

G.I.-30 SEPT. 1976

GEOCRES No. \_\_\_\_\_

DIST. 54 REGION \_\_\_\_\_W.P. No. 774-93-00

CONT. No. \_\_\_\_\_

W. O. No. \_\_\_\_\_

STR. SITE No. \_\_\_\_\_

HWY. No. 11LOCATION GLEN ROBERTS ROADCONNECTION STATIONS 9+740 TO  
9+960NO PAGES

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. \_\_\_\_\_

REMARKS: \_\_\_\_\_  
\_\_\_\_\_  
\_\_\_\_\_  
\_\_\_\_\_

**Geotechnical Foundation Report  
Embankment Section  
Trout Creek ByPass  
Glen Roberts Road Connection  
Stations 9+740 to 9+960  
GWP No. 774-93-00  
District 54, Sudbury**

Prepared for:

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SO7524G/L/F  
March, 1999



- **Sand/Sand and Gravel Till**

Beneath the silt deposit and immediately beneath the silty clay (approximately stations 9+870 to 9+950), a deposit of dense sand/sand and gravel till was encountered.

The thickness of the deposit varies from about 1 m to 2.7 m with moisture contents in the 10% to 15% range.

- **Bedrock**

Probable bedrock was encountered at fairly shallow depths of between 5 m to 6 m below grade.

### **3.0 GROUNDWATER**

The groundwater table was established within 1 m of existing grade over the relatively flat, poorly drained terrain.

### **4.0 RECOMMENDATIONS**

Based on the proposed profile, as shown on Drawing 1, two embankment sections will be required along this portion of Glen Roberts Road Connection, i.e.:

- (a) An approximately 6 m high fill between stations 9+745 to 9+775 (existing ground level at approximate elevation 318.2 m with proposed grade at approximate elevation 324.5).

and

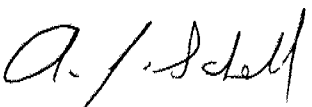
- (b) An approximately 4.5 m high fill between station 9+880 to station 9+955 (existing ground level at approximate elevation 320.8 m with a proposed grade at 325.5 m).


## 7.0 CLOSURE

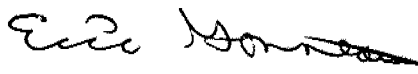
The field investigations were supervised by Mr. E.A. Gonneau, Senior Project Manager. The memorandum report was written by Mr. I.W. Gore, M.Sc., P.Eng., Principal Geotechnical Engineer and reviewed by Mr. E.A. Gonneau, P.Eng., and Mr. R.B. Dodds, P.Eng., Ph.D.

Yours truly,

**TROW CONSULTING ENGINEERS LTD.**

  
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Encl.

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT

SAND

GRAVEL

FINE

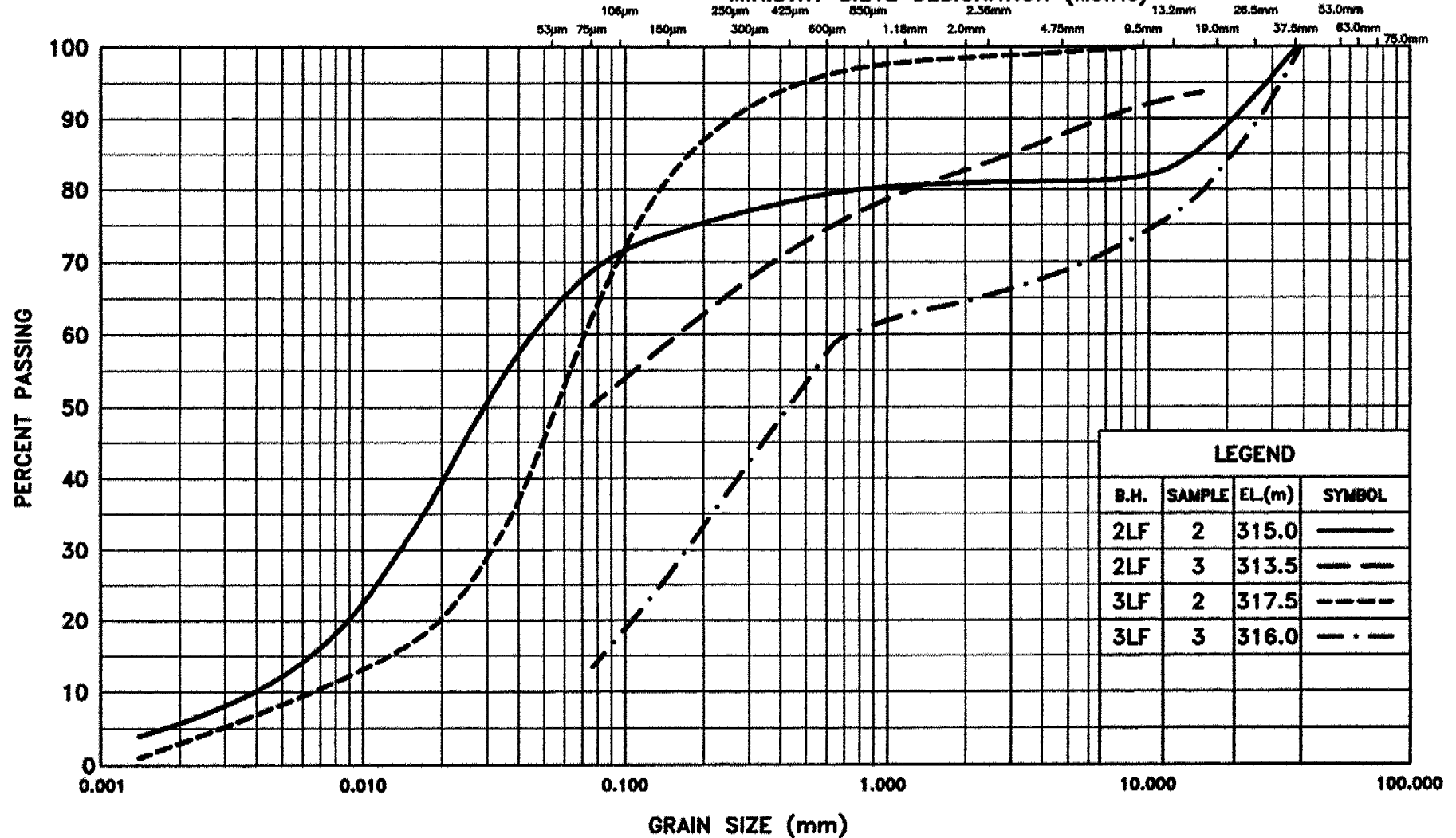
MEDIUM

COARSE

FINE

COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

2LF, SS-2  
2LF, SS-3  
3LF, SS-2  
3LF, SS-3

## GRAIN SIZE DISTRIBUTION

SILT, with SAND seams  
SAND, with zones of SILT  
SILTY SAND  
SILTY SAND

FIGURE 1

W.P 774-93-00



PROJ. No. S07524GLF

## NOTES ON SAMPLE DESCRIPTIONS

- All descriptions included in this report follow the I.S.S.M.F.E. as suggested in the Canadian Foundation Manual. The laboratory grain-size analysis also follows this classification system. Others may designate the unified classification system as their source; a comparison of the two is shown for your information. Please note that, with the exception of those samples where the grain-size analysis has been carried out, all samples are classified visually and the accuracy of visual examination is not sufficient to differentiate between the classification systems or exact grain sizing.

