 HWY 11		BOREHOLE TYPE Hollow Stem				COMPILED BY CK								
DATUM Geodetic		DATE 26 February 1999 - 27 February 1999				CHECKED BY ZSO								
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						
							20 40 60 80 100	20 40 60 80 100	10 20 30					
322.0	brown SILTY SAND to SANDY SILT compact to dense damp		17	SS	36									0 67 (33)
16.2														
			18	SS	33									
			19	SS	17									
318.6	brown SAND & GRAVEL with cobbles & boulders		20	SS	100/40									RC21: REC=61% RQD=23%
19.6														
	wet													
317.0	DIORITE BEDROCK massive, moderately closely jointed		21	RC										RC22: REC=58% RQD=21% RC23: REC=81% RQD=44% RC24: REC=74% RQD=0% RC25: REC=45% RQD=13%
21.2														
			22	RC										
			23	RC										
			24	RC										
			25	RC										
313.2	END OF BOREHOLE		26	RC										RC26: REC=100% RQD=83% RC27: REC=75% RQD=75%
25.0														
	WL IN PIEZOMETER: Feb 27/99: 19.3m													

RECORD OF BOREHOLE No SL3

1 OF 2

METRIC

W.P. 468-93-01 LOCATION N 5043087.1 E 318584.5 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem COMPILED BY CK
DATUM Geodetic DATE 21 February 1999 CHECKED BY ZSO

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w_p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w_L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa						WATER CONTENT (%)
						20	40	60	80	100	10	20	30	
333.3 0.0	frozen		1	SS	12									Station 20 + 860 12.8 Rt NBL C/L
			2	SS	26									0 96 (4)
			3	SS	28									
			4	SS	21									0 96 (4)
	brown SAND fine to medium compact dry to damp		5	SS	26									
			6	SS	30									2 88 (10)
			7	SS	26									
			8	SS	29									
			9	SS	32									
320.5 12.8	dense													
	WL on completion: none Dynamic Cone Penetration Test conducted 2.0m West of Borehole.													

Continued Next Page

+ ³ X ³ Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No SL3

2 OF 2

METRIC

W.P. 468-93-01 LOCATION N 5043087.1 E 318584.5 ORIGINATED BY AD
 DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem COMPILED BY CK
 DATUM Geodetic DATE 21 February 1999 CHECKED BY ZSO

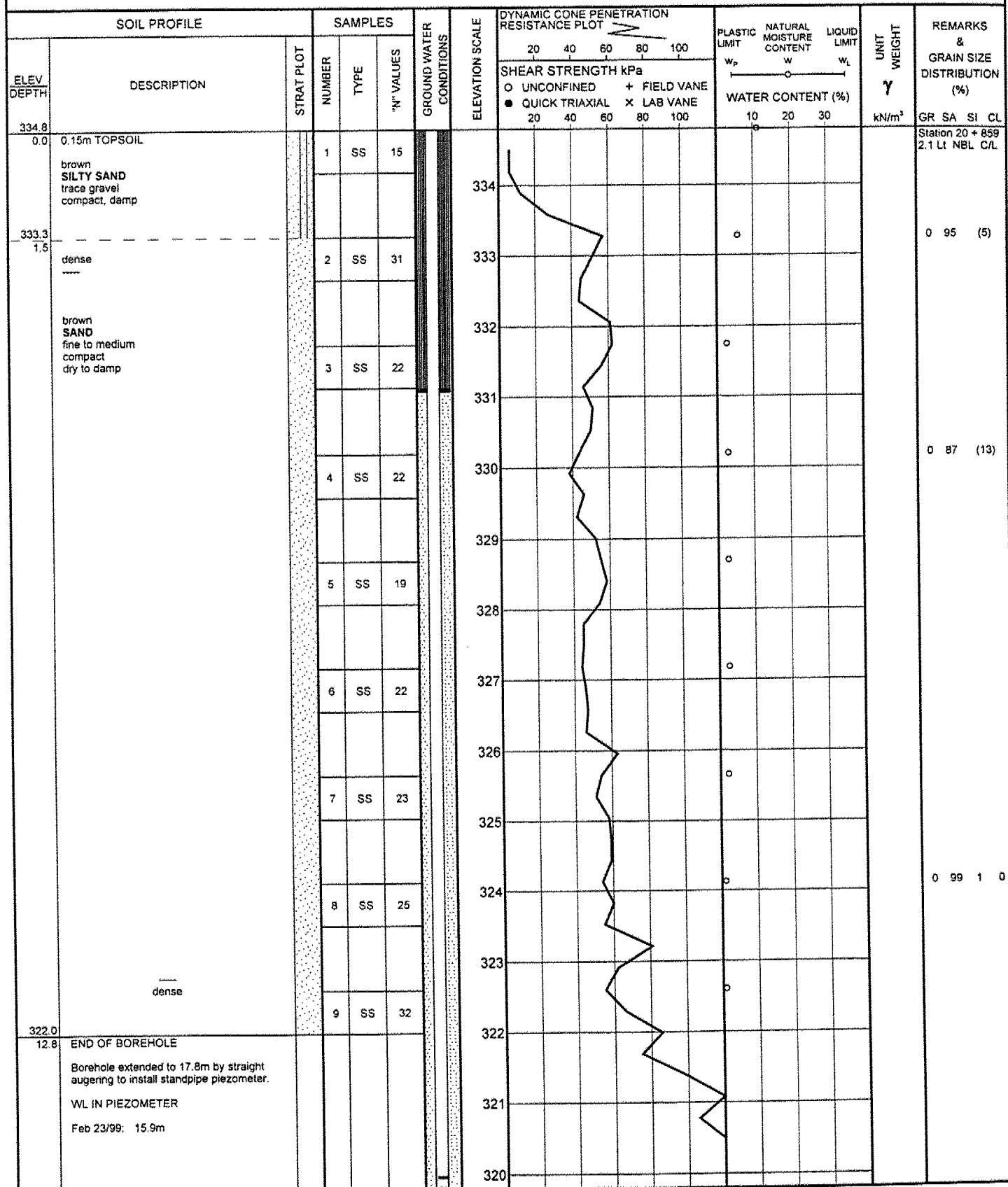
SOIL PROFILE		SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
312.0														
21.4	END OF DCPT @ 21.4m													

RECORD OF BOREHOLE No SL5

1 OF 2

METRIC

W.P. 468-93-01 LOCATION N 5043080.8 E 318571.0 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Solid Stem/Hollow Stem COMPILED BY CK
DATUM Geodetic DATE 22 February 1999 CHECKED BY ZSO



Continued Next Page

+ 3, x 3: Numbers refer to Sensitivity
○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No SL5

2 OF 2

METRIC

W.P. 468-93-01 LOCATION N 5043080.8 E 318571.0 ORIGINATED BY AD
 DIST 52 HWY 11 BOREHOLE TYPE Solid Stem/Hollow Stem COMPILED BY CK
 DATUM Geodetic DATE 22 February 1999 CHECKED BY ZSO

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 (%)					
314.1														
20.7	END OF DCPT @ 20.7m Dynamic Cone Penetration Test conducted 2m West of Borehole													

RECORD OF BOREHOLE No SL6

1 OF 1

METRIC

W.P. 468-93-01 LOCATION N 5043037.3 E 318592.8 ORIGINATED BY AD
DIST 52 HWY 11 BOREHOLE TYPE Hollow Stem COMPILED BY CK
DATUM Geodetic DATE 25 February 1999 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
								○ UNCONFINED		+ FIELD VANE		● QUICK TRIAXIAL						
340.9							20	40	60	80	100							
0.0	0.2m TOPSOIL		1	SS	12											GR SA SI CL		
	brown SAND fine to medium compact to dense damp		2	SS	31											Station 20 + 810 0.6 Rt NBL C/L		
339.5																		
1.4	SAND to SAND & GRAVEL compact to very dense dry		3	SS	28											20 73 (7)		
			4	SS	61													
			5	SS	31													
			6	SS	43													
			7	SS	54											50 46 (4)		
335.9																		
5.0	END OF BOREHOLE																	
	WL on completion: none																	

RECORD OF BOREHOLE No SL7

1 OF 1

METRIC

W.P. 468-93-01 LOCATION N 5043097.2 E 318568.4 ORIGINATED BY AD
 DIST 52 HWY 11 BOREHOLE TYPE Solid Stem/Hollow Stem COMPILED BY CK
 DATUM Geodetic DATE 23 February 1999 CHECKED BY ZSO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE LIQUID LIMIT CONTENT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		w _p	w	w _L		
332.8 0.0	frozen		1	SS	58									GR SA SI CL Station 20 + 875 1.8 Rt NBL C/L
	brown SILTY SAND with occasional Clayey Silt seams dense, damp						332							
			2	SS	43		331							
							330							
329.5 3.3			3	SS	42		329							47 47 (6)
	brown SAND to SAND & GRAVEL dense damp to dry						328							
			4	SS	31		327							
							326							44 51 5 0
			5	SS	47		325							
			6	SS	42		324							
323.9 8.9							323							
	brown SAND fine to medium dense damp to dry		7	SS	47		322							0 95 (5)
							321							
			8	SS	41									
			9	SS	38									
320.0 12.8	END OF BOREHOLE						320							
	WL on completion: none													

MEMORANDUM



To: V. Minassian, P. Eng.
Senior Project Engineer
Planning and Design, Northern Region

August 9, 1999

From: Pavements and Foundations Section
Room 315, Central Bldg.

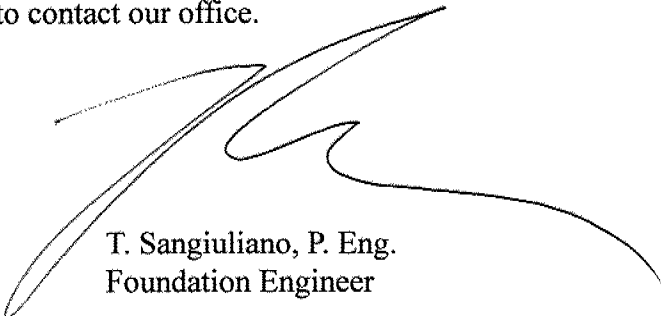
Tel: (416) 235-5267
Fax: (416) 235-5240

Re: Draft Foundation Investigation Report Review
Star Lake Rd Overpass, SBL
Hwy 11
WP 466-93-00
District 54, Sudbury

We have completed a review of the Final Foundation Investigation and Design Report for the abovementioned structure. In general, the report has addressed our review comments previously submitted in a memorandum dated June 14, 1999.

As mentioned in our previous memorandum, however, we maintain that in employing the Hiley Dynamic Formula, piles are recommended to be driven to a minimum tip elevation using an ultimate capacity equivalent to twice the Factored Capacity at ULS. Consequently, it is recommended that piles be driven to an ultimate of 3200 kN (2 x 1600 kN) rather than 4000 kN. In driving the piles to a higher ultimate capacity at the site, there is an increased risk in damaging the piles.

If you require additional assistance, please do not hesitate to contact our office.



T. Sangiuliano, P. Eng.
Foundation Engineer

for

D. Dundas, P. Eng.
Senior Foundation Engineer

cc. T. Kazmierowski

MEMORANDUM



To: P. Lecoarer, P. Eng.
Project Manager
Planning and Design, Northern Region

July 15, 1999

From: Pavements and Foundations Section
Room 315, Central Bldg.

Tel: (416) 235-5267
Fax: (416) 235-5240

Re: Draft Foundation Investigation Report Review
Hwy 518 (Fern Glen) Underpass
Hwy 11
WP 466-93-00
District 54, Sudbury

We have completed a review of the Draft Foundation Investigation and Design Report for the abovementioned structure. Our review comments are contained in this memorandum.

Our review is based on verifying that the Foundation Investigation and Design Reports satisfy the terms of reference for completeness. The Consultant is responsible for the technical accuracy of the recommendations contained in the report. Any deficiency identified in this memorandum is intended to alert the Consultant but shall not relieve the Consultant of any responsibility for their work.

Review comments are given under two major categories:

- I Discussion and Recommendations
- II Factual Component

I Discussion and Recommendations

5.0

The first paragraph in this section describes the existing ground surface elevations and the proposed profile grades of both Hwy 11 and Fern Glen Rd. It is recommended that the proposed profile grade of Hwy 11 be superimposed on Sections A-A and B-B on Drawing 1.

5.1 Foundations

5.1.1 Shallow Foundations

In Table 1, BH F6 is not referenced as a borehole for the west abutment.

Shallow foundations founded on bedrock have been recommended for the west abutment and pier. At the east abutment, the foundation alternatives discussed include spread footings on bedrock, spread footings on a compacted granular "A" pad and also drilled caissons. Discussion of the feasibility of drilled caissons and any recommendations should be given under Section 5.1.2. entitled Deep Foundations.

A discussion is included in this section regarding the requirement for frost protection. The report states that the frost protection requirement is a function of the "massiveness" or equivalently "jointing" of the bedrock. Although this is technically correct, the report should clearly recommend based on the knowledge of the bedrock and the experience of foundation performance in the area the frost protection requirement.

The report identifies several options to augment the sliding resistance of the shallow foundation on bedrock (shear keys, dowels, surface roughening, rock anchors). The horizontal capacity and bond stress of the rock should be included in the report to facilitate the design of dowels and shear keys. The report should also comment on the relative practicality and costs of the options mentioned. Although integral abutments are one option at the site, the report should disclose whether a shallow foundation is feasible.

5.1.1.1 West Abutment

It has been recommended that the west abutment foundation be founded on bedrock at Elevation 336 m or lower in order to minimize the effects of a bedrock surface that slopes from west to east. The bedrock surface elevation slopes from approximate Elevation 337 to 337.5 m to 335.3 m within the west abutment footing area. Although the bedrock continues to dip to an approximate Elevation of 334 to 331 m at the pier location, the need and benefits for the slope effect minimization requires further explanation. The excavation may be required to simply facilitate the design of the abutment wall in view of the shallow bedrock in the area. BH F6 encountered the bedrock at Elevation 338 m and the probe holes encountered refusal at Elevations as shallow as 337.5 m

The last sentence in the first paragraph of this section states that "this recommended founding elevation may however have to be revised (ie somewhat lowered) after excavation depending on actual conditions. The report should provide the reason for the possible change in founding elevation (for example, is it the possibility that the bedrock surface is lower?).

The report recommends that the footing be placed outside a plane defined as 2H:1V from the toe of the cut slope, but not less than 2 m. We consider this recommendation too conservative. On previous MTO projects, we have employed minimum edge distances equivalent to 1 m.

5.1.1.3 East Abutment

In the opening sentence in this section the large variation in the bedrock surface profile is mentioned. As a result, the report presents other options for the foundation design of the east abutment. It appears that the report recommends the footings on bedrock as the first option. It is recommended that this be explicitly stated in the report based on a comparison of the costs, risks and feasibility and all the foundation options.

The report recommends a bearing resistance of 300 kPa at SLS for foundations founded on a minimum 1 metre thick Granular 'A' pad. On MTO projects in the past under similar conditions, we have recommended a bearing resistance of 350 kPa at SLS. In our opinion, the Consultant's recommendation is too conservative.

5.1.2 Deep Foundations

The report discusses caissons as an option at the east abutment. Driven steel piles are not discussed as an option. It is recommended that this option be discussed in the report. As mentioned earlier, the caisson option should also be discussed under this section.

In the bottom paragraph, line 6 on page 10 of the report, reference is made that "While the settlement of the west abutment..." Reference should be to the east abutment.

5.2 Lateral Earth Pressures

On page 12, the report discusses the placement of rock fill as backfill. It is recommended that the report makes reference to OPSD 3505 and the text in the report be consistent with the requirements illustrated on this drawing.

The report suggests that Retained Soil Systems "may not be very suitable for this project" due to the expected variations in the rock surface. Retained soil systems have been placed at sites with similar conditions in the past.

5.3 Approach Embankments

In describing the compaction of the select subgrade material, reference should be made to OPSS 501. The placement of rock fill should also be discussed under this section of the report.

II Factual Component

1 Introduction

In the second paragraph, recommendations should be for the "foundation design of the proposed structure and approach fills/cuts". The term "foundation" is used on MTO projects rather than geotechnical for structures and related earth/rock works

4.0 Subsurface Conditions

The sand to gravelly sand deposit is not mentioned in Section 4.0 and then is described in Section There is consequently some inconsistency.

It is recommended that the last sentence in this section be deleted: "The following paragraphs are only meant to complement and summarize these data". The report should state that soil descriptions are given in this section of the report.