UNIFIED SOIL CLASSIFICATION	Fines (silt or clay)			Sand			Gravel			Cobbles																					
				Fine	Medium	Coarse	Fine	Coarse																							
I.S.S.M.F.E. SOIL CLASSIFICATION	Clay	Silt			Sand			Gravel			Cobbles																				
		Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse																					
	Sieve Sizes																														
	0.001	0.002	0.003	0.004	0.006	0.008	0.01	0.02	0.03	0.04	0.06	0.075	0.08	0.1	0.2	0.3	0.4	0.6	0.8	1.0	2.0	3.0	4.0	6.0	8.0	10	20	30	40	60	80
	Particle Size (mm)																														

- FILL:** Where fill is designated on the borehole log, it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of the site fill material. All fills should be expected to contain obstructions such as large concrete pieces of subsurface basements, floors, tanks, etc.; none of these may have been encountered in the borehole. Since boreholes cannot accurately define the contents of fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact and correct composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant on-going and future settlements. Some fill material may be contaminated by toxic waste that renders it unacceptable for deposition in any but designated land fill sites. Unless specifically stated, the fill on this site has not been tested for contaminants that may be considered hazardous. This testing and a potential hazard study can be carried out if you so request. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common but are not detectable using conventional geotechnical procedures.
- TILL:** The term till on the borehole logs indicate that the material originates from a geological process associated with glaciation. As a result of this geological process, the till must be considered heterogeneous in composition and, as such, may contain pockets and/or seams of material such as sand, gravel silt or clay. As till often contains cobbles (60 to 200 mm) or boulders (over 200 mm), contractors may encounter them during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size, or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited areas; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till material.

## NOTES ON SAMPLE DESCRIPTIONS (Cont'd)



Project No: S07524G/L/F

Drawing No: 2B

4. The following table gives a description of the soil based on particle sizes. With the exception of those samples where grain-size analyses have been performed, all samples are classified visually. The accuracy of visual examination is not sufficient to differentiate between this classification system or exact grain size.

Soil Classification		Terminology	Proportion
Clay	< 0.002 mm	"trace" (eg. trace sand)	1% - 10%
Silt	0.002 to 0.06 mm	"some" (eg. some sand)	10% - 20%
Sand	0.06 to 2 mm	adjective (eg. sandy)	20% - 35%
Gravel	2 to 60 mm	and (eg. and sand)	> 35%
Cobbles	60 to 200 mm	noun (eg. boulders)	> 35% and
Boulders	> 200 mm		main fraction

Classification system as suggested in the Canadian Foundation Engineering Manual, 3rd Edition, unless otherwise noted.

The compactness of cohesionless soils and the consistency of cohesive soils are defined by the following:

Cohesionless Soil		Cohesive Soil	
Compactness	Standard Penetration Resistance "N" Blows/0.3 m	Consistency	Undrained Shear Strength (kPa)
Very Loose	0 to 4	Very Soft	< 12
Loose	4 to 10	Soft	12 - 25
Compact	10 to 30	Firm	25 - 50
Dense	30 to 50	Stiff	50 - 100
Very Dense	Over 50	Very Stiff	100 - 200
		Hard	> 200

### 5. Rock Coring

Where rock drilling was carried out, the term RQD (Rock Quality Designation) is used. The RQD is an indirect measure of the number of fractures and soundness of the rock mass. It is obtained from the rock cores by summing the length of core recovered, counting only those pieces of sound core that are 100 mm or more in length. The RQD value is expressed as a percentage and is the ratio of the summed core lengths to the total length of core run. The classification based on the RQD value is given below.

RQD Classification	RQD
Very poor quality	< 25
Poor quality	25 - 50
Fair quality	50 - 75
Good quality	75 - 90
Excellent quality	90 - 100

$$\text{Recovery Designation \% Recovery} = \frac{\text{Length of Core Per Run}}{\text{Total Length of Run}} \times 100$$

# RECORD OF BOREHOLE BH-1LF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~9+750, on centreline of Glen Roberts Road Connection ORIGINATED BY S.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE August 18, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT		UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20   40   60   80	wp   —   w   —   wl	WATER CONTENT (%)			
							SHEAR STRENGTH:   Cu, KPa					
					UNCONFINED   QUICK TRIAXIAL   FIELD VANE   LAB SHEAR							
					20   40   60   80		10   20   30   40					GR   SA   (SI & CL)
318.35	GROUND SURFACE											
0.00	PEATY TOPSOIL, ~300 mm over SILTY CLAY, brown to grey, some SILT seams, traces of organics. (soft)											
315.75			1	SS	2							
2.60	SILT, with SAND seams, brown, wet. (loose)		2	SS	7							
314.69												
3.66	SILTY SAND & GRAVEL TILL, grey. (very dense)		3	SS	79							
313.17												
5.18	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER											
<div>Notes: 1) This borehole forms part of the Glen Roberts Service Road Foundation Investigation. 2) Borehole drilled at U.T.M. coordinates 5 091 279.1 N, 315 086.2 E. 3) Water level was at surface &amp; hole was open to ~1.7 m depth on completion.</div>												



# RECORD OF BOREHOLE BH-2LF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~9+775, on centreline of Glen Robert Road Connection  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55  
 DATUM Geodetic DATE August 18, 1998

ORIGINATED BY S.M.  
 COMPILED BY M.D.  
 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA	NUMBER	TYPE			20	40	60	80					
318.38	GROUND SURFACE														
0.00	PEATY TOPSOIL, ~250 mm over SILTY CLAY, stratified with SILT seams, grey. (soft)		1	SS	5										
315.64	SILT, with SAND seams, grey. (compact)		2	SS	11										19% 11% 70%
313.82	SAND, zones of SILT with gravel & some cobbles, TILL-LIKE structure, grey. (compact to dense)		3	SS	16										12% 38% 50%
312.59	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER														
5.79	Notes: 1) This borehole forms part of the Glen Roberts Service Road Foundation Investigation. 2) Borehole drilled at U.T.M. coordinates 5 091 302.4 N, 315 077.1 E. 3) Water level was at surface & hole was open to ~1.8 m depth on completion.														



# RECORD OF BOREHOLE BH-3LF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~9+920, on centreline of Glen Roberts Road Connection ORIGINATED BY S.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE August 18, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80				wp — w — wl					
								SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL X FIELD VANE LAB SHEAR				WATER CONTENT (%) 10 20 30 40					
320.81	GROUND SURFACE														GR SA (SI & CL)		
0.00	PEATY TOPSOIL, ~200 mm over					320											
320.21	SILTY SAND, brown, moist. (loose)																
0.60	SILTY CLAY, seams of SILT, wet. (soft)																
318.07			1	SS	5												
318.07																	
2.74	SILTY SAND, some coarse SAND layers, brown to grey, wet. (compact to dense)		2	SS	7										1% 35% 64%		
	A few cobbles at base.		3	SS	35										31% 56% 13%		
315.48	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER																
5.33	Notes: 1) This borehole forms part of the Glen Roberts Service Road Foundation Investigation. 2) Borehole drilled at U.T.M. coordinates 5 091 437.3 N, 315 024.0 E. 3) Water level was at ~1.2 m & hole was open to ~2.3 m depth on completion.																





**Foundation Investigation  
and Design Report  
Bridge Structure, Highway 522 Underpass  
(Site 44-370)  
Trout Creek By-Pass  
(King's Highway 11)  
District 54, Sudbury  
GWP No. 774-93-00  
W.P. No. 770-93-01**

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**SO7524G/C  
July, 1999**

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### Appendix B

Grain Size Analyses

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## Preface

Work Project GWP 774-93-00 is one of a series of projects for the four lane expansion of Highway 11. The project involves the four lane design of Highway 11, from 4.0 km south of Highway 522, northerly for 7.9 km. It will result in the construction of a westerly by-pass of the existing Highway 11 and the Town of Trout Creek.

This work project is located in the Townships of Laurier and Himsworth South, within the geographic District of Parry Sound. The project requires geotechnical input for the following major components:

- New pavement design for the entire length of the four lane by-pass, including associated service roads.
- New structure, Trout Creek South Interchange (underpass), Site 44-372.
- New structure, Trout Creek, Northbound Lanes, Site 44-371.
- New structure, Trout Creek, Southbound Lanes, Site 44-371.
- New structure, Highway 522 (underpass), Site 44-370.
- New structure, Trout Creek North Interchange (underpass), Site 44-369.

The following report deals with the new bridge structure at Highway 522, Site 44-370. Separate reports will be submitted for the additional components.

## **PART 1 FOUNDATION INVESTIGATION**

### **1.1 Introduction**

This submission presents the results of a geotechnical foundation investigation by Trow Consulting Engineers Ltd. (Trow) for the proposed crossing at Highway 522 and the proposed King's Highway 11 (Trout Creek By-Pass), at Site 44-370. It is Trow's understanding that a two span structure will be constructed, with the central pier located in the median of the proposed King's Highway 11. This report contains factual information (obtained from the field investigation) pertaining to the design parameters required for the bridge foundations and related earthworks.