The colour of the soil should be included in the descriptions. Generally, the MTO does not include the adjectives (for example, "fine to medium") in the title block of the soil description. This also applies to the borehole logs.

4.5 Bedrock

It is recommended that the report elaborate on the varying bedrock surface at the site. In paragraph 3 of this section, it is stated that "while in the area of the east abutment location a more complex picture emerges with rock elevations ranging from" The report should discuss how the rock is dipping.

In describing the degree of weathering, the term "somewhat" is used (second last paragraph on page 5). An appropriate geotechnical description should be used such as "slightly".

We recommend that the Consultant be directed to acknowledge and address the concerns and issues raised in this memo.

We trust these comments are sufficient for your purposes. If you require additional assistance, please do not hesitate to contact our office.

T. Sangiuliano, P. Eng.
Foundation Engineer

for

D. Dundas, P. Eng.
Senior Foundation Engineer

cc. T. Kazmierowski

G.I.-30 SEPT. 1976

GEOCRES No. 31E-125DIST. 52 REGION W.P. No. 403/404-97-00
GWP: 290-97-00CONT. No. W. O. No. STR. SITE No. 44-380 S/NHWY. No. 69LOCATION Lawson Bay Rd.
OolpanNo of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. REMARKS:

FOUNDATION INVESTIGATION REPORT
FOR
LAWSON BAY ROAD OVERPASS
W.P. 290-97-00, SITE 44-380 N & S
HIGHWAY 69, DISTRICT 52
HUNTSVILLE, ONTARIO

{ W.P. 403-97-00
W.P. 404-97-00 }

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Job No. 97TF088A

June, 1998

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FOUNDATION INVESTIGATION REPORT

For

Lawson Bay Road Overpass

W.P. 290-97-00, Site 44-380 N & S

Highway 69, District 52, Huntsville

INTRODUCTION

This report summarizes the results of the foundation investigation carried out for construction of the proposed Highway 69 overpass at Lawson Bay Road (Station 18+838 Highway 69 chainage).

The report pertains to the bridge structures and approaches within about 20 m of the abutments, between approximate stations 18+820 to 18+875 southbound lanes and 18+800 to 18+855 northbound lanes, Highway 69 chainage.

SITE DESCRIPTION

The site is located about 15 km north of MacTier and some 1.5 km west of the existing Highway 69 alignment. The proposed structures will carry Highway 69 four-lane traffic over a realigned Lawson Bay Road. At the overpass, Lawson Bay Road will run approximately east-west.

The bridge location is presently wooded. The ground surface undulates slightly with areas of low bedrock outcrops.

The site is located in the Precambrian Laurentian peneplane. The topography is irregular in detail with many small lakes separated by ridges of Precambrian bedrock. The surface in general is relatively flat. The overburden in the region is typically shallow but can vary

substantially in thickness over short distances. Swamp environments have developed in areas of poor drainage.

INVESTIGATION PROCEDURES

The fieldwork was carried out on February 28 and March 1, 1998 and comprised 12 boreholes drilled at the locations shown on Drawings 1 and 2.

The boreholes were drilled to refusal on bedrock/inferred bedrock at depths of 0.1 to 2.2 m. Two of the boreholes at each structure were extended an additional 3.2 to 3.4 m into the bedrock using NQ rock coring equipment.

The boreholes were advanced using continuous flight hollow stem augers, powered by a track-mounted CME-55 drillrig, supplied and operated by a specialist drilling contractor, working under the full-time supervision of a member of our engineering staff.

The overburden composition was defined by examination of auger cuttings and downhole observation since bedrock was very shallow. Standard penetration testing and simultaneous sampling using a conventional split spoon sampler were conducted once where a slightly greater overburden thickness was encountered. The groundwater conditions in the boreholes were closely monitored during the course of the fieldwork.

Samples of the recovered rock core were subjected to unconfined compressive strength tests.

SUMMARIZED SUBSURFACE CONDITIONS

Reference is made to the appended Log of Borehole sheets for details of the subsurface conditions including soil classifications, inferred stratigraphy, standard penetration test "N" values, rock core descriptions and groundwater observations. Stratigraphic profiles prepared from the borehole data are presented on Drawings 1 and 2.

The stratigraphy revealed in the boreholes generally comprised a surficial topsoil layer overlying discontinuous sand/silt/clay deposits mantling bedrock. Surficial fill was encountered in one borehole and bouldery material was revealed locally. The strata encountered are summarized below.

Sand and Gravel Fill

A 600 mm thick layer of sand and gravel fill (road base) was encountered surficially in borehole 380-4.

Topsoil

Topsoil was encountered surficially in all boreholes except borehole 380-4. The topsoil layer was 50 to 300 mm thick and comprised silty clay judged to have a medium organic content.

Sand

Silty sand was revealed below the topsoil in boreholes 320-1, 2, 5, 10 and 11. The sand contained occasional to numerous cobbles and boulders. It was penetrated at 1.2 m depth in borehole 380-11 and mantled bedrock/inferred bedrock in the remaining boreholes.

Silt

A 300 mm thick layer of sandy silt was encountered below the fill in borehole 380-4.

Clay

A 1.0 m thick layer of silty clay was revealed below the topsoil in borehole 380-9 and below the sand in borehole 380-11. The clay was judged to be soft to firm in borehole 380-9 and stiff in borehole 380-11. It was bouldery below 1.7 m depth in borehole 380-11.

Bedrock

Bedrock or inferred bedrock was contacted below the topsoil, sand, silt or clay in all boreholes at depths of 0.1 to 2.2 m (elevation 239.9 to 244.5).

A description of the rock cores recovered from boreholes 380-2, 5, 9 and 11 is provided on Table I. The bedrock consists of hornblende and biotite migmatite, locally granitic pegmatite in borehole 380-11. Core recovery ranged from 88 to 100% (average 97%) and the RQD typically ranged from 58 to 100% (average 86%). The unconfined compressive strength of selected core samples were as follows:

Borehole No.	Depth (m)	Unconfined Compressive Strength (MPa)
380-2	1.8 - 1.9	62.0
380-5	1.4 - 1.5	36.2
380-9	2.5 - 2.6	65.0
380-11	2.2 - 2.3	85.1

Groundwater

Upon completion of augering, free water was observed in boreholes 380-1 and 11 at 0.7 and 0.3 m depth (elevation 240.3 and 242.0), respectively. Borehole

380-9 was drilled within 150 mm of ponded snowmelt water. Free water was not observed in the remaining boreholes during the course of the fieldwork. Observed water levels are subject to seasonal fluctuations and rainfall patterns.

CLOSURE

The fieldwork was carried out under the supervision of D. L. Watson. The equipment was supplied by Longyear Canada Inc.

The report was written by M.R. Anderson, Project Engineer and reviewed by D.W. Kerr, Manager of Geotechnical and Geo-Environmental Services, Hamilton.



Yours very truly

Peto MacCallum Ltd.

A handwritten signature of Murray R. Anderson in black ink.

Murray R. Anderson, M.Eng., P.Eng.
Project Engineer



A handwritten signature of Dennis W. Kerr in black ink.

Dennis W. Kerr, M.Eng., P.Eng.
Manager Geotechnical and
Geo-Environmental Services
Hamilton

MRA:mmma

TABLE I

**ROCK CORE DESCRIPTION
WP 290-97-00, Site No. 44-380 N & S**

CORE RECOVERY					CORE DESCRIPTION	
BOREHOLE	CORE NO.	DEPTH (m)	RECOVERY (%)	RQD (%)	DEPTH (m)	DESCRIPTION
380-2	1	1.24 - 1.55	92	58	1.24 - 4.57	HORBLENDE MIGMATITE (with occ. 300 mm thick layers of biotite migmatite), black and grey, banded, fine to medium crystalline, high strength, unweathered; with lenses/seams of biotite mica; wide spaced dipping partings; moderately to highly altered for 50 to 100 mm, yellow to rust coloured, friable, low strength
	2	1.55 - 3.00	96	84		
	3	3.00 - 4.57	100	95		
380-5	1	0.76 - 1.68	100	67*	0.76 - 3.96	HORNBLENDE MIGMATITE , black and grey, banded to homogeneous, fine to medium crystalline, medium to high strength, unweathered; with minor muscovite mica; moderate to wide spaced joints, rough planar, tight
	2	1.68 - 3.20	100	93		
	3	3.20 - 3.96	100	100		
380-9	1	1.07 - 1.42	100	0*	.07 - 1.42	BIOTITE MIGMATITE , black, homogeneous, fine to medium crystalline, low to medium strength, unweathered; very close to close spaced dipping partings parallel to schistosity, rough planar, tight
	2	1.42 - 2.87	98	77	1.42 - 4.47	
	3	2.87 - 4.47	97	90		BIOTITE MIGMATITE , black, banded, slightly granitized, medium to high strength, unweathered; with thin layers of white or grey pegmatite; close to moderate spaced partings, rough planar, tight

RQD = Rock Quality Designation

* Low RQD due to casing disturbance

Logged by J. Wright

TABLE I Cont'd

ROCK CORE DESCRIPTION
WP 290-97-00, Site No. 44-380 N & S

CORE RECOVERY					CORE DESCRIPTION	
BOREHOLE	CORE NO.	DEPTH (m)	RECOVERY (%)	RQD (%)	DEPTH (m)	DESCRIPTION
380-11	2	2.21 - 2.41	88	0*	2.21 - 3.53	GRANITIC PEGMATITE, grey, coarse crystalline, high strength, unweathered; with lenses of biotite mica; close to moderate spaced partings, rough planar, tight
	3	2.41 - 3.99	98	95		
	4	3.99 - 5.61	92	81	3.53 - 4.78	BIOTITE MIGMATITE, black, homogeneous, moderately granitized, medium strength, unweathered; close to moderate spaced dipping partings parallel to schistosity, rough planar, tight
					4.78 - 5.61	GRANITIC PEGMATITE, grey, coarse crystalline, high strength, unweathered; with lenses of biotite mica; close to moderate spaced partings, rough planar, tight

RQD = Rock Quality Designation

* Low RQD due to casing disturbance

Logged by J. Wright

LIST OF ABBREVIATIONS

PENETRATION RESISTANCE

STANDARD PENETRATION RESISTANCE 'N', - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A STANDARD SPLIT SPOON SAMPLER 0.3m INTO THE SUBSOIL. DRIVEN BY MEANS OF A 63.5kg HAMMER FALLING FREELY A DISTANCE OF 0.76m.

DYNAMIC PENETRATION RESISTANCE : - THE NUMBER OF BLOWS REQUIRED TO ADVANCE A 51mm, 60 DEGREE CONE, FITTED TO THE END OF DRILL RODS, 0.3m INTO THE SUBSOIL. THE DRIVING ENERGY BEING 475 J PER BLOW.

DESCRIPTION OF SOIL

THE CONSISTENCY OF COHESIVE SOILS AND THE RELATIVE DENSITY OR DENSENESS OF COHESIONLESS SOILS ARE DESCRIBED IN THE FOLLOWING TERMS :-

<u>CONSISTENCY</u>	<u>'N' BLOWS/0.3 m</u>	<u>c kPa</u>	<u>DENSENESS</u>	<u>'N' BLOWS/0.3 m</u>
VERY SOFT	0 - 2	0 - 12	VERY LOOSE	0 - 4
SOFT	2 - 4	12 - 25	LOOSE	4 - 10
FIRM	4 - 8	25 - 50	COMPACT	10 - 30
STIFF	8 - 15	50 - 100	DENSE	30 - 50
VERY STIFF	15 - 30	100 - 200	VERY DENSE	> 50
HARD	> 30	> 200		
W.T.P.L. WETTER THAN PLASTIC LIMIT		D.T.P.L. DRIER THAN PLASTIC LIMIT		
A.P.L. ABOUT PLASTIC LIMIT				

TYPE OF SAMPLE

S.S	SPLIT SPOON	T.W.	THINWALL OPEN
W.S.	WASHED SAMPLE	T.P.	THINWALL PISTON
S.B.	SCRAPER BUCKET SAMPLE	O.S.	OESTERBERG SAMPLE
A.S.	AUGER SAMPLE	F.S.	FOIL SAMPLE
C.S.	CHUNK SAMPLE	R.C.	ROCK CORE
S.T.	SLOTTED TUBE SAMPLE		
	P.H. SAMPLE ADVANCED HYDRAULICALLY		
	P.M. SAMPLE ADVANCED MANUALLY		

SOIL TESTS

Qu	UNCONFINED COMPRESSION	L.V	LABORATORY VANE
Q	UNDRAINED TRIAXIAL	F.V	FIELD VANE
Qcu	CONSOLIDATED UNDRAINED TRIAXIAL	C	CONSOLIDATION
Qd	DRAINED TRIAXIAL		

▲,Δ - Undisturbed and remoulded shear strength determined from in situ vane test.

■ - Undrained shear strength determined from pocket penetrometer test.

LOG OF BOREHOLE NO. 380-1

N 5 010 790
E 278 735

PROJECT W.P. 290-97-00, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE Lawson Bay Road Overpass, Site 44-380 S
LOCATION Station 18+822 (Highway 69) Centreline SBL
BORING METHOD Continuous Flight Hollow Stem Augers

OUR PROJECT 97TF088A
ENGINEER M. R. Anderson
TECHNICIAN D. L. Watson

SOIL PROFILE				SAMPLES		SHEAR STRENGTH C_u				LIQUID LIMIT W_L			GROUNDWATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	DYNAMIC CONE PENETRATION \times STANDARD PENETRATION TEST \bullet				PLASTIC LIMIT W_P			
						BLOWS/0.3M				WATER CONTENT W			
						BLOWS/0.3M				WATER CONTENT W			
						BLOWS/0.3M				WATER CONTENT W			
0	GROUND ELEVATION 241.05					20	40	60	80	10	20	30	Upon completion of augering, free water at 0.75m.
-0.10	TOPSOIL : Dark brown silty clay, medium organic												
-1.12	SAND : Light brown silty sand, moist to wet		240										
1.5	BOREHOLE TERMINATED UPON REFUSAL TO AUGER AT 1.12m BEDROCK ASSUMED.		239										
3.0													
4.5													
6.0													
7.5													
9.0													
10.5													
12.0													
13.5													
15.0													
16.5													

NOTES:

CHECKED BY: *[Signature]*

LOG OF BOREHOLE NO. 380-2

N 5 010 813
E 278 725

PROJECT W.P. 290-97-00, HIGHWAY 69, DISTRICT 52, HUNTSVILLE

OUR PROJECT 97TF088A

SITE Lawson Bay Road Overpass, Site 44-380 S

LOCATION Station 18+846 (Highway 69) 7.0m Lt. of Centreline SBL

BORING DATE Feb. 28, 1998 ENGINEER M. R. Anderson

BORING METHOD Continuous Flight Hollow Stem Augers & NQ Rock Coring

TECHNICIAN D. L. Watson

SOIL PROFILE				SAMPLES				SHEAR STRENGTH C_u ▲				LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W W_P W W_L				GROUNDWATER OBSERVATIONS AND REMARKS
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N - VALUES	DYNAMIC CONE PENETRATION x STANDARD PENETRATION TEST ●				WATER CONTENT %					
							BLOWS/0.3M				20 40 60 80				10 20 30	
0	GROUND ELEVATION 242.53															
0.20	TOPSOIL : Dark brown silty clay, medium organic		242											Upon completion of augering, no free water, no cave.		
1.24	SAND : Reddish brown silty sand, damp, occasional boulder		241	1	RC		310	92	58	100						
1.5	BEDROCK : Hornblende Migmatite, with occasional layers of biotite migmatite		240	2	RC		1450	96	84	100						
3.0			239													
4.5			238	3	RC		1570	100	95	100						
4.57	BOREHOLE TERMINATED AT 4.57m		237				RUN (mm)	RECOVERY (%)	ROD (%)	DRILL WATER RETURN (%)						
6.0																
7.5																
9.0																
10.5																
12.0																
13.5																
15.0																
16.5																

NOTES:

CHECKED BY: *MA*

LOG OF BOREHOLE NO. 380-3

N 5 010 806
E 278 740

PROJECT W.P. 290-97-00, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE Lawson Bay Road Overpass, Site 44-380 S
LOCATION Station 18+838 (Highway 69) 7.0m Rt. of Centreline SBL
BORING METHOD Continuous Flight Hollow Stem Augers

OUR PROJECT 97TF088A
BORING DATE March 1, 1998 ENGINEER M. R. Anderson
TECHNICIAN D. L. Watson