### **1.2 Site Description and Geological Setting**

#### **1.2.1 Site Description**

The site is located in the Township of South Himsworth at the proposed Highway 11 (Trout Creek By-Pass), and Highway 522 intersection, approximate Station 11+224 along the proposed Highway 11, which corresponds to Station 10+000, along Highway 522 at this location.

The proposed new, two-span bridge will be constructed to carry Highway 522 traffic over the four lanes of King's Highway 11. An approximately 8 m grade increase of Highway 522 at the bridge abutments is anticipated, in accordance with the proposed grading plan.

The terrain at the proposed bridge structure is relatively flat, although the grade of the existing Highway 522 rises gently towards both the west and east sides. The existing grade of Highway 522, at the bridge site, is at elevation 315 m and the road then rises gradually some 5 m, over a distance of approximately 240 m on the east side (up to Station 10+240), and 3 m, over approximately 200 m on the west side (up to Station 9 + 800).

The grade of Highway 522 will be raised to elevation 323 m at the bridge to accommodate the proposed four lanes of the by-pass. This arrangement will require approach embankments along Highway 522, approximately 240 m long (from the east side) and 200 m long (from the west side).

No bedrock outcrops are visible in the immediate vicinity of the proposed bridge; however, a rock cut is visible approximately 200 m along Highway 522 on the west side. There are mature trees, with heavy underbrush on either side of Highway 522, i.e. along the alignment of the proposed King's Highway 11.

### **1.2.2 Geological Setting**

According to OGS Maps 2544 and 2556, as well as Ontario Geological Survey Map P. 3160 (Quaternary geology, South River area), the site is located in what is known as the Central Gneiss Belt, i.e. mainly felsic igneous rocks of the Mesoproterozoic Group.

The overburden is expected to be relatively shallow, comprising, for the most part, of detritic sands and gravels with some prodeltaic deposits, mainly silts. A thin layer of basal, stoney, glacial till can be expected immediately over the bedrock.

## **1.3 Investigative Procedures**

### **1.3.1 General**

Part 1 of this report describes the investigative procedures adopted for the geotechnical assessment of the Highway 522 flyover structure at Trout Creek By-Pass, King's Highway 11. Properties of the overburden soils were obtained by in-situ and laboratory testing and the procedures employed during the investigation, are described below.

### **1.3.2 Field Investigation**

The field work for the investigation related to the proposed bridge structure was carried out between May 12 and May 13, 1998, and on May 25, 1998, and consisted of five(5) boreholes (BH's 1-CF to 5-CF), three(3) dynamic cone penetration tests (C-1CF to C-3CF) and four(4) additional auger probes (AP-1CF to AP-4CF). At least three (3) explorations were completed at each of the foundation elements. The probes, dynamic cones and boreholes were advanced to depths ranging from 3.7 m to 11.4 m.

The borehole, probe and dynamic cone penetration locations are shown on the attached site plan, Drawing 1, in Appendix A. These locations, as well as the surface elevations, were established by a survey crew from Marshall Macklin Monaghan, and are referenced to geodetic datum.

The boreholes, cones and probes were advanced through the overburden soils using a truck mounted CME-55 drill, equipped with solid and hollow stem augers, and operated by a soils drilling contractor, Marathon Drilling Limited. Soil samples were obtained by using a 51 mm O.D. split-spoon sampler in conjunction with standard penetration tests (ASTM D1586) at approximately 0.75 m and 1.5 m intervals. The standard penetration (N) values, together with the blows from the dynamic cone penetration tests, were recorded and used to provide an assessment of the compactness of the overburden soils. The recovered soil samples were used for identification and laboratory testing.

Upon completion, boreholes were backfilled with auger cuttings from the same boreholes, and compacted at regular intervals by applying back pressure with the auger. Where boreholes were advanced within the pavement surface, cold mix asphalt was placed to adequately patch the damaged area.

At three (3) of the borehole locations, i.e. at each of the three foundation elements, conventional rock coring techniques were used to advance the boreholes approximately 3 metres into the underlying bedrock. An "NQ" size core barrel and casing were used and core samples of the bedrock were retrieved for rock quality determination and classification.

Details of the soil and bedrock conditions encountered in the boreholes are included on the logs in the attached Appendix A. The additional two standard information sheets included with the logs, provide further details on soil descriptions for classification purposes.

### **1.3.3 Laboratory**

The laboratory testing program for select soil samples consisted of the following:

- Natural moisture content determinations
- Grain size distribution analyses
- Laboratory shear tests

The laboratory test results are summarized on the attached borehole logs in Appendix A. The grain size distribution for selected soil samples are presented in Appendix B.

## 1.4 Subsurface Conditions

The borehole locations are shown on the site plan, Drawing 1, in Appendix A. Also included in Appendix A are the borehole, probe and dynamic cone penetration logs. Soil sections, longitudinal, as well as at each of the three foundation elements, are plotted on Drawings 1A, 1B and 1C. Based on this information, the following different soil layers were encountered:

- fill
- silt
- sand/sand and gravel
- bedrock

A summary of the above soil strata encountered in the boreholes, and inferred from the probes and dynamic cone penetration tests, is presented below.

### 1.4.1 Fill

The fill at the test locations is associated with the road construction materials for the existing Highway 522. Beneath the present asphalt (~50 mm thick) and the base and subbase granulars (~300 mm thick), an underlying layer of sand fill was generally encountered, which extends to depths of 1 m to 2 m below the road grade. The sand fill is generally fine, although it contains random pieces of old asphalt, some gravel and/or cobble sizes, as well as minor organic staining and contamination. At borehole 3-CF, a sand stratum extends somewhat deeper, to about 3 m depth. Since this deposit also contains minor organic inclusions, it is also believed to be fill or possibly alluvial in origin, having been deposited by previous meanderings of an adjacent small creek, which runs beneath the existing Highway 522 in this area.

The compactness of the fill, based on the standard penetration resistance, "N", values ranged from 4 to 35 blows/300 mm, indicating a loose to dense state.



Grain size analyses on samples of the material confirm that the deposit is mainly a fine sand with a silt fraction of between 7% to 20%. Moisture contents vary from less than 10% above the water table to about 20% below the groundwater table.

#### 1.4.2 Silt

A deposit of silt was encountered in three boreholes (boreholes 2-CF, 3-CF and 5-CF). This silt stratum is absent beneath the east abutment (boreholes 1-CF and 4-CF). The silt contains some sand seams and odd layering where it is slightly cohesive. The standard penetration resistance “N” values ranged from 3 to 9 blows/300 mm, indicating a very loose to loose state of compaction. The thickness ranged from 1 m (borehole 3-CF) to 3 m (borehole 2-CF), and the moisture content from 25% to 35%.

#### 1.4.3 Sand and Gravel

A basal zone of sand and gravel was encountered in all five boreholes, with the exception of borehole 2-CF. The deposit is reasonably well-graded with up to 31% silt sizes in places. At some locations, the deposit appears to be weakly cemented, exhibiting a “till-like” structure. At borehole 4-CF, the sand and gravel contains odd, small pieces of wood and cobble sizes. At this particular location, the sand and gravel deposit is likely fill, associated with backfill around the existing, adjacent, 780 mm diameter, CSP culvert.

The standard penetration resistance “N” values, with the exception of borehole 4, ranged from 12 to 41 blows/300 mm, indicating a compact to dense condition. The thickness ranges from about 1 m at borehole 1-CF to 4 m at borehole 3-CF. Moisture contents range from 5% to 20%, although at borehole 4-CF, higher moisture contents of 25% to 30% were measured.

#### 1.4.4 Bedrock

Bedrock was confirmed by retrieving “NQ” size cores in the boreholes (BH’s 1-CF, 2-CF and 3-CF), i.e. at one borehole beneath each of the three foundation elements. Based on the borehole, probe and dynamic cone penetration tests, the bedrock level was established at the following depths:

- East Abutment (BH’s 1-CF and 4-CF, AP-3-CF, C-1CF)  
3.7 m (~El. 311 m) to 4.5 m (~El. 310.4 m)

- Centre Pier (BH 2-CF, AP-1-CF, AP-2-CF)  
5.0 m (~El. 310 m) to 5.3 m (~El. 309.1 m)
- West Abutment (BH's 3-CF and 5-CF, AP-4-CF, C-2CF)  
5.3 m (~El. 310 m) to 8.1 m (~El. 307.2 m)

Detailed descriptions of the rock are presented in Table 1-1 in Appendix A. Generally, the bedrock can be described as a pinkish, light grey, biotite-Hornblende gneiss. The rock is strong and unweathered for the most part.