SOIL PROFILE			SAMPLES		SHEAR STRENGTH C_u				LIQUID LIMIT W_L			GROUNDWATER OBSERVATIONS AND REMARKS		
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N - VALUES	DYNAMIC CONE PENETRATION x STANDARD PENETRATION TEST • BLOWS/0.3M				WATER CONTENT %			
							20	40	60	80	W_P		W	W_L
0	GROUND ELEVATION 241.94													
0.15	TOPSOIL : Dark brown silty clay, medium organic													
	BOREHOLE TERMINATED UPON REFUSAL TO AUGER AT 0.15m BEDROCK ASSUMED.		241										Upon completion of augering, no free water, no cave.	
1.5														
3.0														
4.5														
6.0														
7.5														
9.0														
10.5														
12.0														
13.5														
15.0														
16.5														

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LOG OF BOREHOLE NO. 380-4

N 5 010 826
E 278 724

PROJECT W.P. 290-97-00, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE Lawson Bay Road Overpass, Site 44-380 S
LOCATION Station 18+859 (Highway 69) 7.0m Lt. of Centreline SBL
BORING METHOD Continuous Flight Hollow Stem Augers

OUR PROJECT 97TF088A
ENGINEER M. R. Anderson
TECHNICIAN D. L. Watson

SOIL PROFILE				SAMPLES			SHEAR STRENGTH C_u ▲			LIQUID LIMIT W_L PLASTIC LIMIT W_p WATER CONTENT W W_p W W_L			GROUNDWATER OBSERVATIONS AND REMARKS	
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N - VALUES	DYNAMIC CONE PENETRATION x STANDARD PENETRATION TEST ●				WATER CONTENT %			
							BLOWS/0.3M				20 40 60 80			10 20 30
0	GROUND ELEVATION 242.48													
0.60	<u>SAND AND GRAVEL FILL</u> : Dark brown sand and gravel, trace of silt, damp		242											
0.90	<u>SILT</u> : Brown sandy silt, moist		241											
1.5	BOREHOLE TERMINATED UPON REFUSAL TO AUGER AT 0.90m BEDROCK ASSUMED.												Upon completion of augering, no free water, no cave.	
3.0														
4.5														
6.0														
7.5														
9.0														
10.5														
12.0														
13.5														
15.0														
16.5														

NOTES:

CHECKED BY: *MD*

LOG OF BOREHOLE NO. 380-5

N 5 010 819
E 278 739

PROJECT W.P. 290-97-00, HIGHWAY 69, DISTRICT 52, HUNTSVILLE

OUR PROJECT 97TF088A

SITE Lawson Bay Road Overpass, Site 44-380 S

LOCATION Station 18+851 (Highway 69) 7.0m Rt. of Centreline SBL

BORING DATE Mar. 1, 1998 ENGINEER M. R. Anderson

BORING METHOD Continuous Flight Hollow Stem Augers & NQ Rock Coring

TECHNICIAN D. L. Watson

SOIL PROFILE				SAMPLES		SHEAR STRENGTH C_u ▲				LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W			GROUNDWATER OBSERVATIONS AND REMARKS	
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N - VALUES	DYNAMIC CONE PENETRATION x STANDARD PENETRATION TEST ●				WATER CONTENT %			
							BLOWS/0.3M				10 20 30			
0	GROUND ELEVATION 242.35						20	40	60	80				Upon completion of augering, no free water, no cave. * Low RQD from casing disturbance
0.05	TOPSOIL : Dark brown silty clay		242											
0.76	SAND : Brown silty sand, with numerous cobbles and boulders		241	1	RC		920	100	67*	100				
1.5	BEDROCK : Hornblende Migmatite		240	2	RC		1520	100	93	100				
3.0			239											
3.96				3	RC		760	100	100	100				
4.5	BOREHOLE TERMINATED AT 3.96m		238											
6.0														
7.5														
9.0														
10.5														
12.0														
13.5														
15.0														
16.5														

NOTES:

CHECKED BY: *[Signature]*

LOG OF BOREHOLE NO. 380-6

N 5 010 842
E 278 730

PROJECT W.P. 290-97-00, HIGHWAY 69, DISTRICT 52, HUNTSVILLE

OUR PROJECT 97TF088A

SITE Lawson Bay Road Overpass, Site 44-380 S

LOCATION Station 18+875 (Highway 69) Centreline SBL

BORING DATE Feb. 28, 1998 ENGINEER M. R. Anderson

BORING METHOD Continuous Flight Hollow Stem Augers

TECHNICIAN D. L. Watson

SOIL PROFILE			SAMPLES		SHEAR STRENGTH C_u				LIQUID LIMIT W_L				GROUNDWATER OBSERVATIONS AND REMARKS		
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N - VALUES	DYNAMIC CONE PENETRATION x STANDARD PENETRATION TEST • BLOWS/0.3M				PLASTIC LIMIT W_P WATER CONTENT W WATER CONTENT %				
							20	40	60	80	10	20		30	
0	GROUND ELEVATION 244.59														
0.10	TOPSOIL : Dark brown silty clay, medium organic		244											Upon completion of augering, no free water, no cave.	
1.5	BOREHOLE TERMINATED UPON REFUSAL TO AUGER AT 0.10m BEDROCK ASSUMED.														
3.0															
4.5															
6.0															
7.5															
9.0															
10.5															
12.0															
13.5															
15.0															
16.5															

NOTES:

CHECKED BY: *[Signature]*

LOG OF BOREHOLE NO. 380-7

N 5 010 770
E 278 776

PROJECT W.P. 290-97-00, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE Lawson Bay Road Overpass, Site 44-380 N
LOCATION Station 18+801 (Highway 69) Centreline NBL
BORING METHOD Continuous Flight Hollow Stem Augers

OUR PROJECT 97TF088A
BORING DATE Feb. 28, 1998 ENGINEER M. R. Anderson
TECHNICIAN D. L. Watson

SOIL PROFILE			SAMPLES		SHEAR STRENGTH C_u				LIQUID LIMIT W_L			GROUNDWATER OBSERVATIONS AND REMARKS		
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N - VALUES	DYNAMIC CONE PENETRATION x STANDARD PENETRATION TEST •				WATER CONTENT %			
							BLOWS/0.3M				WATER CONTENT %			
							20	40	60	80	W_p		W	W_L
0	GROUND ELEVATION 242.68													
0.08	TOPSOIL : Dark brown silty clay, medium organic		242											
	BOREHOLE TERMINATED UPON REFUSAL TO AUGER AT 0.08m BEDROCK ASSUMED.											Upon completion of augering, no free water, no cave.		
1.5														
3.0														
4.5														
6.0														
7.5														
9.0														
10.5														
12.0														
13.5														
15.0														
16.5														

NOTES:

CHECKED BY: *ms*

LOG OF BOREHOLE NO. 380-8

N 5 010 793
E 278 766

PROJECT W.P. 290-97-00, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE Lawson Bay Road Overpass, Site 44-380 N
LOCATION Station 18+825 (Highway 69) 7.0m Lt. of Centreline NBL BORING DATE Feb. 28, 1998
BORING METHOD Continuous Flight Hollow Stem Augers

OUR PROJECT 97TF088A
ENGINEER M. R. Anderson
TECHNICIAN D. L. Watson

SOIL PROFILE				SAMPLES		SHEAR STRENGTH C_u ▲				LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W			GROUNDWATER OBSERVATIONS AND REMARKS	
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N - VALUES	DYNAMIC CONE PENETRATION x STANDARD PENETRATION TEST ●				WATER CONTENT %			
							BLOWS/0.3M							
							20	40	60	80	10	20		30
0	GROUND ELEVATION 243.43												Upon completion of augering, no free water, no cave.	
0.30	TOPSOIL : Dark brown silty clay, medium organic		243											
	BOREHOLE TERMINATED UPON REFUSAL TO AUGER AT 0.30m BEDROCK ASSUMED.													
1.5														
3.0														
4.5														
6.0														
7.5														
9.0														
10.5														
12.0														
13.5														
15.0														
16.5														

NOTES:

CHECKED BY: *DLW*

LOG OF BOREHOLE NO. 380-9

N 5 010 780
E 278 787

PROJECT W.P. 290-97-00, HIGHWAY 69, DISTRICT 52, HUNTSVILLE

OUR PROJECT 97TF088A

SITE Lawson Bay Road Overpass, Site 44-380 N

LOCATION Station 18+817 (Highway 69) 7.0m Rt. of Centreline NBL BORING DATE Feb. 28, 1998 ENGINEER M. R. Anderson

BORING METHOD Continuous Flight Hollow Stem Augers & NQ Rock Coring

TECHNICIAN D. L. Watson

SOIL PROFILE				SAMPLES		SHEAR STRENGTH C_u				LIQUID LIMIT W_L			GROUNDWATER OBSERVATIONS AND REMARKS	
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N - VALUES	DYNAMIC CONE PENETRATION x STANDARD PENETRATION TEST				WATER CONTENT %			
							BLOWS/0.3M				WATER CONTENT %			
							20	40	60	80	10	20		30
0	GROUND ELEVATION 242.61													
0.08	TOPSOIL : Dark brown silty clay, medium organic		242										150mm of snowmelt water at surface	
1.07	CLAY : Soft to firm, light brown silty clay, some sand, medium plastic, W.T.P.L.		241	1	RC		350	100	0*	100			* Low RQD due to casing disturbance	
1.5	BEDROCK : Biotite Migmatite		240	2	RC		1450	98	77	100				
3.0			239											
4.5	BOREHOLE TERMINATED AT 4.47m		238	3	RC		1600	97	90	100				
4.47							RUN (mm)	RECOVERY (%)	RQD (%)	DRILL WATER RETURN (%)				
6.0														
7.5														
9.0														
10.5														
12.0														
13.5														
15.0														
16.5														

NOTES:

CHECKED BY: *[Signature]*

LOG OF BOREHOLE NO. 380-10

N 5 010 806
E 278 764

PROJECT W.P. 290-97-00, HIGHWAY 69, DISTRICT 52, HUNTSVILLE

OUR PROJECT 97TF088A

SITE Lawson Bay Road Overpass, Site 44-380 N

LOCATION Station 18+838 (Highway 69) 7.0m Lt. of Centreline SBL

BORING DATE Mar. 1, 1998 ENGINEER M. R. Anderson

BORING METHOD Continuous Flight Hollow Stem Augers

TECHNICIAN D. L. Watson

SOIL PROFILE				SAMPLES		SHEAR STRENGTH C_u ▲				LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W			GROUNDWATER OBSERVATIONS AND REMARKS	
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N - VALUES	DYNAMIC CONE PENETRATION x STANDARD PENETRATION TEST ●				WATER CONTENT %			
							BLOWS/0.3M				WATER CONTENT %			
							20	40	60	80	10	20		30
0	GROUND ELEVATION 242.72												Upon completion of augering, no free water, no cave.	
0.08	TOPSOIL : Dark brown silty clay, medium organic		242											
0.86	SAND : Light brown silty sand													
1.5	BOREHOLE TERMINATED UPON REFUSAL TO AUGER AT 0.86m BEDROCK ASSUMED.		241											
3.0														
4.5														
6.0														
7.5														
9.0														
10.5														
12.0														
13.5														
15.0														
16.5														

Upon completion
of augering,
no free water,
no cave.

NOTES:

CHECKED BY: *[Signature]*

LOG OF BOREHOLE NO. 380-11

N 5 010 800
E 278 779

PROJECT W.P. 290-97-00, HIGHWAY 69, DISTRICT 52, HUNTSVILLE

OUR PROJECT 97TF088A

SITE Lawson Bay Road Overpass, Site 44-380 N

LOCATION Station 18+830 (Highway 69) 7.0m Rt. of Centreline NBL **BORING DATE** Feb. 28, 1998 **ENGINEER** M. R. Anderson

BORING METHOD Continuous Flight Hollow Stem Augers & NQ Rock Coring

TECHNICIAN D. L. Watson

SOIL PROFILE			SAMPLES			SHEAR STRENGTH C_u ▲				LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W W_P — W — W_L			GROUNDWATER OBSERVATIONS AND REMARKS	
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3m N - VALUES	DYNAMIC CONE PENETRATION x STANDARD PENETRATION TEST •				WATER CONTENT %			
							BLOWS/0.3M							
	GROUND ELEVATION 242.30						20	40	60	80	10	20	30	
0	-0.08	TOPSOIL : Dark brown silty clay, medium organic	242											
	1.20	SAND : Light brown silty sand, saturated	241											
1.5	-1.65	CLAY : Stiff, grey silty clay, some sand, medium plastic, D.T.P.L.		1	SS		6	100mm & bouncing						
	2.21	bouldery (cored)	240	2	RC		200	88	0*	100				
3.0		BEDROCK : Granitic Pegmatite	239	3	RC		1580	98	95	100				
	3.53	Biotite Migmatite												
			238											
4.5														
	4.78	Granitic Pegmatite		4	RC		1620	92	81	100				
			237											
	5.61	BOREHOLE TERMINATED AT 5.61m												
6.0			236				RUN (mm)	RECOVERY (%)	RQD (%)	DRILL WATER RETURN (%)				
7.5														
9.0														
10.5														
12.0														
13.5														
15.0														
16.5														

Upon completion of augering, free water at 0.28m.
* Low RQD due to casing disturbance

Upon completion
of augering, free
water at 0.28m.
* Low RQD due to
casing disturbance

NOTES:

CHECKED BY: *MA*

LOG OF BOREHOLE NO. 380-12

N 5 010 823
E 278 769

PROJECT W.P. 290-97-00, HIGHWAY 69, DISTRICT 52, HUNTSVILLE
SITE Lawson Bay Road Overpass, Site 44-380 N
LOCATION Station 18+854 (Highway 69) Centreline NBL
BORING METHOD Continuous Flight Hollow Stem Augers

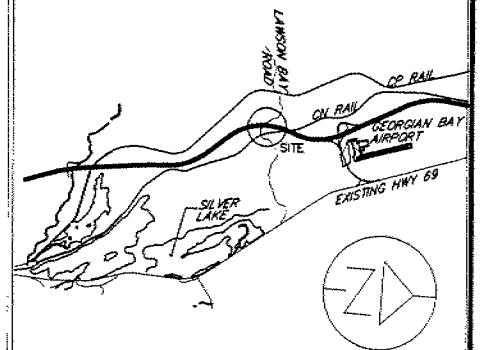
OUR PROJECT 97TF088A
ENGINEER M. R. Anderson
TECHNICIAN D. L. Watson

SOIL PROFILE			SAMPLES		SHEAR STRENGTH C_u				LIQUID LIMIT W_L			GROUNDWATER OBSERVATIONS AND REMARKS		
DEPTH in METRES	DESCRIPTION	LEGEND	ELEVATION	NUMBER	TYPE	BLOWS/0.3M N - VALUES	DYNAMIC CONE PENETRATION x STANDARD PENETRATION TEST •				WATER CONTENT %			
							BLOWS/0.3M				WATER CONTENT %			
							20	40	60	80	W_p		W	W_L
0	GROUND ELEVATION 241.63													
-0.10	TOPSOIL : Dark brown silty clay, medium organic		241										Upon completion of augering, no free water, no cave.	
	BOREHOLE TERMINATED UPON REFUSAL TO AUGER AT 0.10m BEDROCK ASSUMED.													
1.5														
3.0														
4.5														
6.0														
7.5														
9.0														
10.5														
12.0														
13.5														
15.0														
16.5														

NOTES:

CHECKED BY: *[Signature]*

DISTRICT MUNICIPALITY OF PARRY SOUND
GEOG TWP CONGER



KEY PLAN

0.5 km 1 km

BOREHOLE	LOCATION	ELEVATION
380-1	N 5 010 790 E 278 735	241.05
380-2	N 5 010 813 E 278 725	242.53
380-3	N 5 010 806 E 278 740	241.94
380-4	N 5 010 826 E 278 724	242.48
380-5	N 5 010 819 E 278 739	242.35
380-6	N 5 010 842 E 278 730	244.59



LEGEND
 BOREHOLE
 OBSERVED WATER LEVEL
 (DURING OR UPON COMPLETION OF DRILLING)
NOTE
 1. REFER TO LOG OF BOREHOLE SHEETS FOR DETAILED SUBSURFACE CONDITIONS
 2. THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT BOREHOLE LOCATIONS. BETWEEN BOREHOLES, THE BOUNDARIES ARE ASSUMED FROM GEOLOGICAL EVIDENCE.