Rock core recovery was 100% for all runs and the Rock Quality Designation (RQD) values ranged from 66% to 100%.

## 1.5 Groundwater Conditions

Information regarding the groundwater levels at the site was obtained by measuring the water levels in the open boreholes after completion of drilling. The groundwater table, at the time of the field work, was established at a depth of about 2 m to 2.5 m below the grade of Highway 522, which is close to the grade of the surrounding, poorly drained, relatively flat terrain.

Seasonal variations in the water table should be expected with higher levels during wetter periods of the year (such as spring thaw and late fall) and lower levels during drier periods.

## **Part 2    Engineering Discussions and Recommendations**

### **2.1    General**

The following subsections address geotechnical considerations pertaining to the proposed two-span bridge for the Highway 522 underpass of the Trout Creek By-Pass (King's Highway 11). A two-span bridge is proposed to carry Highway 522 traffic over the four lanes of the new By-Pass. The central pier will be located in the By-Pass median, with the abutments located on the west and east sides of the south and north bound lanes of Highway 11.

### **2.2    Foundations**

Based on the explorations, bedrock was encountered at depths of approximately 4 m at the east abutment (8 m from the proposed profile grade), 5 m at the central pier (7.5 m from the proposed profile grade), and 5 m to 8 m at the west abutment (7 m from the proposed profile grade). Foundation alternatives include piled foundations (integral abutments can be considered, although the rock is probably too shallow at the east abutment and pier), and/or spread foundations excavated to sound bedrock. At the west abutment, excavations of up to 8 m below grade will be required, i.e. where the deepest bedrock was encountered. Consequently, at the west abutment, it may be possible to establish the foundations at a higher level on the native, compact, granular soils.

Other options include installing a compacted granular mat over competent material (till and/or rock) to support the foundations, or possibly using caisson-type foundations. However, conventional augered caisson foundations are not normally practical in Northern Ontario because of potential, unknown, sharp irregularities in the bedrock surface, difficulties "seating" casing into the hard, strong, Precambrian rock, as well as the potential excavation difficulties with water seepages in the granular overburden and possible boulders near the rock contact.

Foundation options and design parameters are discussed in the following paragraphs.

## 2.2.1 Foundations on Bedrock

### 2.2.1.1 East Abutment

At the location of the east abutment (BH-1 CF, 4-CF, cone 1-CF and AP3-CF), there is approximately 4 m of overburden soil overlying bedrock. As such, it would be feasible to excavate down to the rock and place the foundation directly on the bedrock surface. For the purpose of design, in accordance with the Ontario Highway Bridge Design Code, the following bearing capacities can be used for spread footings placed directly on the gneiss bedrock, subject to inspection by a qualified geotechnical engineer.

Table 2-1 Spread Footing Capacity on Bedrock	
	Spread Footing
Factored Bearing Capacity at ULS	7,500 kPa

The above Factored Bearing Capacity at ULS applies to spread footings placed directly on bedrock with a good Rock Mass Quality (RQD>75). The bearing capacity at SLS will not govern for a spread footing founded on bedrock, since the loads required to produce unacceptable settlements of the structure will be much larger than the recommended values for the factored capacity at ULS.

For the east abutment area, the borehole, cone and probe data indicate that the construction of spread footings on bedrock would require excavation and removal of approximately 4 m of overburden soil, i.e. down to approximate El. 310 m. The footing area must be cleared of all loose materials prior to placement of concrete and inspected by a qualified geotechnical engineer to verify the Rock Mass Quality.

As per Section 6-8.4.2 of the Ontario Highway Bridge design code, a reduction factor would normally be applied to the Ultimate Bearing Resistance at ULS (7,500 kPa) to account for the effects of inclined loadings. Recent comments, however, received from the Pavement and Foundation Section of MTO indicate that *"Although the OHBDC Code talks about bearing resistance reduction due to inclined loadings for footing on bedrock. The OHBDC committee has decided that no such reduction will be required if the footing is constructed on bedrock"*. As such, for spread footings on

bedrock, the structural engineer should consult with the Ministry to confirm that a reduction factor for inclined loadings need not be applied.

### 2.2.1.2 *Centre Pier*

The subsurface conditions for the centre pier location (BH-2CF, AP1-CF and AP2-CF) are similar to those discussed for the east abutment, although the overburden is slightly thicker, i.e. approximately 5 m. As a result, spread footings placed directly on bedrock using the bearing values given in table 2-2, below, are an option for the central pier foundations subject to inspection during construction by a qualified geotechnical engineer. The bedrock surface is expected at a depth of about 5 m below grade (~El. 310 m).

TABLE 2-2 Spread Footing Capacity on Bedrock	
	Spread Footing
Factored Bearing Capacity at ULS	7,500 kPa

The above Factored Bearing Capacity at ULS applies to spread footings placed directly on bedrock with a good Rock Mass Quality (RQD>75). The bearing capacity at SLS will not govern for a spread footing founded on bedrock, since the loads required to produce unacceptable settlements of the structure will be much larger than the recommended values for the factored capacity at ULS.

### 2.2.1.3 *West Abutment*

The subsurface conditions for the west abutment location (Boreholes 5-CF, 3-CF, cone 2-CF and AP4-CF) are similar to those discussed for the centre pier and east abutment; however, on the south side (cone C-2CF and borehole 3-CF), the rock is locally deeper, i.e. at a depth of some 8 m below grade. Foundations placed directly on bedrock can be designed using the bearing values specified in Table 2-3, below, subject to inspection during construction by a qualified geotechnical engineer.

TABLE 2-3 Spread Footing Capacity on Bedrock	
	Spread Footing
Factored Bearing Capacity at ULS	7,500 kPa

The above Factored Bearing Capacity at ULS applies to spread footings placed directly on bedrock with a good Rock Mass Quality (RQD>75). The bearing capacity at SLS will not govern for a spread footing founded on bedrock, since the loads required to produce unacceptable settlements of the structure will be much larger than the recommended values for the factored capacity at ULS.

Given the depth to bedrock at this abutment, and the high water table, necessitating 6 m to 8 m excavations below the groundwater table, spread footings located on the bedrock may not be the most prudent foundation alternative at this site, as construction difficulties may arise. As such, the other foundation options discussed in this report may be more suitable for this abutment.

As a further alternative, given the depth to rock at this abutment, foundations placed at a higher elevation on the compact and dense, native, sand/sand and gravel strata should also be considered. The compact and dense sand/sand and gravel stratum was encountered at a depth of about 4 m (elevation 311 m) beneath the west abutment location. Since the level of the sand and gravel stratum is about 2.5 m below the water table, dewatering will be required if this option is selected. Foundations placed on the undisturbed, dewatered, sand/sand and gravel stratum could be designed using the bearing capacity values specified in Table 2-4, below.

TABLE 2-4 Spread Footing Capacity on Sand/Sand & Gravel	
	Spread Footing
Factored Bearing Capacity at ULS	300 kPa
Bearing Capacity at SLS	150 kPa

Although founding the abutment on the underlying native sand/sand and gravel strata will require shallower excavations than founding the abutment on bedrock, excavations, will still be required below the water table. This may require dewatering, which may include sheet piling to accomplish.

As such, considering the low allowable bearing resistance values and the potential need for dewatering, this option is not recommended.

#### 2.2.1.4 *Anticipated Bedrock Elevations*

The following table summarizes the location and estimated bedrock elevations at the three foundation elements.