MINISTRY OF TRANSPORTATION
ENGINEERING AND RIGHT OF WAY OFFICE
SURVEYS AND PLANS SECTION

PROPOSED CROSSING

AT

LAWSON BAY ROAD

AND

KING'S HIGHWAY 69 SOUTH LANES

DISTRICT MUNICIPALITY OF PARRY SOUND CON 9

LOT 4 GEOG TWP CONGER TWP OF HUMPHREY

SCALE AS SHOWN DISTRICT 52 HUNTSVILLE REGION NORTHERN
 WP/WO 290-97-00 PROFILE C-790-69-108 PLAN 8-774-69-044

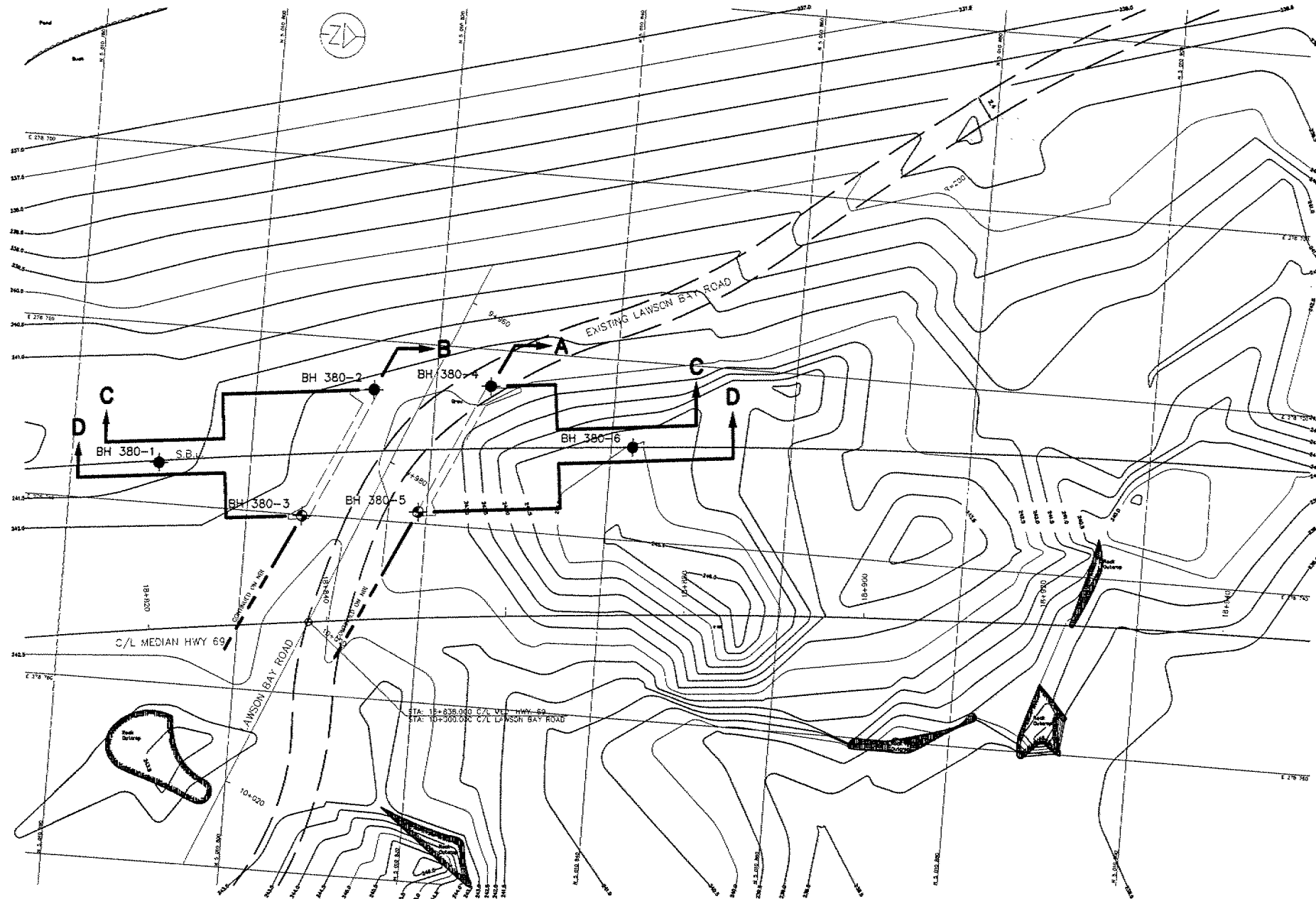
SURVEY 97 12 PLAN 97 12

SITE 44-380 PLAN E-774-69-107

Peto MacCallum Ltd.
CONSULTING ENGINEERS
45 BURNFORD ROAD HAMILTON, ONTARIO L8E 3C4

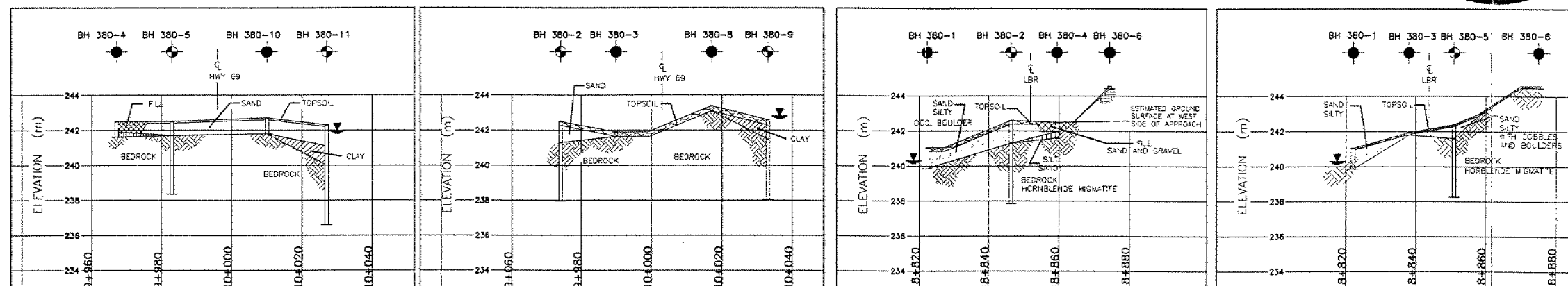
DRAWN JS DATE SCALE JOB NO. DRAWING NO.
 CHECKED MRA JUNE 1998 AS SHOWN 97TF088A 1
 APPROVED DWH

BOREHOLE LOCATION PLAN
AND SOIL PROFILES



BOREHOLE LOCATION PLAN

SCALE 1:500



SECTION A-A

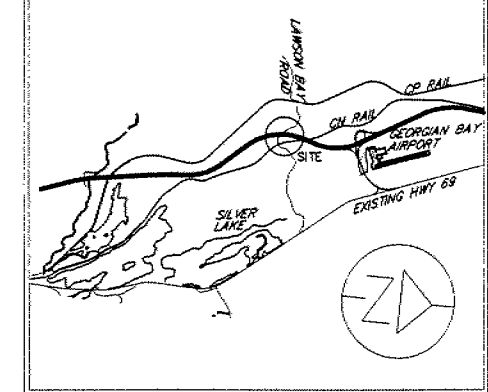
SECTION B-B

SECTION C-C

SECTION D-D

SOIL PROFILES

SCALE: VERTICAL 1:500
HORIZONTAL 1:1500



KEY PLAN
 0.5km 0 1km

BOREHOLE	LOCATION	ELEVATION
380-7	N 5 010 770 E 278 776	242.68
380-8	N 5 010 793 E 278 766	243.43
380-9	N 5 010 780 E 278 787	242.61
380-10	N 5 010 806 E 278 764	242.72
380-11	N 5 010 800 E 278 779	242.30
380-12	N 5 010 823 E 278 769	241.63



LEGEND
 BOREHOLE AND ROCK CORE
 OBSERVED WATER LEVEL
 (DURING OR UPON COMPLETION OF DRILLING)
NOTE
 1. REFER TO LOG OF BOREHOLE SHEETS FOR DETAILED SUBSURFACE CONDITIONS.
 2. THE BOUNDARIES BETWEEN SOIL STRATA HAVE BEEN ESTABLISHED ONLY AT BOREHOLE LOCATIONS. BETWEEN BOREHOLES, THE BOUNDARIES ARE ASSUMED FROM GEOLOGICAL EVIDENCE.

MINISTRY OF TRANSPORTATION
 ENGINEERING AND RIGHT OF WAY OFFICE
 SURVEYS AND PLANS SECTION

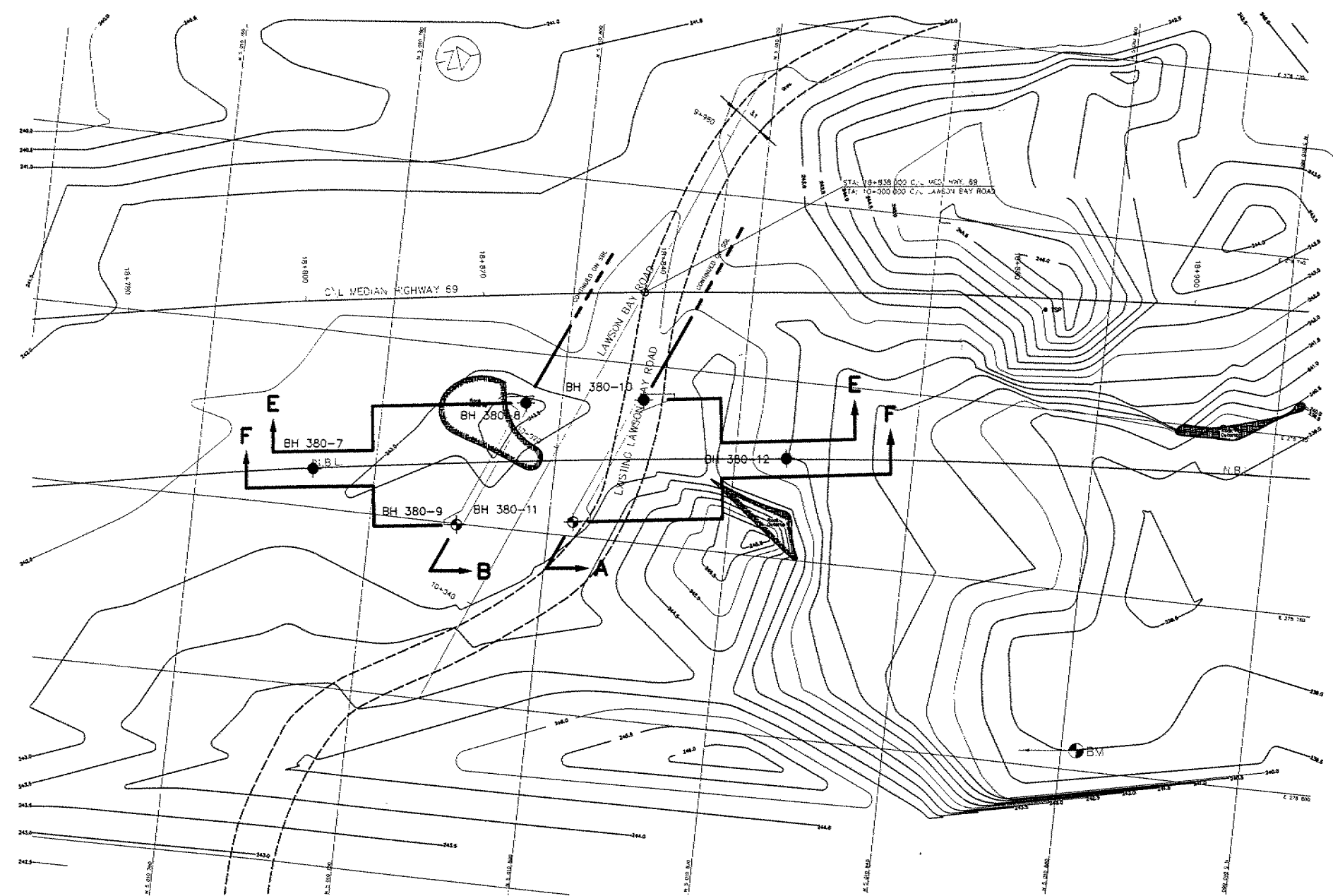
PROPOSED CROSSING
 AT
 LAWSON BAY ROAD
 AND
 KING'S HIGHWAY 69 NORTH LANES
 DISTRICT MUNICIPALITY OF PARRY SOUND
 LOT 4 GEOG TWP CONGR CON 2 TWP OF HUMPHREY

SCALE AS SHOWN	DISTRICT 52 HUNTSVILLE	REGION NORTHERN
WP/VO 290-97-00	PROFILE C-790-69-106	PLAN B-774-69-044
SURVEY 97 12	PLAN 97 12	
SITE 44-380	PLAN E-774-69-105	

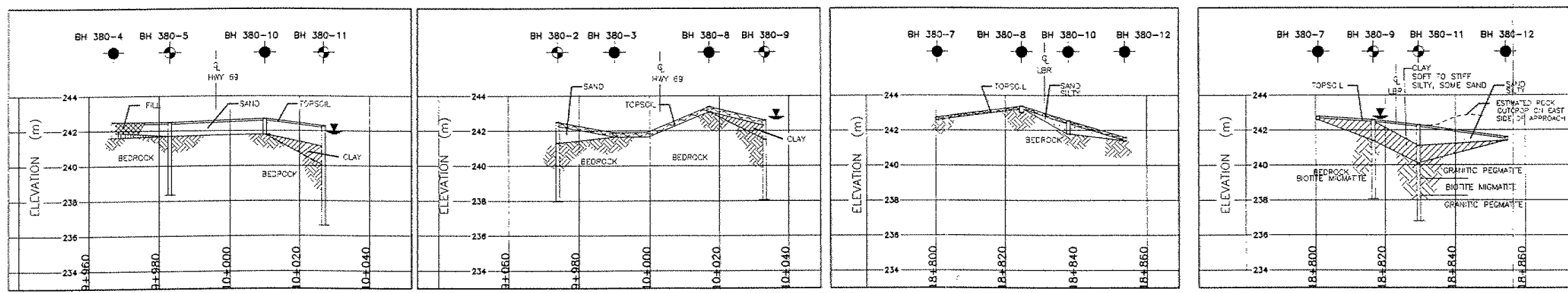
Peto MacCallum Ltd.
 CONSULTING ENGINEERS
 45 BURNHAM ROAD, HAMILTON, ONTARIO L8E 3J7

DRAWN JS	DATE JUNE 1998	SCALE AS SHOWN	JOB NO. 97TF088A	DRAWING NO. 2
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BOREHOLE LOCATION PLAN
 AND SOIL PROFILES



BOREHOLE LOCATION PLAN
 SCALE 1:800



SECTION A-A

SECTION B-B

SOIL PROFILES
 SCALE: VERTICAL 1:500
 HORIZONTAL 1:1500

SECTION E-E

SECTION F-F

FOUNDATION DESIGN REPORT
FOR
LAWSON BAY ROAD OVERPASS
W.P. 290-97-00, SITE 44-380 N & S
HIGHWAY 69, DISTRICT 52
HUNTSVILLE, ONTARIO

Distribution:

15 cc: McCormick Rankin Corporation
1 cc: PML Hamilton
1 cc: PML Toronto
1 cc: PML Barrie

Job No. 97TF088A

June, 1998

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FOUNDATION DESIGN REPORT

For

Lawson Bay Road Overpass
W.P. 290-97-00, Site 44-380 N & S
Highway 69, District 52, Huntsville

INTRODUCTION

This report provides geotechnical comments and recommendations regarding design and construction of foundations, abutments and approaches at the proposed Lawson Bay Road overpass structures at Highway 69.

Construction of two single span overpass structures is planned. At the overpass location, the proposed four-lane Highway 69 will be constructed approximately 3.0 m above the existing ground surface (road grade at elevation 245 to 246). Road grades on Lawson Bay Road under the structures will be near elevation 239, some 4 m below existing grade (based on preliminary grade information (Sheets 14 and 24 of the Environmental Assessment/Route Planning Study, W.P. 529-89-00) and existing ground surface elevations determined at borehole locations).

The subsurface stratigraphy revealed at the overpass site generally comprised a surficial topsoil layer overlying discontinuous sand/silt/clay deposits, mantling bedrock. Bedrock/inferred bedrock was contacted at depths of 0.1 to 2.2 m.

FOUNDATIONS

Spread Footings

Based on the borehole information, it is considered that the structure may be supported on conventional spread footings founded on bedrock. Foundations bearing on the sound bedrock

at/below elevations 241.3 to 241.8 (southbound lanes) and 240.1 to 243.1 (northbound lanes) may be designed using a factored bearing resistance of 10,000 kPa at the ultimate limit state.