<b>Table 2-5</b> <b>Location and Estimated Elevation of Bedrock Foundation</b>			
<b>Location</b>	<b>Boreholes &amp; Probe Holes</b>	<b>Overburden Thickness (m)</b>	<b>Approximate Bedrock Elevation (m)</b>
East Abutment	Borehole 1-CF	3.7	311.0
	Borehole 4-CF	4.4*	310.4
	Probe Hole AP-3CF	4.1*	311.0
	Cone C-1CF	3.7*	311.4
	Cone C-3CF	5.1*	310.0
Centre Pier	Borehole 2CF	5.2	309.9
	Probe Hole AP-1CF	5.0*	310.0
	Probe Hole AP-2CF	5.3*	310.0
West Abutment	Borehole 3CF	8.1	307.2
	Borehole 5CF	5.3*	310.0
	Cone C-2CF	8.2*	307.5
	Probe Hole AP-4CF	6.7*	309.1

#### *\*Assumed bedrock level*

The above elevations are for preliminary design purposes and were estimated based on the factual borehole, dynamic cone and auger probe holes drilled near the abutment and pier locations. Interpolation between boreholes and probe holes is approximate, and as such, actual footing elevations will depend on the conditions encountered at the time of construction. The bedrock surface in Northern Ontario is known to be erratic. The rock surface at the footing base must be cleaned of all loosened or highly fractured rock and be inspected by a qualified geotechnical engineer to verify the Rock Mass Quality prior to placement of concrete.

### 2.2.2 Footings on Compacted Granular Pad

It may be feasible to establish footings on a Granular A (or equivalent) compacted granular pad. The existing upper loose soils should be subexcavated down to bedrock at both the east and west abutments, as well as at the central pier. As an alternative, at the west abutment, it may only be necessary to excavate down to 4 m (El. 311 m), i.e. down to the compact and/or dense, native, granular soil horizon, provided that the groundwater is controlled to prevent disturbance. The granular pad, when placed over the rock (pier and east abutment) and/or over sand and gravel (west abutment), should extend horizontally a minimum of 1.0 m beyond the plan limits of the footing and have side slopes no steeper than 1 horizontal to 1 vertical. The granular material should be compacted to 100 percent Standard Proctor Maximum Dry Density.

The bearing capacities recommended for the abutment footings placed on this compacted granular pad design, based on the Ontario Highway Bridge Design Code, are as follows:

<b>TABLE 2-6</b> <b>Spread Footing Capacity on Compacted Granular Mat</b>	
	Spread Footing
Factored Bearing Capacity at ULS	400 kPa
Bearing Capacity at SLS	200 kPa

As an alternative to using a granular pad, it would also be feasible to “upfill” over the bedrock, or possibly over the native sand and gravel at the west abutment, and up to underside of the proposed foundations, with lean concrete (typically 15 Mpa mix).

Footings placed on a granular pad beneath the groundwater table will require additional construction considerations. It is very difficult to compact the granular material to an acceptable Standard Proctor Maximum Dry Density by excavation and replacement methods underwater. Since the groundwater table was measured to be above the bedrock level at the time of the investigation, it is probable that dewatering will be required to ensure the granular material can be placed and compacted at optimum moisture levels. Alternatively, the granular material could be replaced by a nominal 400 mm clear stone beneath the groundwater table.



### 2.2.3 Frost Protection

Frost cover is not required for footings placed directly on bedrock. Due to the open nature of bridges, for footings placed on a granular pad, or the native granular soil for the west abutment, a minimum frost cover of 2 m should be provided.

### 2.2.4 Sliding Resistance

The computation of the sliding resistance of the spread footings shall be carried out in accordance with O.H.B.D.C. An unfactored friction angle,  $\phi$  of 32 degrees can be used for sliding along the bedrock and footing base and  $\phi$  of 35 degrees for sliding along granular soils (native sand/sand and gravel or engineered granular pad).

If the factored resistance against sliding failure is inadequate based on friction, then the footings normally could be anchored into bedrock by means of keys, dowels or sockets. However, given the hardness of the bedrock encountered at the site sockets and keys will likely be impractical, and developing adequate resistance against sliding of spread footings founded on the sloping bedrock at the site will likely require dowels. An unfactored coefficient of passive earth pressure,  $K_p'$ , equal to 3.7, can be used for design of a passive resistance key.

### 2.2.5 Piled Foundation

#### 2.2.5.1 *Capacity and Length*

Piling could be considered for the foundation elements at this site. However, because of the proximity of bedrock at the east abutment and central pier (<5 m below grade), piles may only be feasible for support at the west abutment, i.e. where bedrock is deeper. End bearing piles are normally only feasible when the length/width ratio exceeds 10, i.e. a length of 3 m for a 300 mm pile section, since it is very difficult to achieve adequate lateral support for the piles, and the piles will tend to rotate if the applied loading is eccentric.

Piles driven to bedrock can be designed based on the following Limit States design values in accordance with the O.H.B.D.C.

<b>Table 2-7</b> <b>Design Pile Capacities (kN)</b>		
	<b>HP 310x79</b>	<b>HP 310x110</b>
Factored Axial Resistance*	1430 kN	2000 kN
* Note: Structural Office Policy Memo 98-01, April 15, 1998		

Based on the attached borehole logs in Appendix A, the previous Table 2-5 shows a summary of the approximate bedrock elevation at the test locations at which piles would be expected to be founded. Drawings 1A, 1B and 1C in Appendix A show interpreted soil and rock subsurface profiles at the two abutments and pier.

It should be noted that the elevations given in Table 2-5 are approximate. Furthermore, although not experienced in the borings put down at this site, the bedrock elevation in this part of the country can be variable and may change rapidly over a very short distance.

#### 2.2.5.2 Construction

All piles should be driven to bedrock. Since the boreholes indicate that the bedrock elevations are relatively uniform, the potential for irregular, steeply sloping bedrock at the foundation locations is considered to be low to moderate. The bedrock in this part of Northern Ontario, however, is known to be variable. As such, some minor problems may arise during pile seating. At some locations, the piles may have a tendency to skip over the bedrock surface resulting in alignment problems and deeper penetration. In the event that this problem occurs, somewhat longer piles may be required and, in some cases, piles may have to be added or replaced.

To minimize seating difficulties, rock injector points should be considered to facilitate proper seating. All piles must be fitted with reinforcing plates welded to the flanges as per OPSD 3301 to minimize pile damage. It is recommended that, during pile driving and upon initial contact with the bedrock, the pile driving energy should be reduced and subsequently increased incrementally until the piles have been sufficiently seated. OSLO, or similar rock points, installed and driven in accordance with OPSD 3304 and OPSS 903, respectively, may be considered. Once the locations and orientations of the piles have been determined (i.e. during the preliminary design stage), the use of such methods will be determined and recommendations will be provided by this office as required.

All lateral loads at the abutments should be supported using inclined piles.

### 2.2.5.3 *Horizontal Subgrade Reaction Parameters*

Should finite element modelling techniques be utilized, the Horizontal Subgrade Reaction Parameters can be calculated as follows:

$$k = k_1 Z/B$$

Where:  $k_1$  = coefficient of horizontal subgrade reaction for a 300 mm (1 ft.) Wide pile at 300 mm (1 ft.). The values of  $k_1$  are given below in tonnes/m<sup>3</sup>

$Z$  = depth

$B$  = width of pile

	Dry	Submerged
Sand and Gravel Fill (compact)	700	450
Silt (loose)	225	110
Sand and Gravel (compact)	700	450
Sand and Gravel (loose)	350	175

*These values are for design in the elastic range and are taken from published values.*

## 2.2.6 Caisson Foundations

### 2.2.6.1 *Bearing Capacity*

Where bedrock is deeper at the west abutment, a caisson type foundation system to rock could be considered. Caisson foundations placed directly on bedrock could be designed using the bearing values specified in Table 2-8, below.

<b>TABLE 2-8</b> <b>Caisson Foundation Capacity on Bedrock</b>	
	<b>Spread Footing</b>
Factored Bearing Capacity at ULS	5,000 kPa

The above Factored Bearing Capacity at ULS applies to caisson foundations placed directly on bedrock with a good Rock Mass Quality (RQD>75). The bearing capacity at SLS will not govern for a caisson founded on bedrock, since the loads required to produce unacceptable settlements of the structure will be much larger than the recommended values for the factored capacity at ULS.