The capacity at serviceability limit states normally allows for 25 mm of compression of the foundation and founding medium. Considering the bedrock to be non-yielding, the design is not expected to be governed by settlement since the loading required to produce deformation will be much larger than the factored capacity at ULS.

The surface of the bedrock below the footing should be benched or socketed to provide a level founding surface. In general, the founding elevation between footings should be stepped at a maximum inclination of 1 horizontal to 1 vertical.

We note that a bedrock cut of about 3 m will be required to establish the proposed vertical alignment of Lawson Bay Road. The footings should be founded below the level of any bedrock that has been disturbed by rock excavation activities, and below a line inclined upwards at 1 : 2 (H : V) from the base of the excavation.

Footings bearing on sound bedrock should not require protection from frost.

Prior to placement of structural concrete, all foundation excavations should be examined by qualified geotechnical personnel to verify the competency of the founding surface.

Alternative Designs

This site is not considered suitable for construction of an integral abutment bridge due to the shallow depth to bedrock. We understand that perched abutments supported on structural approach fill are also not suitable.

ABUTMENT WALLS

The abutment walls should be designed to resist the unbalanced lateral earth pressure imposed by the backfill adjacent to the wall. The lateral earth pressure, p , may be computed using the equivalent fluid pressures presented in Section 6-7.4 of the Ontario Highway Bridge Design Code (OHBDC, 3rd Edition, 1991) or employing the following equation, assuming a triangular pressure distribution:

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

where K = coefficient of lateral earth pressure

γ = unit weight of free-draining
granular material

h = depth below final grade (m)

q = surcharge load (kPa), if present

Free-draining granular material or rock fill should be used as backfill behind the wall. The following parameters are recommended for design:

	Granular "A"	Granular "B"	Rock Fill
Angle of Internal Friction (degrees)	35	32	35
Unit Weight (kN/m ³)	22.8	21.2	18.0
Active Earth Pressure Coefficient (K_a)	0.27	0.31	0.27
At Rest Earth Pressure Coefficient (K_o)	0.43	0.47	0.43
Passive Earth Pressure Coefficient (K_p)	3.69	3.25	3.69

A weeping tile system and/or weeping holes should be installed to minimize the build-up of hydrostatic pressure behind the wall. The weeping tiles should be surrounded by a properly

designed granular filter or geotextile to prevent migration of fines into the system. The drainage pipe should be placed on a positive grade and lead to a frost-free outlet.

If spread footings are employed, the horizontal force will be resisted in part by the friction force developed between the underside of footing and the bedrock. An unfactored friction factor of 0.6 is recommended for footings on bedrock.

We understand that installation of dowels into bedrock is being considered to increase the lateral resistance of footings founded on bedrock. The increased lateral resistance will be provided by the increased sliding resistance developed at the interface between the footing and rock due to the increased vertical pressure created by the stress in the anchor. A factored rock-grout bond stress of 1.4 MPa at the ultimate limit state (resistance factor of 0.4 applied) is recommended for design of rock anchors. The anchors should extend a minimum 30 bar diameters into sound bedrock.

APPROACH FILL

Backfilling adjacent to the structures should be carried out in conformance with Ontario Provincial Standards specifications for granular or rock backfill.

Recommendations for approach construction are presented in the Pavement Design Report.

EXCAVATION AND GROUNDWATER CONTROL

Excavation for construction of footings is expected to be carried out within the thin surficial overburden deposits and bedrock.

Excavation of the overburden should be relatively straightforward. Large boulders may be encountered. The overburden is classified as a Type 3 soil according to Occupational Health and Safety Act criteria; sidewalls in temporary excavations exceeding 1.2 m depth should be inclined at 1 horizontal to 1 vertical.

It is anticipated that groundwater seepage or surface water entering the excavation will be handled readily by conventional sump pumping.

All work should be carried out in accordance with the Occupational Health and Safety Act (Ontario Regulation 213/91) and with local/MTO regulations.

CLOSURE

This report was written by M.R. Anderson, Project Engineer and reviewed by D.W. Kerr, Manager of Geotechnical and Geo-Environmental Services, Hamilton.

Yours very truly

Peto MacCallum Ltd.



Murray R. Anderson, M.Eng., P.Eng.
Project Engineer

A handwritten signature in black ink, appearing to read "M. R. Anderson", written over a horizontal line.



Dennis W. Kerr, M.Eng., P.Eng.
Manager Geotechnical and
Geo-Environmental Services
Hamilton

A handwritten signature in black ink, appearing to read "D. W. Kerr", written over a horizontal line.

MRA:mmma

PAVEMENT DESIGN REPORT
FOR
LAWSON BAY ROAD OVERPASS
W.P. 290-97-00, SITE 44-380 N & S
HIGHWAY 69, DISTRICT 52
HUNTSVILLE, ONTARIO

Distribution:

15 cc: McCormick Rankin Corporation
1 cc: PML Hamilton
1 cc: PML Barrie
1 cc: PML Toronto

Job No. 97TF088E

June, 1998

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PAVEMENT DESIGN REPORT
For
Lawson Bay Road Overpass
W.P. 290-97-00, Site 44-380 N & S
Highway 69, District 52, Huntsville

INTRODUCTION

The Lawson Bay Road overpass structures are being constructed as an advance contract to the overall 26.5 km four laning of Highway 69, W.P. 290-97-00. The following roadworks are associated with the advance construction of the bridges:

- 50 m approaches to future north and south bound lanes of Highway 69, approximate Station 18+790 to Station 18+900, (Conger Twp.).
- Lawson Bay Road, Revised Alignment (Intersects Hwy 69 centreline median at Station 18+838.000), approximate Station 9+875 to Station 10+190.

It is understood the bridge structures and approach fills on Hwy 69 will be constructed, but no granulars or paving will be involved. Lawson Bay Road will be reconstructed (unpaved) and would provide access for subsequent construction projects.

This report presents the geotechnical investigation and survey data as well as recommendations pertaining to the roadworks associated with the advance contract.

DESIGN CRITERIA

The approved Design Criteria for the overall project was provided with a memorandum dated March 26, 1998, from Mr. P. Lecoarer, P. Eng., Senior Project Engineer, Planning and Design Section, MTO Northern Region.

Documents showing the proposed design standards and traffic volumes for Highway 69 and the proposed design standards for Lawson Bay Road are extracted from the approved Design Criteria and provided in Appendix A. No traffic volumes are provided for Lawson Bay Road.

Lawson Bay Road (unpaved) will be used as access for subsequent construction projects.

PHYSIOGRAPHY AND GEOLOGY

The report area is part of the Precambrian Laurentian peneplane. Although the general surface of the country is relatively flat the topography is quite irregular in detail and the area is dotted with many small lakes separated by rocky ridges. Soil cover is generally sparse. The region is well wooded. Swamp environments have generally developed within relatively low lying, poorly drained areas.

The site is underlain by granitic and metamorphic gneisses, and in many places the structural alignment of the gneisses influences the topography. In this regard, the rivers and lakes underlain by the gneisses of the Moon River syncline in Conger and Freeman Townships follow the foliation trends of the gneisses.

The bedrock formations are of Precambrian age and are largely composed of veined, banded, and homogeneous pink and grey migmatitic gneisses produced by injection and granitization of metamorphic gneisses of various types.

A frost penetration depth of 1.6 m has been provided by MTO for this project.

INVESTIGATION PROCEDURES

The geotechnical investigation for the advance contract was carried out as part of the overall project, which was conducted during the period December 1997 to April 1998.

The investigation consisted of test holes put down using hand auger, backhoe and/or power auger. The test hole program was carried out in accordance with the requirements of MTO Northern Region Pavement Design Practices And Guidelines (May 20, 1997), involving test holes along centreline of pavement and offsets left and right.

Soil samples were recovered from each test hole for field identification. Representative samples were returned to our laboratory for detailed examination and moisture content determinations.

The test holes were referred horizontally to centreline median for Highway 69, and centreline for Lawson Bay Road, as staked out in the field by Totten Sims Hubicki Associates. Elevations were established relative to the ground surface at the control line.

SUBSURFACE CONDITIONS

Reference is made to Appendix B for the geotechnical survey data collected within and adjacent to the limits of the advance contract. Reference is also made to the test holes drilled in connection with the foundation investigation for the proposed structures.

Subsurface conditions comprised relatively thin discontinuous overburden consisting of surficial topsoil (up to 800 mm thick) over localized sand or silt with sand. Bedrock was exposed locally, or encountered beneath the overburden typically at depths of less than 1 m (2.5 m locally). The soils were moist to wet.

A swamp exists immediately north of the advance contract project area.

RECOMMENDATIONS

Pavement Structure

It is understood that only the approach fills for Hwy 69 will be constructed. No granulars or paving will be involved.

Lawson Bay Road will be gravel surface (unpaved). For the anticipated bedrock subgrade provide a minimum 200 mm of Granular "A". A minimum 300 mm rock shatter should be provided.

The above granular depth would be suitable for construction traffic. Provision should be made for fine grading after the construction period.

Conversion Factors

Use the following:

Granular "A" 2.4 t/m³

Cut Sections

The proposed profile for Lawson Bay Road was not finalised at the time of this report. However, based on the original alignment shown in the Preliminary Design Report, up to 5 m cut could be expected. The excavation will occur within bedrock with shallow (less than 1 m) overburden cover. The limits of rock cut, including earth pockets, will be determined when the final road grades are established.

Rock Cut

Rock excavation will require standard rock excavation techniques including blasting. Refer to OPSD 201.01.

Rock Cuts With Earth Pockets

Treat rock cuts with earth pockets at grade as follows:

- Excavate earth and rock to the full depth of earth pocket, or to a maximum 1.6 m below the profile grade, whichever is less, for the full roadway width.
- Backfill with rock fill or granular material.
- Provide for positive drainage from excavated earth pockets along the full length of the pockets.

There is the potential for drainage from earth pocket excavation areas at or near the limit of a cut to be blocked by local rock knobs. Remove rock knobs to

sufficient depth (maximum 1.6 m below profile grade) to ensure adequate drainage.

Transition Treatment

Transition zones should be treated in accordance with applicable OPSD 205.01 to 205.05. Use $t = 1.6$ m for transition treatment depth. Topsoil ranges between 100 and 800 mm depth. Use $H = 300$ mm. Topsoil will form part of the grubbing quantity and will not be available for reuse.

Disposal of Cut Material

Excavated rock will be used as embankment material. Excavated overburden will be generally unsuitable for use as embankment material, but may be utilized for slope flattening.

Slope Treatment

No particular treatment is specified for rock cuts. It is recommended however, that the exposed rock faces be examined visually for any planes of weakness which should be investigated/addressed on an as required basis during construction.

Fill Sections

Up to 6 to 7 m of fill is required for construction of the structure approaches along Hwy. 69 north and south bound lanes.

The investigation indicates the proposed fill sections will be founded on bedrock or shallow sand or silt (less than 1 m) over bedrock, where no major construction problems are anticipated.

It is anticipated that the embankment will be constructed with rock fill material generated from sections cut through bedrock at the site. Refer to OPSD 201.02 and 202.01.

A borrow source of rock fill, if required, had not been identified at the time of this report.

For high rock fills, MTO Northern Region practice is to provide 2.0 m wide berms so that no uninterrupted rock fill slope is greater than 6.0 m high. For this site, provide berm at mid-height of slope.

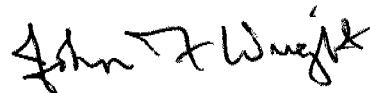
Where slope flattening is proposed, provide drainage gap in accordance with OPSD 202.02. Where slopes are flattened to eliminate the need for guide rail, provide granular infilled drainage gap in accordance with Northern Region practice, refer to Appendix C.

CLOSURE

The field investigation was carried out under the direction and supervision of Mr. J. F. Wright, B.Sc., Senior Geologist and Mr. E. Wong, P. Eng. This report was prepared by Mr. J. F. Wright and reviewed by Mr. T. Lee-Bun, P.Eng. Manager, Geotechnical Engineering, Barrie office.

Yours very truly

Peto MacCallum Ltd.



John F. Wright, B.Sc.
Senior Geologist



JFW:mmma



Turney Lee-Bun, P.Eng.
Manager, Geotechnical Engineering
Barrie

APPENDIX A

EXTRACT FROM DESIGN CRITERIA

Ministry of
Transportation

Ontario

DESIGN CRITERIA

Page 1 of 1

P.03

Date:

February 1998

Reg.Rev.Date:

GROUP WORK PROJECT 290-97-00 Dist. No. 52 Hwy. No. 69

TYPE OF PROJECT Grading, Drainage, Granular Base, Hot Mix Paving, Structures, Partial Illumination

LOCATION From South of Tower Road Northerly to 2.7 km North of Hwy 141 LENGTH 26.6 km

LIMITS FROM STA 11+000 PLAN 790-69/5-0 TO STA 14+900 PLAN 451-69/122-0

HIGHWAY CLASSIFICATION

MINIMUM STOPPING SIGHT DISTANCE

EQUIVALENT MINIMUM "K" FACTOR

GRADES MAXIMUM

MINIMUM RADIUS

PAVEMENT WIDTH

SHOULDER WIDTH

SHOULDER ROUNDING

MEDIAN WIDTH

ROW WIDTH

POSTED SPEED

MISCELLANEOUS

PRESENT CONDITIONS	DESIGN STANDARDS	PROPOSED STANDARDS
	RFD 120	RFD 120
	245 m	300 m
	K = 120 Crest; 60 Sag	K = 180 Crest; 100 Sag
	3%	2%
	650 m	1000 m
	4 @ 3.75 m	4 @ 3.75 m
	1.0 m Lt 3.0 m Rt	1.0 m Lt (a) 3.0 m Rt
	1.0 m	1.0 m
	30 m	30 m
	100 m	100 m (b)
	100 km/h	100 km/h

RECOMMENDED BY:

P. Eng.

PROJECT MANAGER
HIGHWAY 88 JOINT VENTURE (MAG/TSN)

DISTRICT ENGINEER

MANAGER, CONSTRUCTION OFFICE

APPROVED BY:

REGIONAL MANAGER

ENGINEERING OFFICE

98/03/25

DATE OF APPROVAL

TRAFFIC DATA: See page No. 2

Ministry of
Transportation

Ontario

DESIGN CRITERIA

Page 2 of 1

P.04

Date:

February 1998

Reg. Rev. Date:

GROUP WORK PROJECT 290-97-00 Dist. No. 52 Hwy. No. 69

TYPE OF PROJECT Grading, Drainage, Granular Base, Hot Mix Paving, Structures, Partial Illumination

LOCATION From South of Tower Road Northerly to 2.7 km North of Hwy 141 LENGTH 26.61

LIMITS FROM STA 11+000 PLAN 790-69/5-0 TO STA 14+900 PLAN 451-69/122-0

TRAFFIC DATA

EXISTING HIGHWAY 69				
		2001	2011	2021
Muskoka Road 11 (W)	AADT	7600	9100	10600
	SADT	13000	15800	18100
	DHV	1490	1780	2080
	PHV	1460	1750	2040
	% Com	8.1%		
	AR*	0.8	(PAR 1.0)	
	LOS	E	F	
Highway 169 (E) - South Junction of Buckeye Road	AADT	7000	8400	9800
	SADT	12000	14400	16800
	DHV	1370	1650	1920
	PHV	1440	1730	2020
	% Com	9.2%		
	AR*	0.8	(PAR 1.0)	
	LOS	F	F	
Murphy Road (E)	AADT	7000	8400	9800
	SADT	12000	14400	16800
	DHV	1370	1650	1920
	PHV	1440	1730	2020
	% Com	10.7%		
	AR*	0.6	(PAR 1.0)	
	LOS	E	F	
Hwy 612 (W)	AADT	7300	8800	10200
	SADT	12500	15000	17400
	DHV	1430	1720	2000
	PHV	1450	1740	2030
	% Com	10.7%		
	AR*	0.9	(PAR 1.0)	
	LOS	F	F	

Ministry of
Transportation

DESIGN CRITERIA

Date:

February 1998

Reg./Rev./Date:

GROUP WORK PROJECT 280-97-00 Dist. No. 52 Hwy. No. 59

TYPE OF PROJECT Grading, Drainage, Granular Base, Hot Mix Paving, Structures, Partial Illumination

LOCATION From South of Tower Road Northerly to 2.7 km North of Hwy 141 LENGTH 26.8 km

LIMITS FROM STA 11+000 PLAN 790-89/5-0 TO STA 14+900 PLAN 451-89/122-0

Lawson Bay Road, From 72 m West of Highway 69 Easterly 244 m		
ITEM	DESIGN STANDARD	PROPOSED STANDARD
Classification	RLU 50	RLU 50
Design Speed (km/h)	50	50
Minimum Stopping Sight-Distance (m)	85	215
Equivalent 'K' Factor		
Sag	12	50
Crest	8	8
Maximum Grade (%)	12	2.5
Minimum Radius (m)	90	90
Pavement Width (m)	5.5	6.5
Shoulder Width (m)	1.0	1.0
Minimum ROW Width (m)	20	20
Posted Speed (km/h)	50	50

Highway 141, From Highway 69 Easterly 1180 m		
ITEM	DESIGN STANDARD	PROPOSED STANDARD
Classification	RAU 100	RAU 100
Design Speed (km/h)	100	100
Minimum Stopping Sight-Distance (m)	185	680
Equivalent 'K' Factor		
Sag	45	180
Crest	70	70
Maximum Grade (%)	8	1.6
Minimum Radius (m)	420	450
Pavement Width (m)	7.0	7.0
Shoulder Width (m)	2.5	2.5
Minimum ROW Width (m)	40	40
Posted Speed (km/h)	80	80

APPENDIX B

GEOTECHNICAL SURVEY DATA

GEOTECHNICAL SURVEY DATA

W.P. 290-97-00

SURVEY DATE

December 1997 to April 1998

TYPE OF SURVEY

Peto MacCallum Ltd.
(Hand Auger, Power Auger, Backhoe)

NOTES

1. Conditions and pavement depths apply only to the date of the survey.
2. The boundaries between the strata have been established only at test hole locations. Between test holes the boundaries are assumed and may be subject to error.
3. Soils are described according to the MTO Soils Classification System.
4. Abbreviations for test holes and test data conform to OPSD 100.06.

WP 290-97-00 Highway 69

District 52, Huntsville

Lawson Bay Road

Datum Centre Line Pavement

9+785	C/L	D	9+820	10.5	RT C/L	D+1.70
0- 100	Br Sa And Gr		0- 200		Blk Siy Sa Tps	
100	NFP (Frozen)		200		NFP BR	

9+785	C/L	D	9+840		C/L	D
0- 100	Br Sa And Gr		0		NFP BR	
100	NFP (Frozen)					

9+785	10.5	LT C/L	D+500	9+840	10.5	LT C/L	D-2.80
0- 600		Blk Siy Sa Tps		0		NFP BR	
600		NFP BR					

9+785	10.5	RT C/L	D+1.70	9+840	10.5	RT C/L	D-400
0		NFP BR		0- 400		Blk Siy Sa Tps	
				400		NFP BR	

9+800	C/L	D	9+860		C/L	D
0- 400	Blk Siy Sa Tps		0- 600		Blk Siy Sa Tps	
400	NFP BR		600		NFP BR	

9+800	10.5	LT C/L	D+1.40	9+860	10.5	LT C/L	D-1.00
0- 400		Blk Siy Sa Tps		0- 500		Blk Siy Sa Tps	
400		NFP BR		500		NFP BR	

9+800	10.5	RT C/L	D+500	9+860	10.5	RT C/L	D+1.70
0- 300		Blk Siy Sa Tps		0- 200		Blk Siy Sa Tps	
300		NFP BR		200		NFP BR	

9+820	C/L	D	9+880		C/L	D
0- 200	Blk Siy Sa Tps		0- 300		Blk Siy Sa Tps	
200	NFP BR		300		NFP BR	

9+820	10.5	LT C/L	D-1.50	9+880	10.5	LT C/L	D+1.60
0- 800		Br F Sa Tr Si		0		NFP BR	
800		NP BR					

9+880	10.5	RT C/L	D-1.40				
0- 500		Blk Siy Sa Tps					
500- 800		Br F To Med Sa W Si					
800		NFP BR					

Lawson Bay Road
Datum Centre Line Pavement

9+900	C/L	D	0- 300	Blk Siy Sa Tps
0- 400		Blk Siy Sa Tps	300	NFP BR
400- 800		Br F To Med Sa W Si		
800		NFP BR		
9+900	10.5 LT C/L	D+700	9+960	C/L D
0- 200		Blk Siy Sa Tps	0- 200	Blk Siy Sa Tps
200		NFP BR	200	NFP BR
9+900	10.5 RT C/L	D+500	9+960	10.5 LT C/L D+100
0- 200		Blk Siy Sa Tps	0- 100	Br Sa And Gr Fill
200		NFP BR	100	NFP BR
9+920	C/L	D	9+960	10.5 RT C/L D-200
0- 200		Blk Siy Sa Tps	0	NFP BR
200- 600		Br F To Med Sa W Si		
600		NFP BR		
9+920	10.5 LT C/L	D+300	9+980	C/L D
0- 200		Blk Siy Sa Tps	0- 300	Br Sa And Gr Fill
200- 600		Br F To Med Sa W Si	300	NFP BR
600		NFP BR		
9+920	10.5 RT C/L	D-400	9+980	10.5 LT C/L D+300
0- 200		Blk Siy Sa Tps	0	NFP BR
200- 600		Br F To Med Sa W Si		
600		NFP BR		
9+940	C/L	D	9+980	10.5 RT C/L D-700
0- 300		Blk Siy Sa Tps	0- 100	Blk Siy Sa Tps
300- 600		Br F To Med Sa Tr Gr	100	NFP BR
600		NFP BR		
9+940	10.5 LT C/L	D-850	10+000	C/L D
0		NFP BR	0- 200	Blk Siy Sa Tps
			200	NFP BR
9+940	10.5 RT C/L	D+300	10+000	10.5 LT C/L D+100
			0- 500	Blk Siy Sa Tps
			500	NFP BR
			10+000	10.5 RT C/L D-900
			0- 200	Blk Siy Sa Tps
			200	NFP BR

Lawson Bay Road
Datum Centre Line Pavement

10+020 C/L D
0- 200 Blk Siy Sa Tps
200 NFP BR

10+020 10.5 LT C/L D+300
0- 400 Blk Siy Sa Tps
400 NFP BR

10+020 10.5 RT C/L D+1.10
0 NFP BR

10+040 C/L D
0- 400 Blk Siy Sa Tps
400- 700 Br F To Med Sa W Si
700 NFP BR

10+040 10.5 LT C/L D+1.40
0- 200 Blk Siy Sa Tps
200 NFP BR

10+040 10.5 RT C/L D-950
0- 300 Blk Siy Sa Tps
300 NFP BR

10+060 C/L D
0- 600 Blk Siy Sa Tps
600 NFP BR

10+060 10.5 LT C/L D+2.10
0 NFP BR

10+060 10.5 RT C/L D-550
0 NFP BR

10+080 C/L D
0- 350 Blk Siy Sa Tps
350 NFP BR

10+080 10.5 LT C/L D+1.40
0- 200 Blk Siy Sa Tps
200 NFP BR

10+080 10.5 RT C/L D-150
0 NFP BR

10+100 C/L D
0- 200 Blk Siy Sa Tps
200 NFP BR

10+100 10.5 LT C/L D+700
0- 200 Blk Siy Sa Tps
200 NFP BR

10+100 10.5 RT C/L D-1.60
0- 300 Blk Siy Sa Tps
300 NFP BR

10+120 C/L D
0 NFP BR

10+120 10.5 LT C/L D-800
0- 600 Blk Siy Sa Tps
600 NFP BR

10+120 10.5 RT C/L D-1.10
0 NFP BR

10+140 C/L D
0 NFP BR

10+140 10.5 LT C/L D+400
0- 300 Blk Siy Sa Tps
300 NFP BR

10+140 10.5 RT C/L D
0 NFP BR

10+160 C/L D
 0 NFP BR

10+160 10.5 LT C/L D-1.45
 0- 500 Blk Siy Sa Tps
 500 NFP BR

10+160 10.5 RT C/L D-600
 0 NFP BR

10+175 C/L D
 0- 100 Br Sa And Gr Fill
 100 NFP Frozen

10+175 10.5 LT C/L D-1.40
 0- 900 Br F Sa W Si
 900 NFP BR
 Fr Wat @ 300 Above Grade

10+175 10.5 RT C/L D-2.50
 0- 300 Blk Siy Sa Tps
 300- 800 Br F Sa W Si
 800 NFP BR

]

WP 290-97-00 Highway 69

District 52, Huntsville

Highway 69, Northbound Lane, Twp. of Conger

Datum Centre Line Median

18+550 18.8 RT C/L D+1.30
0- 200 Blk F Sa Tps
200- 1.30 Ora Br F Sa W Si Moist
1.30 NFP BR

18+600 18.8 RT C/L D-500
0- 100 Blk Si Tps
100 NFP BR

18+650 4.3 RT C/L D-700
0 NFP BR

18+650 18.8 RT C/L D-800
0- 100 Blk Si Tps
100- 450 Dk Br Si W F Sa
450 NFP BR

18+650 33.3 RT C/L D-1.15
0 NFP BR

18+700 18.8 RT C/L D-1.10
0- 300 Dk Br Si W F Sa Tps
300 NFP BR

18+750 18.8 RT C/L D+100
0- 800 Blk F Sa Tps
800 NFP BR

18+800 18.8 RT C/L D+700
0- 200 Dk Br Sa Tps
200- 1.10 Ora Br F Sa W Si Wet
w @ 700 = 37%
1.10 NFP BR

18+850 C/L D
0- 600 Blk Siy Sa Tps
600 NFP BR

18+850 18.8 RT C/L D+100
0- 800 Blk Siy Sa Tps
800 NFP BR

18+850 37.8 RT C/L D+1.20
0- 100 Blk Org M
100 NFP BR

18+900 18.8 RT C/L D-3.30
0- 500 Blk Siy Sa Tps
500- 2.50 Gry F To Med Sa Tr Si Moist
w @ 1.00 = 17%
2.50 NFP BR
Fr Wat @ 2.00

WP 290-97-00 Highway 69

District 52, Huntsville

Highway 69, Southbound Lane, Twp. of Conger

Datum Centre Line Median

18+650	4.3	LT C/L	D+150	18+850	4.3	LT C/L	D
0- 250		Dk Br Si W F Sa		0		NFP BR	
250		NFP BR					
18+650	18.8	LT C/L	D-3.50	18+850	18.8	LT C/L	D+200
0- 100		Blk Si Tps		0- 600		Blk Siy Sa Tps	
100- 550		Dk Br Si W F Sa		600		NFP BR	
550		NFP BR					
18+650	33.3	LT C/L	D-4.95	18+850	33.3	LT C/L	D+200
0- 100		Blk Si Tps		0- 150		Org M	
100- 800		Dk Br Si W F Sa		150		NFP BR	
800		NFP BR					
18+700	18.8	LT C/L	D+100	18+900	18.8	LT C/L	D+300
0- 300		Dk Br Si W F Sa		0- 400		Blk Say Si Tps	
300		NFP BR		400		NFP BR	
18+750	18.8	LT C/L	D-2.30	18+950	18.8	LT C/L	D-2.10
0- 300		Br Sa Tps		0- 450		Blk Say Si Tps	
300- 1.30		Lt Br Si And F Sa Moist		450		NFP BR	
		w @ 1.00 = 26%					
1.30		NFP BR		18+975		C/L	D
				0- 3.50		Blk Amor Peat	
				3.50- 3.70		Comp Lt Br F To Med Sa Tr Si Wet	
				3.70		NFP BR	
18+800	2.0	LT C/L	D-300	18+975	18.8	LT C/L	D
0- 150		Blk Si Tps		0- 400		Blk Siy Sa Tps	
150- 650		Br Si W F Sa		400		NFP BR	
650		NFP BR					
18+800	18.8	LT C/L	D-1.25	18+975	35.8	LT C/L	D+1.60
0- 200		Blk Si Tps		0- 100		Blk Siy Sa Tps	
200- 750		Br Si W F Sa		100- 600		Dr Br F Sa Tr Si Moist	
750		NFP BR				w @ 400 = 21%	
				600		NFP BR	
18+800	35.5	LT C/L	D-1.95	19+000		C/L	D
0- 300		Dk Br Si		0- 1.00		Blk Amor Peat	
300		NFP BR		1.00		NFP BR	

Highway 69, Southbound Lane, Twp. of Conger
Datum Centre Line Median

19+000 18.8 LT C/L D+50
0- 2.40 Blk Amor Peat
2.40 NFP BR
Fr Wat @ 1.50

19+000 35.8 LT C/L D-150
0- 2.10 Blk Amor Peat
w @ 1.50 = 808%
Org Content @ 1.50 = 83%
2.10- 2.40 L Gry F To Med Sa And Si Wet
w @ 2.15 = 27%
2.40 NFP BR
Fr Wat @ 1.20

19+025 C/L D
0- 2.20 Blk Amor Peat
2.20- 2.70 Soft Gry Siy Cl Wet
2.70 NFP BR
Fr Wat @ 1.00

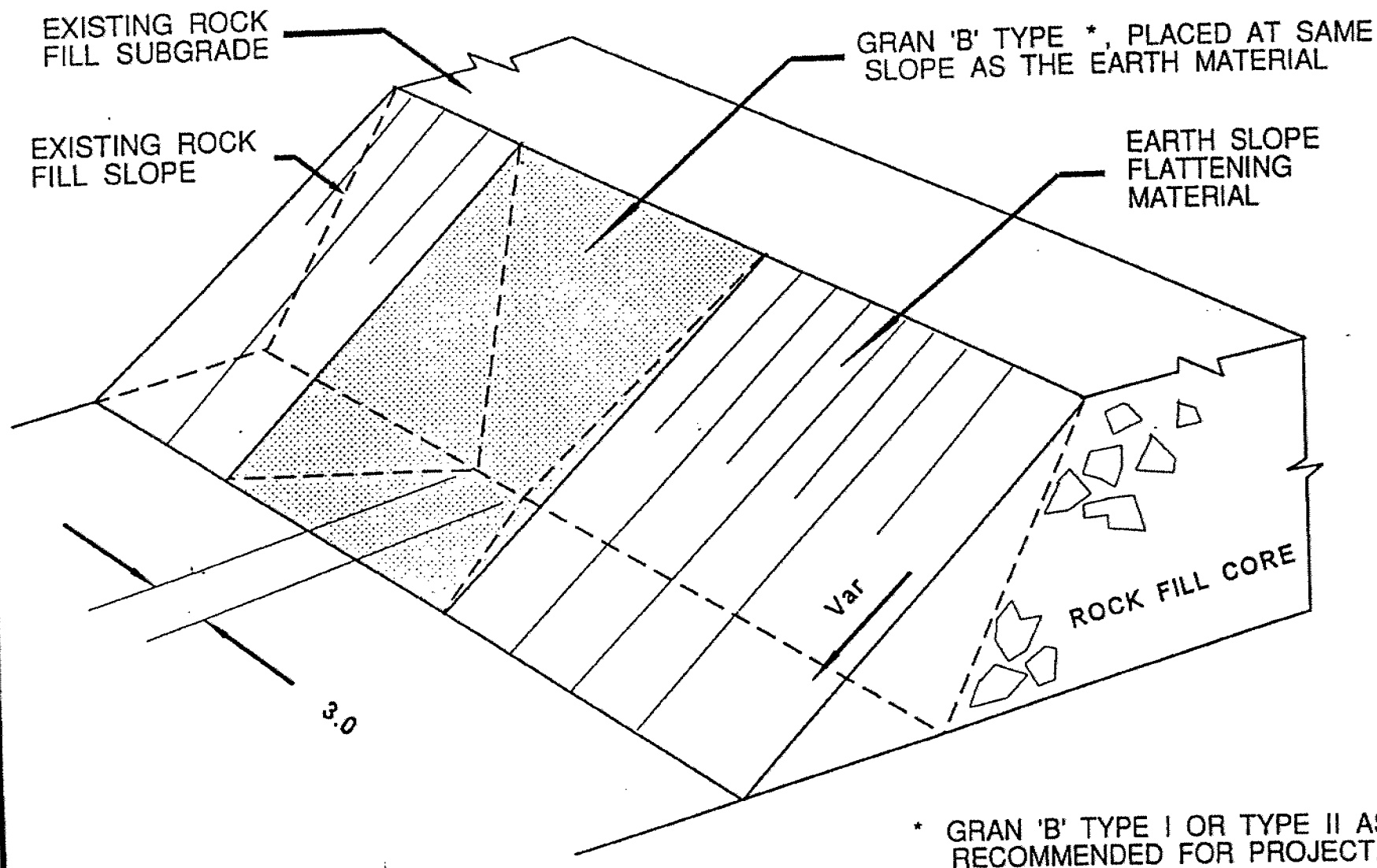
19+025 18.8 LT C/L D
0- 2.50 Blk Amor Peat
2.50- 2.70 Soft Gry Siy Cl Wet
2.70- 3.00 D Gry Sa And Si Wet
w @ 2.80 = 12%
Fr Wat @ 300

19+025 35.8 LT C/L D+150
0- 200 Blk Amor Peat
200 NFP BR

APPENDIX C

ROCKFILL DRAINAGE IN SLOPE FLATTENED AREAS

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



ROCK FILL DRAINAGE IN SLOPE FLATTENED AREAS

NOT TO SCALE

- N-values and soil description should be provided on the cross sections and profiles.
- The cross sections are very small. When the drawing will be reduced to half size for the contract package, then it would not be legible. The cross sections should be produced in large scale. There is enough room on the drawing to draw cross sections in large scale.