#### **2.2.6.2 Construction**

As noted earlier in this report, caisson foundations, which are typically large diameter augered elements, which are cased to rock, are not normally feasible in Northern Ontario. The potential for sharp, unknown irregularities in the bedrock surface, difficulties “seating” and sealing casings at the hard strong bedrock contact, as well as the potential excavation difficulties with water seepages in the granular soils and possible boulders near the rock contact, render augered caissons difficult, if not impractical. As such, if caissons are considered at the west abutment, they will likely have to be installed using a backhoe type excavator with temporary braced shoring to support the open excavation sides and appropriate dewatering procedures..

### **2.3 Backfill**

Backfill to abutments or retaining walls should consist of free-draining granular materials such as Granular “A” and Granular “B” or rock fill. Computation of earth pressures shall be in accordance with Section 6.7.4 of the Ontario Highway Bridge Design Code. Unfactored properties for backfill materials are provided in the following table.

<b>Table 2-9</b> <b>Material Types and Unfactored Properties</b>					
<b>Material</b>	<b>Friction Angle, <math>\phi'</math></b>	<b><math>\gamma(\text{kN/m}^3)</math></b>	<b><math>K_a</math></b>	<b><math>K_p</math></b>	<b><math>K_o</math></b>
Granular A	35 degrees	22.5	0.27	3.7	0.43
Granular B	30 degrees	21.1	0.33	3.0	0.50
Rock Fill	35 degrees	18.0	0.27	3.7	0.43

*Note:  $K_a$  is the earth pressure coefficient corresponding to the active state.*

*$K_p$  is the earth pressure coefficient corresponding to the passive state.*

*$K_o$  is the earth pressure coefficient at rest.*

If rock fill is used as backfill behind abutments, the particle size should be limited to no greater than 300 mm and the backfill must be placed carefully in a manner that does not cause damage to the abutments or other structural components of the bridge.

As shown in figure 6.7.4.3 of the Ontario Highway Bridge Design Code, the lateral earth pressure, as a result of compaction shall be increased by 16 kPa.

## 2.4 Excavations

### 2.4.1 Overburden

Excavations through overburden soil will be required if spread foundations are used. The overburden granular soils are classified as Type 3 soils and the maximum depth of excavation anticipated at the site is approximately 8 metres at the west abutment. As such, excavations in accordance with the Occupational Health and Safety Regulations for Construction Projects for Type 3 soils will be adequate, provided the groundwater in the overburden soil is removed. If appropriate dewatering is not done, the soil would have to be classified as a Type 4 soil and any excavation greater than 1.2 m should then be sloped to 3 horizontal to 1 vertical, starting from the base of the excavation, or appropriate shoring provided.

### 2.4.2 Bedrock

Any removal of bedrock required for the foundations (spread footings and/or at the base of caissons) will require drilling and blasting procedures.

## 2.5 Approach Embankments

No stability or significant settlement problems are anticipated for the approach embankments established over the essentially granular soils. Topsoil and compressible organics (if present) must be removed from the plan limit of the approach embankments. Based on Trow's adjacent borings for the pavement design of Highway 11, it is likely that the surficial organics will be about 300 mm to 600 mm thick. If rock fill is used to construct the approach embankments, the side slopes and forward slopes should be constructed at a maximum gradient of 1.25H:1V. If Granular "A" or Granular "B" is used, the forward and side slopes should be constructed at 2H (minimum):1V.

The geotechnical conditions are such that integral abutment design could be considered, if feasible, from structural, practical and economical considerations. It should be noted, however, that the depth to bedrock at the east abutment and pier is less than approximately 5 m.

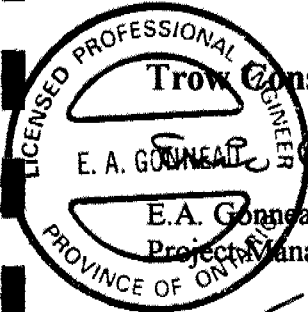
## 2.6 General

The information presented in this report is based on a limited investigation designed to provide information to support an overall assessment of the current geotechnical conditions at the site of the proposed Highway 522/King's Highway 11 Trout Creek By-Pass. The conclusions presented in this report reflect site conditions existing at the time of the investigation. It is noted that the soil boundaries indicated on the logs are inferred from discontinuous sampling and observations during drilling. These boundaries are intended to reflect transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change.

This report has been prepared by Mr. I.W. Gore, P.Eng., and Mr. E.A. Gonneau, P.Eng., and reviewed by Mr. S.E. Gonsalves, P.Eng. The field investigation was performed by Mr. I. Dumpis, C.E.T.

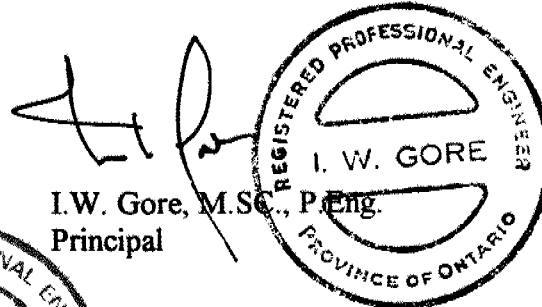
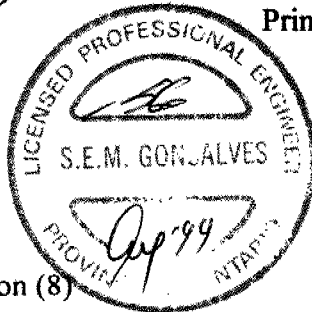
We trust this report is satisfactory for your purposes. Should you have any questions, please do not hesitate to contact this office.

**Trow Consulting Engineers Ltd.**



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Planning & Design  
Mr. E. Gallant

Marshall Macklin Monaghan (1)  
Mr. R.D. Kivi, P.Eng.

**APPENDIX A**

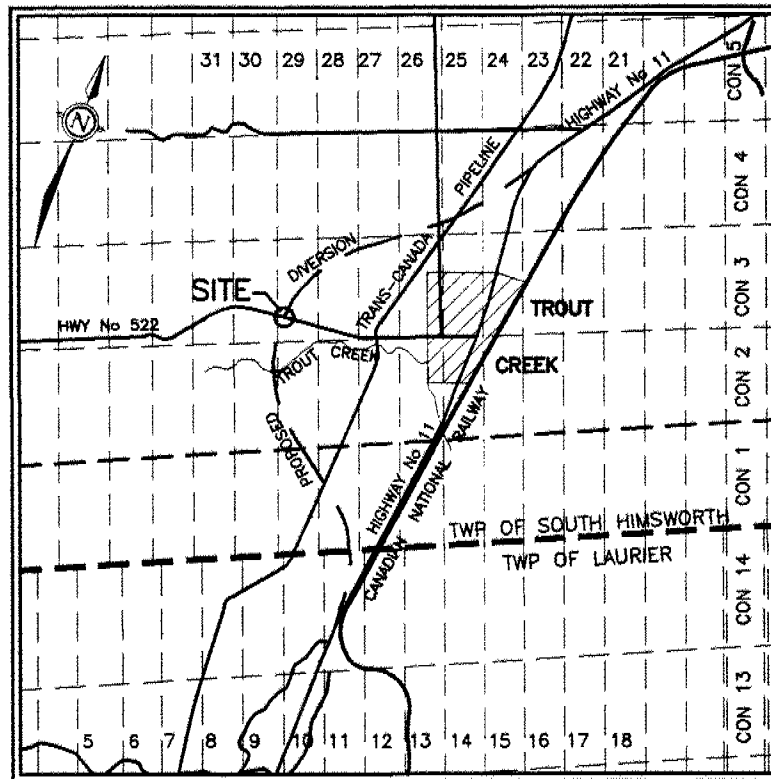
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METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES  
UNLESS OTHERWISE SHOWN

PLATE No 509-11/176-0  
DRAWING No 05090011176  
CONT No  
WP No 774-93-00

SHEET



## KEY PLAN

1 km 0 1 km



TROW CONSULTING ENGINEERS LTD.  
SUDBURY, ONTARIO

**Trow**

PROJ. No. S07524GCF DWG. No. 1

MINISTRY OF TRANSPORTATION  
ENGINEERING OFFICE  
SURVEYS AND PLANS SECTION

### KEY PLAN

PROPOSED CROSSING

AT

SEC HIGHWAY 522

AND

PROPOSED C/L MEDIAN HWY 11

GEOG TWP SOUTH HINSWORTH  
LOT 28

DIST OF PARRY SOUND  
CON 3

SCALE  
1:400

DISTRICT  
PARRY SOUND

REGION  
NORTHERN

ETR  
509-11

SURVEY DATE 97/10

PLAN DATE 97/10

SITE 44-370

PLANE-509-11-12



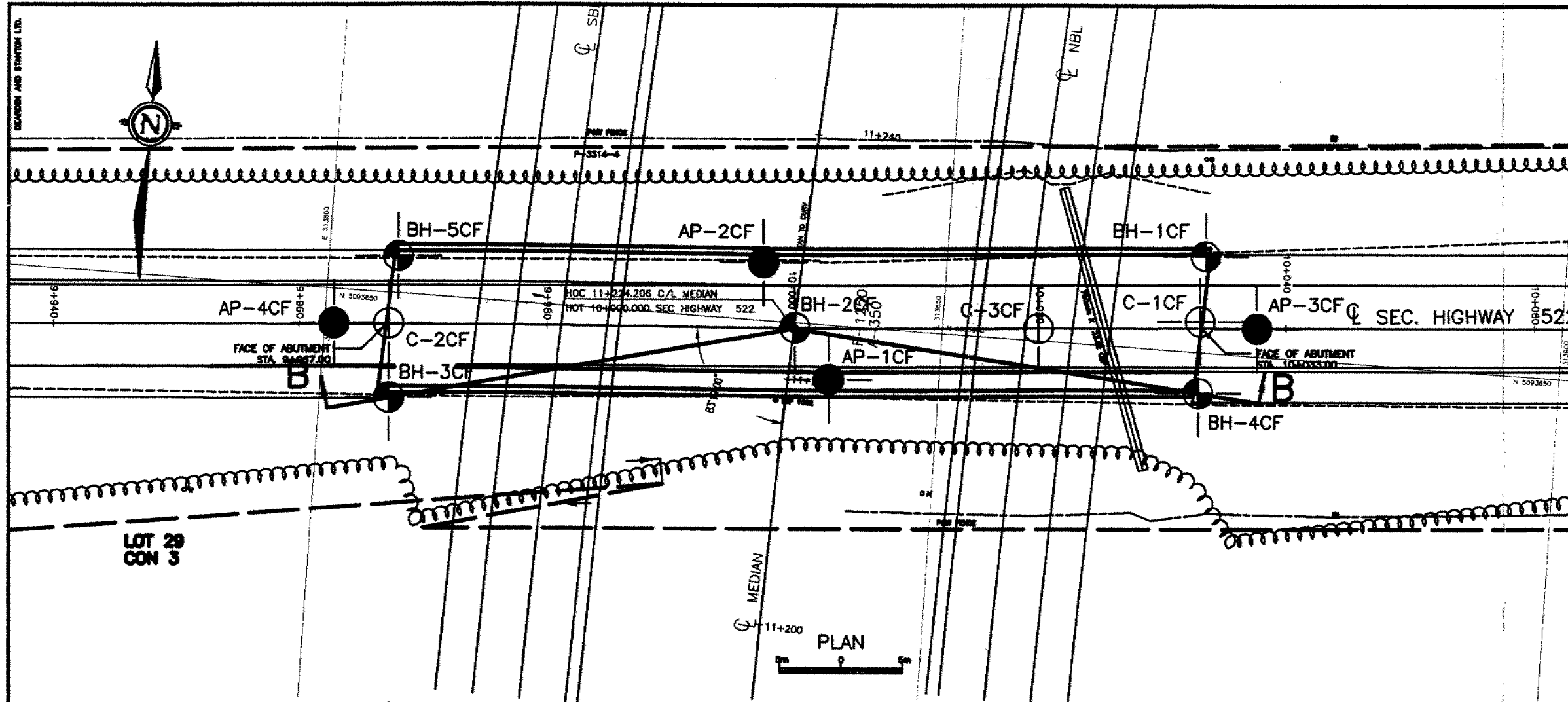


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 CONT No  
 WP No 774-93-00

SHEET

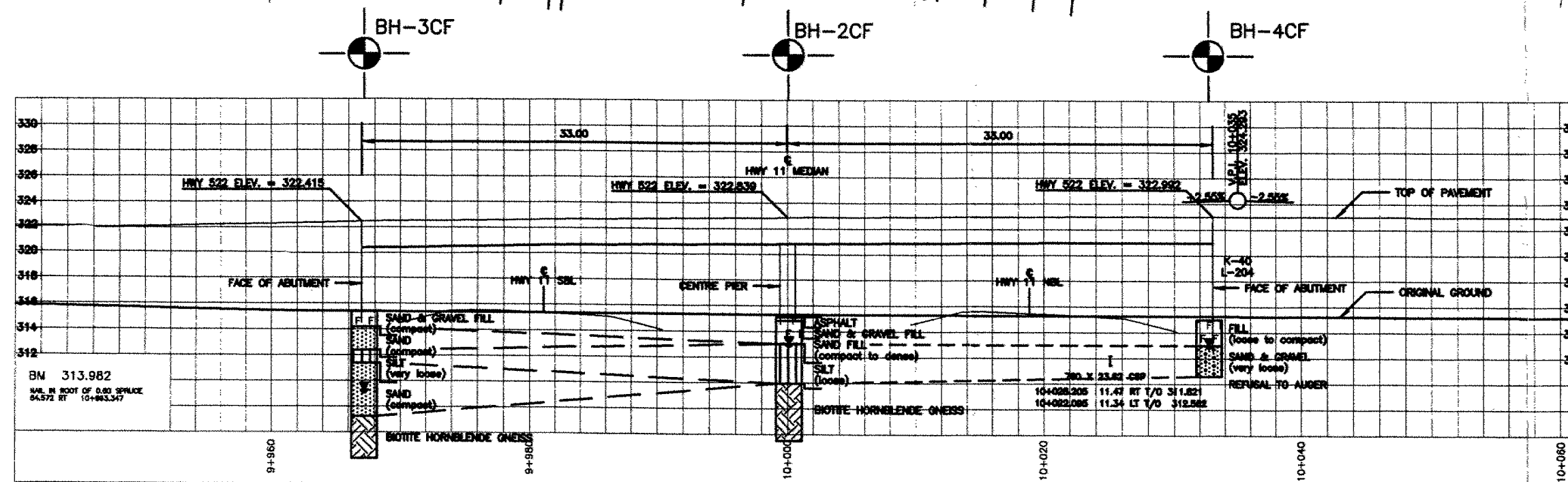
### LEGEND

- BOREHOLE
- AUGER PROBE
- DYNAMIC CONE PENETRATION TEST
- GROUND WATER LEVEL

No.	ELEV.	CO-ORDINATES	
		NORTH	EAST
BH-1CF	314.90	5 093 657.8	313 870.8
BH-2CF	315.13	5 093 649.6	313 838.0
BH-3CF	315.37	5 093 641.5	313 805.6
BH-4CF	314.98	5 093 646.7	313 870.9
BH-5CF	315.24	5 093 653.0	313 805.6
C-1CF	315.14	5 093 652.5	313 870.7
C-2CF	315.67	5 093 647.5	313 805.2
C-3CF	315.18	5 093 651.0	313 857.8
AP-1CF	315.93	5 093 645.6	313 841.0
AP-2CF	314.93	5 093 654.7	313 835.1
AP-3CF	315.16	5 093 652.3	313 875.4
AP-4CF	315.74	5 093 647.2	313 800.8

### METRIC

DIMENSIONS ARE IN METRES  
 AND/OR MILLIMETRES  
 UNLESS OTHERWISE SHOWN



SECTION B-B  
 ON PROFILE OF SEC HIGHWAY 522

TROW CONSULTING ENGINEERS LTD.  
 SUDBURY, ONTARIO  
 PROJ. No. S07524GCF DWG. No. 1B

MINISTRY OF TRANSPORTATION  
 ENGINEERING OFFICE  
 SURVEYS AND PLANS SECTION

### SECTION B-B

### PROPOSED CROSSING

AT  
 SEC HIGHWAY 522

AND  
 PROPOSED C/L MEDIAN HWY 11

GEOG TWP SOUTH HINSMORTH DIST OF PARRY SOUND  
 LOT 29 CON 3

SCALE	DISTRICT	REGION
1:400	PARRY SOUND	NORTHERN
	ETR 509-11	
SURVEY DATE 97/10	PLAN DATE 97/10	
SITE 44-370	PLANE-509-11-12	



**NOTES ON SAMPLE DESCRIPTIONS**

1. All descriptions included in this report follow the I.S.S.M.F.E. as suggested in the Canadian Foundation Manual. The laboratory grain-size analysis also follows this classification system. Others may designate the unified classification system as their source; a comparison of the two is shown for your information. Please note that, with the exception of those samples where the grain-size analysis has been carried out, all samples are classified visually and the accuracy of visual examination is not sufficient to differentiate between the classification systems or exact grain sizing.