Logs:

- For the borehole locations it is preferable to have Northing and Eastings instead of stations and offsets.

Lawson Bay Road Overpass

Drawing:

Drawing No. 1, Job No. 97TF088A, dated March 1998

- Proposed foundation location should be shown on the plan.
- Locations of the boreholes should be shown by coordinates, instead of stations and offset.
- N-values and soil description should be provided on the cross sections and profiles.
- The cross sections are very small. When the drawing will be reduced to half size for the contract package, then it would not be legible. The cross sections should be drawn in large scale.

Rankin Lake Service Road Underpass

Foundations

- Foundation Design Report, Page 3, Last Paragraph. It is recommended to use loose sand to fill the pre-augured holes. Just loose sand is not enough. It should be uniformly graded (Ottawa Sand) or equivalent. The Ministry has specification for the grain size distribution for Ottawa Sand.
- The denseness of the material described on the borehole logs do not agree with the Standard Penetration test results, N-values.

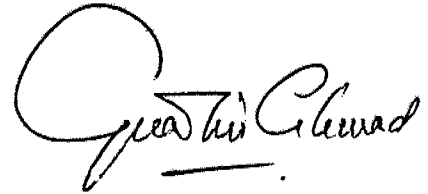
Drawing:

Drawing No. 1, Job No. 97TF088A, dated April 1998

- Proposed foundation locations should be shown on the plan.
- Locations of the boreholes should be shown by coordinates, instead of stations and offset.
- N-values and soil description should be provided on the cross sections and profiles.

- The cross sections are very small. When the drawing will be reduced to half size for the contract package, then it would not be legible. The cross sections should be produced in large scale.

If you have any questions, please advise.

A handwritten signature in black ink, appearing to read 'K. Ahmad', with a horizontal line underneath the name.

K. Ahmad, P. Eng
Foundation Engineer

For

T.C. Kim, P. Eng.
Senior Foundation Engineer

cc: P. Furst
I. Hussain
T. Kazmierowski

GENERAL NOTES

A. DESIGN:

1. LATERAL PRESSURE EXERTED ON SHORING SYSTEM $P = K(\gamma H + q)$

WHERE $K = 0.25$

$\gamma = 21.0 \text{ KN/M}^3$

$H = \text{HEIGHT OF SUPPORTED EARTH}$

$q = \text{COOPER E80 LOADING}$

2. DESIGN COMPLIES WITH THE ONTARIO BUILDING CODE LATEST EDITION AND WITH THE REQUIREMENT OF THE ONTARIO HIGHWAY BRIDGE DESIGN CODE

3. INFORMATION TAKEN FROM STRUCTURAL DRAWINGS PREPARED BY MCCORMICK RANKIN CORPORATION DATED AUG 99

B. MATERIAL:

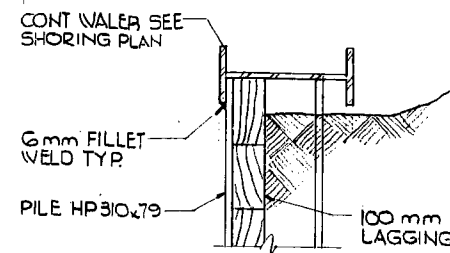
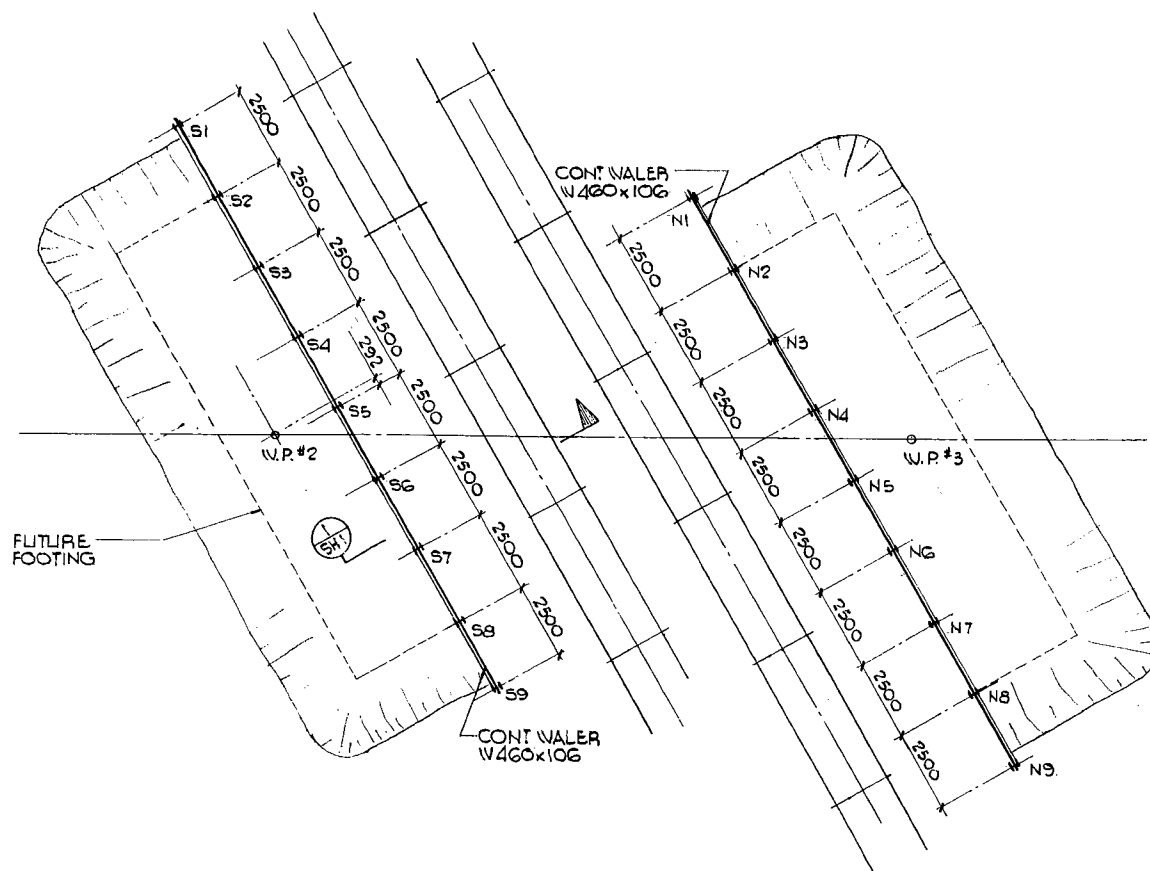
1. STRUCTURAL STEEL TO BE NEW OR SOUND USED MATERIAL CONFORMING TO CSA G40.21 GRADE 300W
2. ALTERNATIVE SECTIONS FOR GRADES OF EQUIVALENT STRENGTH MAY BE SUBSTITUTED SUBJECT TO APPROVAL BY RWB ENGINEERING LTD
3. LAGGING TO BE 100mm (TRUE SIZE) SPRUCE #1 UNLESS OTHERWISE NOTED
4. WELDING SHALL CONFORM TO CSA W59 AND BE PERFORMED BY QUALIFIED WELDERS

C. INSTALLATION

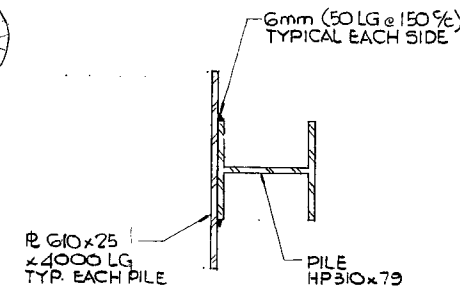
1. DRIVE PILES TO DEPTH SHOWN AND TO LINE
2. EXCAVATE IN 1200 MAX LIFTS INSTALLING LAGGING AS EXCAVATION PROCEEDS. CUT SOIL NEATLY TO ENSURE A TIGHT FIT FOR LAGGING, WEDGE TIGHT AT PILES AS NECESSARY. BACKFILL ALL VOIDS BEHIND LAGGING WITH GRANULAR MATERIAL RAMMED INTO PLACE

D. GENERAL CONTRACTOR

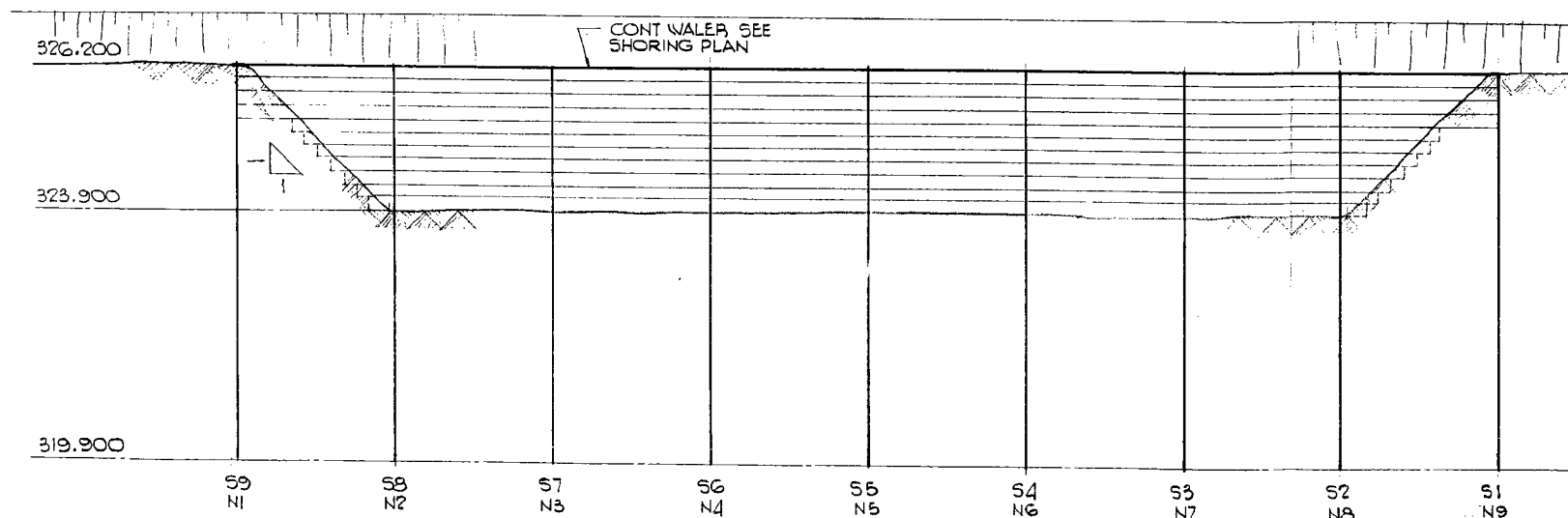
1. GENERAL CONTRACTOR TO PROVIDE THE FOLLOWING:
 - a) CHECK ALL DIMENSIONS PRIOR TO INSTALLATION OF SYSTEM
 - b) LAYOUT STAKES FOR ALL PILES
 - c) CO-ORDINATE SHORING DRAWINGS WITH STRUCTURAL DRAWINGS TO ENSURE NO INTERFERENCE WITH THE PERMANENT CONSTRUCTION
 - d) LOCATE AND IDENTIFY ALL BURIED OR ABOVE GROUND SERVICES THAT MAY INTERFERE WITH INSTALLATION OF SHORING SYSTEM (INCLUDING ALL PUBLIC UTILITIES) MAKE ADJUSTMENT TO SERVICES OR SHORING SYSTEM TO SUIT BEFORE PROCEEDING WITH INSTALLATION
 - e) ADEQUATE PROTECTION TO THE SHORING SYSTEM FROM THE EFFECTS OF WEATHER INCLUDING FROST
 - f) SURVEY OF ALL EXIST CONDITIONS PRIOR TO ANY EXCAVATING OR INSTALLATION OF SHORING
 - g) MONITOR ALL PILES FOR ANY LATERAL MOVEMENTS AND REPORT IMMEDIATELY ANY MOVEMENT EXCEEDING 12mm TO RWB ENGINEERING
 - h) DEWATER AS REQUIRED



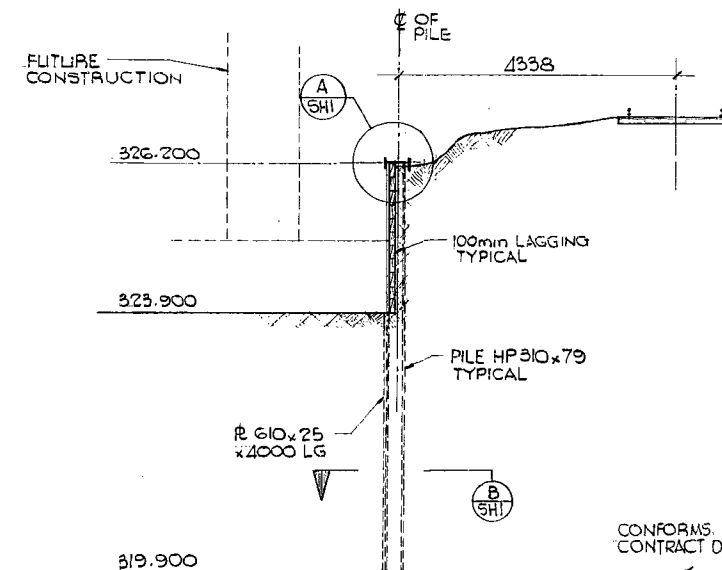
DETAIL A
SHI



DETAIL B
SHI



ELEVATION
SCALE 1:50

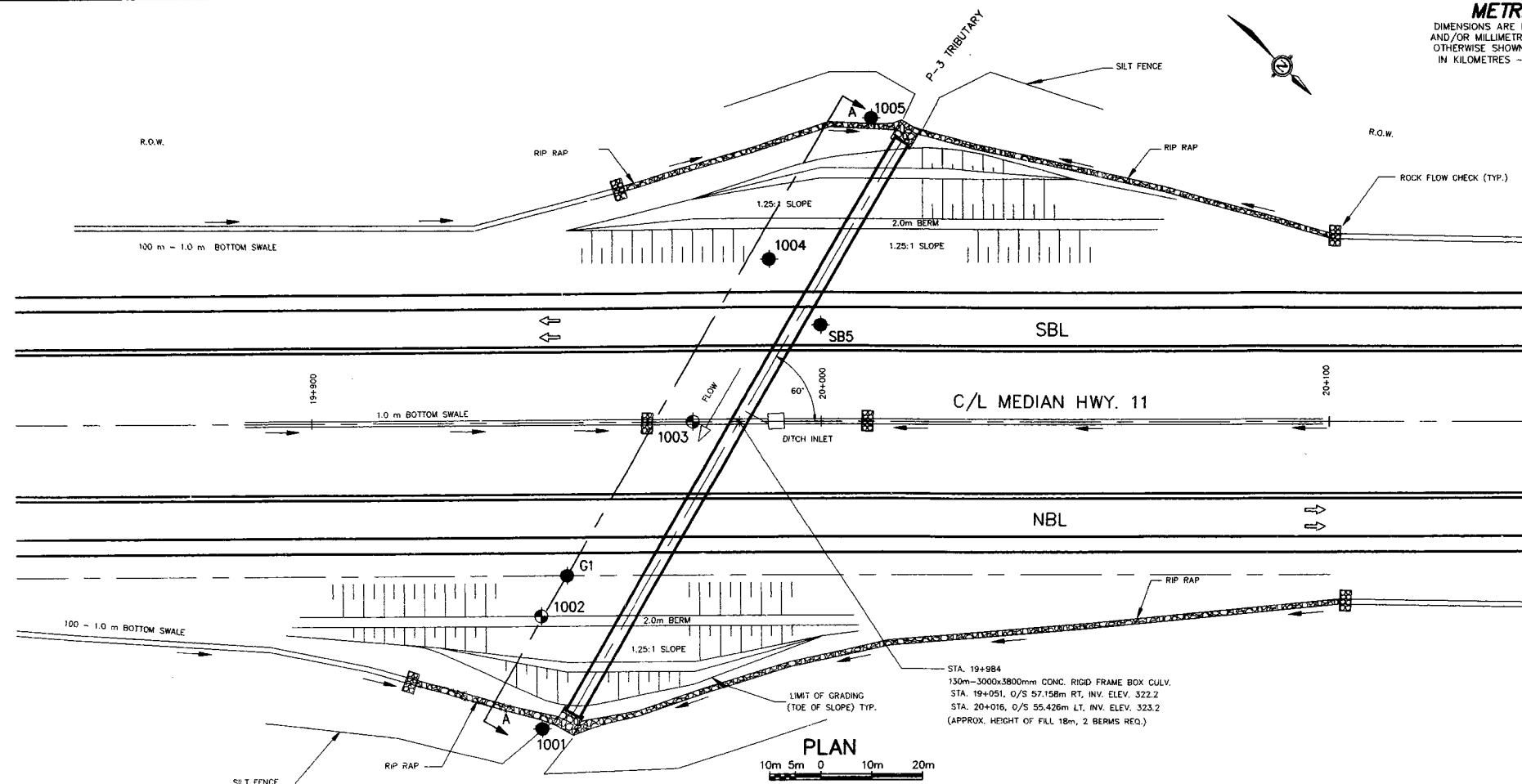


SECTION I
SCALE 1:50
SHI

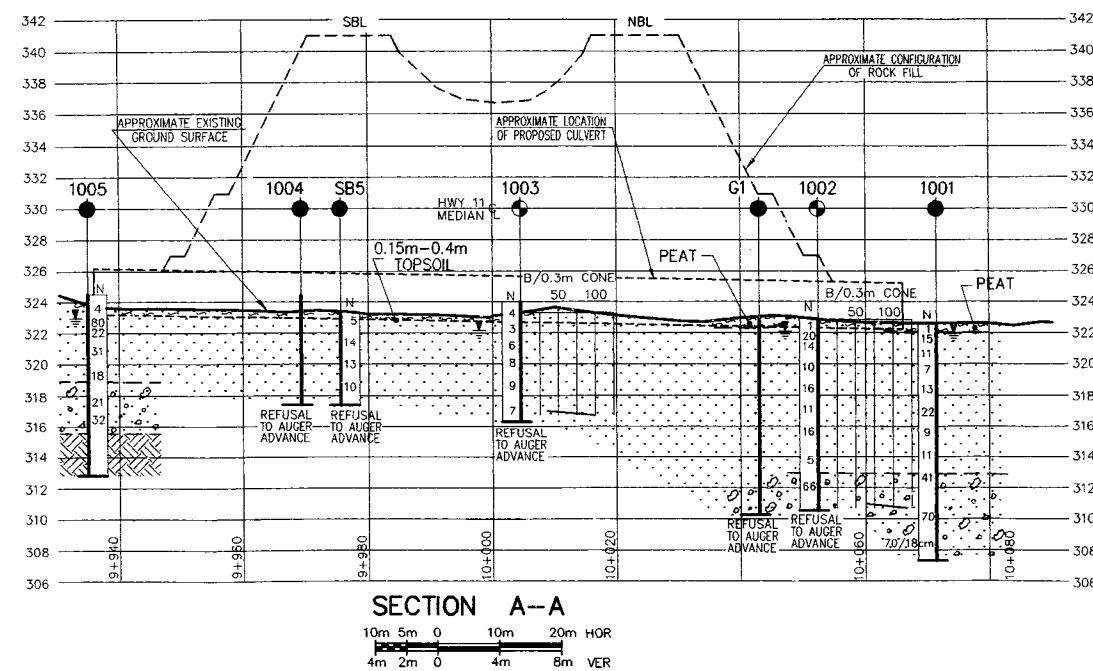
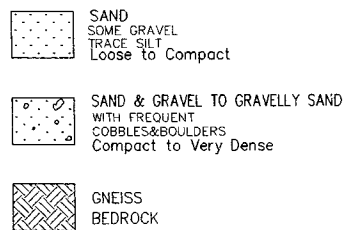
CONFORMS WITH
CONTRACT DOCUMENTS



ISSUED FOR REVIEW		04.13.00
NO.	DESCRIPTION	DATE
DESIGN: CH		
DRAWN: DC		
SCALE: AS NOTED		
DATE: APR 13/00		
STRUCTURAL ENGINEER		
RWB ENGINEERING LTD.		
150 CONSUMERS RD. STE. 505 (416) 756-3102		
WILLOWDALE, ONTARIO M2J 1P9		
CLIENT		
JIM DONN		
PILING LTD		
PROJECT		
MTO 99-221 HIGHWAY 11		
CNR OVERHEAD AT NOVAR		
NBL		
DWG. TITLE		
SHORING PLAN, SECTIONS & DETAILS		
JOB NO.	DWG. NO.	
00B99	SHI	



SOIL STRATIGRAPHY LEGEND



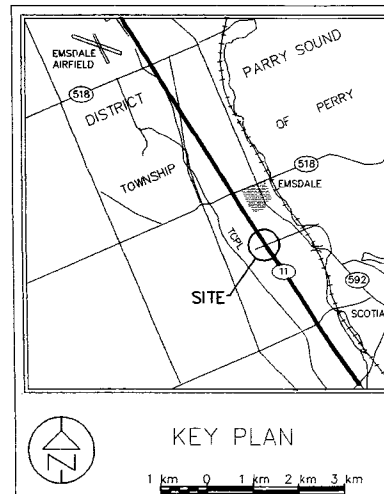
REF. Hwy 11 Site Plan
Dwg. by DELCAN, Dec., 1999

CONT. No.
W.P. No. 466-93-00

PROPOSED CULVERT AT ST. 19+984
& MED. C/L HWY 11
BORE HOLE LOCATIONS & SOIL STRATA

SHEET

AGRA Earth & Environmental Limited



LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊙ Bore Hole & Cone
- 'N' Blows/0.3' (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60' Cone, 475 J/blow)
- WL at time of investigation - Aug. 1999
- WL in Piezometer
- ⊥ Piezometer

No	ELEV.	STATION	CO-ORDINATES OFFSET NORTH EAST
1001	323.4	19+945	59 Rt Med C/L 5 042 383 319 147
1002	323.3	19+948	40 Rt Med C/L 5 042 382 319 132
1003	324.0	19+975	2.0 Lt Med C/L 5 042 373 319 083
1004	324.4	19+980	33 Lt Med C/L 5 042 362 319 150
1005	324.5	20+019	55 Lt Med C/L 5 042 361 319 020
G1	323.2	19+950	30 Rt Med C/L - -
SB5	323.5	20+000	19 Lt Med C/L - -

NOTE: Boreholes G1 and SB5 are documented in AGRA Pavement Design Report for W.P. 466-93-00, Highway 11 Four Lanning, from 2.5 km South of Highway 518E, Northerly 7.3 km at Emsdale.

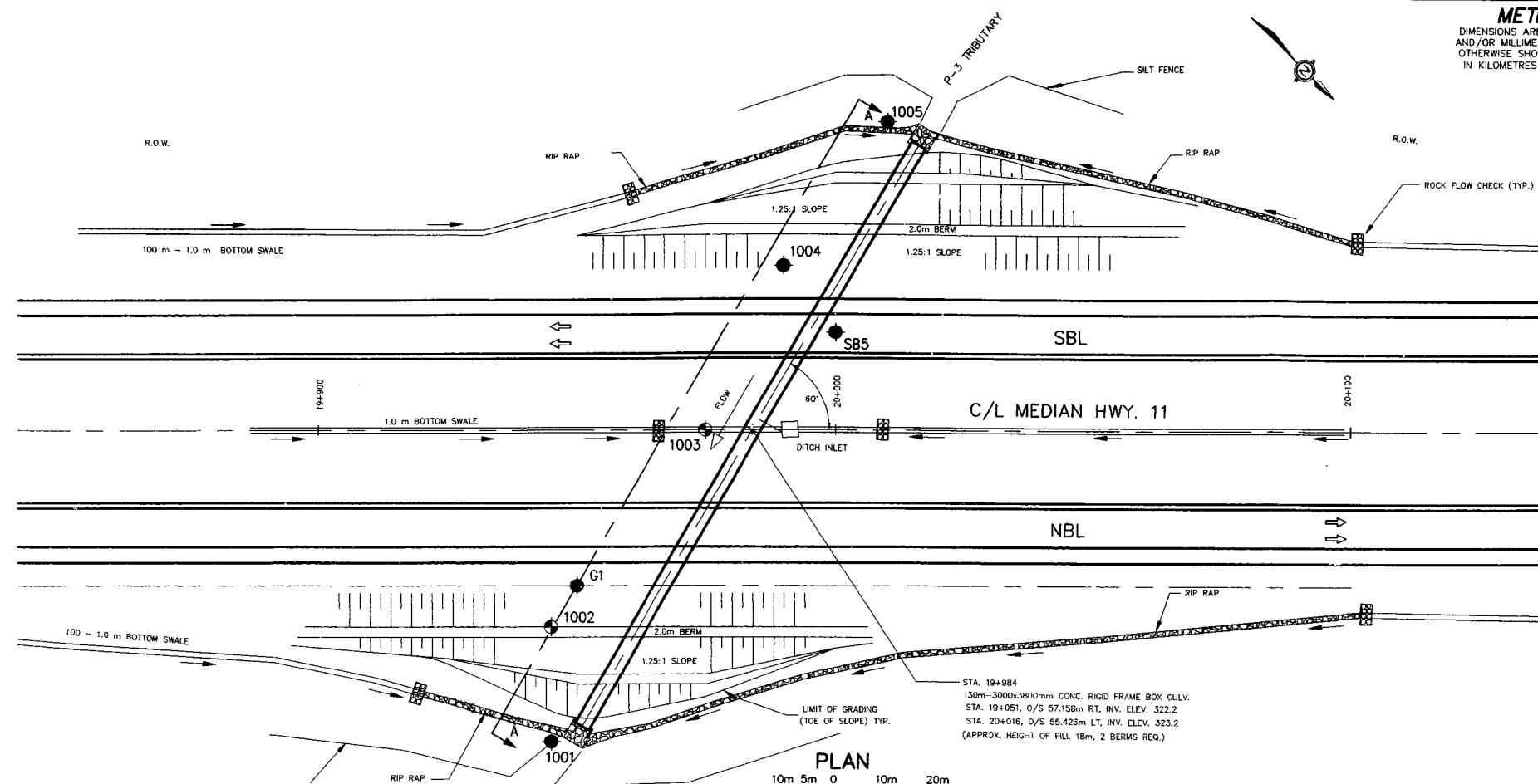
-NOTE-

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

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section GC 2.01 of OPS Gen Cond.

DATE	MA	Revision 1
DATE	BT	DESCRIPTION

HWY No 11	CHECKED AD	DATE Sept. 1999	DIST 52 HUNTSVILLE
SUBM'D SP	CHECKED EVC	APPROVED	SITE 44-304
DRAWN MA	CHECKED EVC	APPROVED	DWG 1



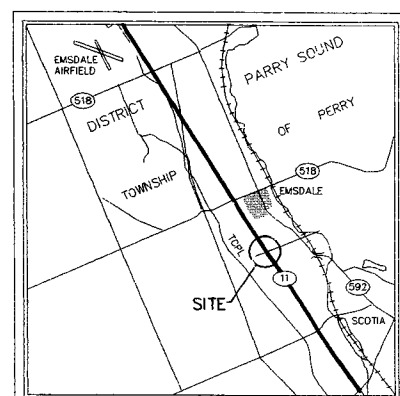
METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES - METRES.

CONT. No.
W.P. No. 466-93-00

PROPOSED CULVERT AT ST. 19+984
& MED. C/L HWY 11
BORE HOLE LOCATIONS & SOIL ST.



AGRA Earth & Environmental Limited



LEGEND

- Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊕ Bore Hole & Cone
- 'N' Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- WL at time of investigation - Aug. 1999
- WL in Piezometer
- Piezometer

No.	ELEV.	STATION	CO-ORDINATES OFFSET NORTH EAST
1001	323.4	19+945	59 Rt Med C/L 5 042 393 319 147
1002	323.3	19+998	49 Rt Med C/L 5 042 382 319 132
1003	324.0	19+975	20 Lt Med C/L 5 042 373 319 083
1004	324.4	19+990	33 Lt Med C/L 5 042 362 319 150
1005	324.5	20+010	55 Lt Med C/L 5 042 361 319 020
G1	323.2	19+950	30 Rt Med C/L - -
SB5	323.5	20+000	19 Lt Med C/L - -

NOTE: Boreholes G1 and SB5 are documented in AGRA Pavement Design Report for W.P. 466-93-00, Highway 11 Four Lanes, from 2.5 km South of Highway 518E, Northerly 7.3 km at Emsdale.

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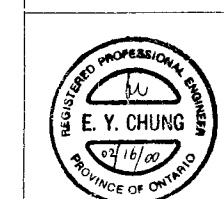
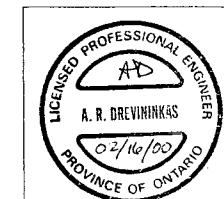
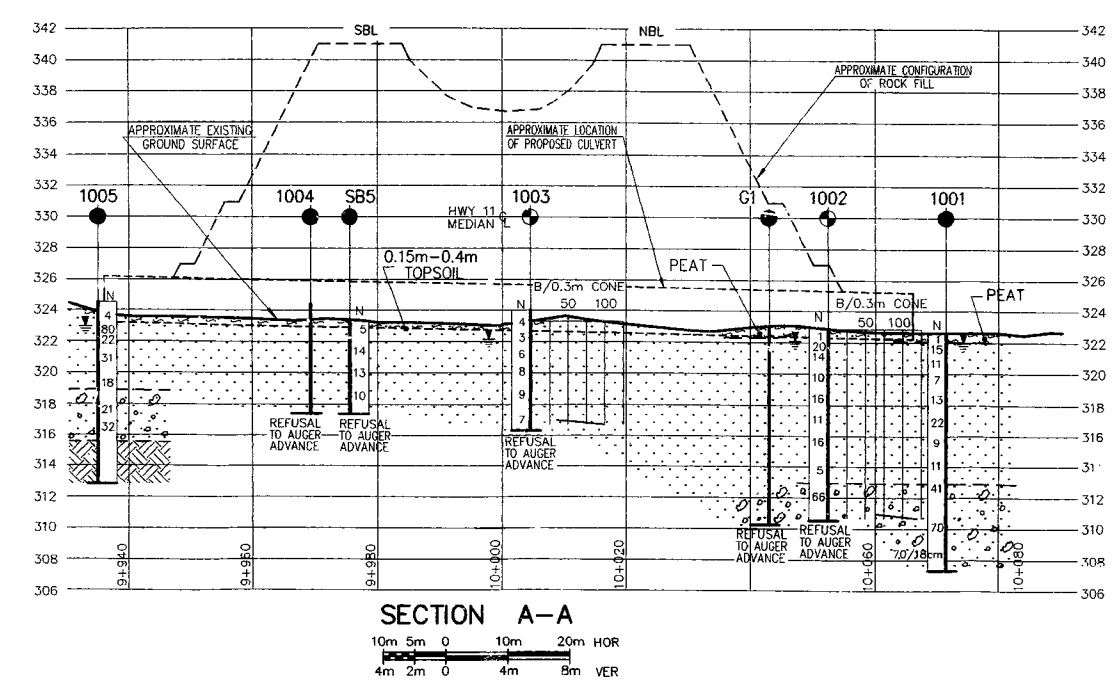
NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview. Information contained in this report and related documents is specific / excluded in accordance with the conditions of Section CC 2.01 of OPS Gen. Cond.

DATE	MA	BY	DESCRIPTION
Feb. 2000	AD		

HWY No. 11	DIST 52 HUNTSVILLE
SUBM'D SP	CHECKED AD DATE Sept. 1999 SITE 44-304
DRAWN MA	CHECKED EYC APPROVED DWG. 1

SOIL STRATIGRAPHY LEGEND

- SAND
SOME GRAVEL
TRACE SILT
Loose to Compact
- SAND & GRAVEL TO GRAVELLY SAND
WITH FREQUENT
COBBLES & BOULDERS
Compact to Very Dense
- GNEISS
BEDROCK



REF. Hwy 11 Site Plan
Dwg. by DELCAN; Dec., 1999