UNIFIED SOIL CLASSIFICATION	Fines (silt or clay)			Sand			Gravel		Cobbles																								
				Fine	Medium	Coarse	Fine	Coarse																									
	Clay	Silt			Sand			Gravel			Cobbles																						
		Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse																							
	I.S.S.M.F.E. SOIL CLASSIFICATION	Sieve Sizes																															
	0.001	0.002	0.003	0.004	0.006	0.008	0.01	0.02	0.03	0.04	0.06	0.075	0.08	0.1	0.2	0.3	0.4	0.6	0.8	1.0	2.0	3.0	4.0	6.0	8.0	10	20	30	40	60	80		
	Particle Size (mm)																																

2. **FILL:** Where fill is designated on the borehole log, it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of the site fill material. All fills should be expected to contain obstructions such as large concrete pieces of subsurface basements, floors, tanks, etc.; none of these may have been encountered in the borehole. Since boreholes cannot accurately define the contents of fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact and correct composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant on-going and future settlements. Some fill material may be contaminated by toxic waste that renders it unacceptable for deposition in any but designated land fill sites. Unless specifically stated, the fill on this site has not been tested for contaminants that may be considered hazardous. This testing and a potential hazard study can be carried out if you so request. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common but are not detectable using conventional geotechnical procedures.
3. **TILL:** The term till on the borehole logs indicate that the material originates from a geological process associated with glaciation. As a result of this geological process, the till must be considered heterogeneous in composition and, as such, may contain pockets and/or seams of material such as sand, gravel silt or clay. As till often contains cobbles (60 to 200 mm) or boulders (over 200 mm), contractors may encounter them during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size, or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited areas; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till material.

# NOTES ON SAMPLE DESCRIPTIONS (Cont'd)



Project No: S07524G/C

Drawing No: 2B

4. The following table gives a description of the soil based on particle sizes. With the exception of those samples where grain-size analyses have been performed, all samples are classified visually. The accuracy of visual examination is not sufficient to differentiate between this classification system or exact grain size.

Soil Classification		Terminology	Proportion
Clay	< 0.002 mm	"trace" (eg. trace sand)	1% - 10%
Silt	0.002 to 0.06 mm	"some" (eg. some sand)	10% - 20%
Sand	0.06 to 2 mm	adjective (eg. sandy)	20% - 35%
Gravel	2 to 60 mm	and (eg. and sand)	> 35%
Cobbles	60 to 200 mm	noun (eg. boulders)	> 35% and
Boulders	> 200 mm		main fraction

Classification system as suggested in the Canadian Foundation Engineering Manual, 3rd Edition, unless otherwise noted.

The compactness of cohesionless soils and the consistency of cohesive soils are defined by the following:

Cohesionless Soil		Cohesive Soil	
Compactness	Standard Penetration Resistance "N" Blows/0.3 m	Consistency	Undrained Shear Strength (kPa)
Very Loose	0 to 4	Very Soft	< 12
Loose	4 to 10	Soft	12 - 25
Compact	10 to 30	Firm	25 - 50
Dense	30 to 50	Stiff	50 - 100
Very Dense	Over 50	Very Stiff	100 - 200
		Hard	> 200

## 5. Rock Coring

Where rock drilling was carried out, the term RQD (Rock Quality Designation) is used. The RQD is an indirect measure of the number of fractures and soundness of the rock mass. It is obtained from the rock cores by summing the length of core recovered, counting only those pieces of sound core that are 100 mm or more in length. The RQD value is expressed as a percentage and is the ratio of the summed core lengths to the total length of core run. The classification based on the RQD value is given below.

RQD Classification	RQD
Very poor quality	< 25
Poor quality	25 - 50
Fair quality	50 - 75
Good quality	75 - 90
Excellent quality	90 - 100

$$\text{Recovery Designation \% Recovery} = \frac{\text{Length of Core Per Run}}{\text{Total Length of Run}} \times 100$$

# RECORD OF BOREHOLE BH-1CF 1 OF 1

METRIC

W.P. 774-93-00

LOCATION 5 093 657.8 N, 313 870.8 E

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Hollow Stem Augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE May 12, 1998

CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT				UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20    40    60    80				wp    —    w <sub>p</sub> —    w <sub>l</sub>					
								SHEAR STRENGTH: Cu, KPa ● UNCONFINED      × FIELD VANE QUICK TRIAXIAL      LAB SHEAR				WATER CONTENT (%) 10    20    30    40					
314.90	GROUND SURFACE														GR    SA    SI + CL		
0.00	FILL, mostly silty sand & gravel with a few cobble sizes, occasional pieces of asphalt, brown, moist. (compact)	F															
		F	1	SS	28												
312.40		F															
2.50	SAND & GRAVEL, pockets of sand, some cobble sizes & possible boulders, brown, wet. (compact)	S															
		S	2	SS	13												
311.18		S															
3.72	BIOTITE HORNBLende GNEISS, pinkish grey, excellent rock quality, unweathered.	H													Rec 100% RQD 98%		
		H	3	NQ													
		H	4	NQ											Rec 100% RQD 100%		
		H	5	NQ											Rec 100% RQD 100%		
308.01	END OF BOREHOLE																
6.89	Notes: 1) This borehole forms part of Highway 522 Underpass Foundation Investigation. 2) Borehole located at station 10+033.3, offset ~6.5 m left of centreline as referenced to Highway 522.																





# RECORD OF BOREHOLE BH-2CF 1 OF 1

METRIC

W.P. 774-93-00

LOCATION 5 093 649.6 N, 313 838.0 E

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Hollow Stem Augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE May 13, 1998

CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20	40	60	80					
315.13	GROUND SURFACE															
314.99	ASPHALT, 50 mm thick over															
0.30	SAND & GRAVEL FILL															
	SAND FILL, some gravel inclusions, traces of organics, brown, moist. (compact/dense)		1	SS	35											0% 80% 20%
			2	SS	25											
313.03	SILT, fine sand inclusions in parts with occasional fine clay layers, grey, wet. (loose)		3	SS	9											
2.10			4	SS	5											
	Some cobble sizes at base.		5	SS	5											
309.95	BIOTITE HORNBLende GNEISS, pinkish grey, fair to excellent rock quality, slightly weathered to unweathered.		6	NQ												Rec 99% RQD 68%
5.18			7	NQ												Rec 100% RQD 75%
			8	NQ												Rec 100% RQD 93%
			9	NQ												Rec 100% RQD 82%
305.50	END OF BOREHOLE															
9.63	Notes: 1) This borehole forms part of Highway 522 Underpass Foundation Investigation. 2) Borehole located at station 10+000.0, on centreline as referenced to Highway 522.															





# RECORD OF BOREHOLE BH-3CF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 093 641.5 N, 313 805.6 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow Stem Augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE May 12, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION			
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	SHEAR STRENGTH: Cu, KPa						WATER CONTENT (%)		
								UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR	wp	w			wl		
							20	40	60	80	10	20	30	40		
315.37	GROUND SURFACE															
0.00	SAND & GRAVEL FILL, occasional lumps of asphalt, brown, moist. (compact)	Π	1	SS	29											
314.17			2	SS	16											
1.20	SAND, brown, wet, traces of organics, (possible FILL). (compact)		3	SS	9											
312.37			4	SS	3											
3.00	SILT, grey, trace of clay, wet. (very loose)															
311.37			5	SS	18											
4.00	SAND, with gravel sizes, brown, wet occasional cobbles. (compact)		6	SS	12											
			7	SS	19											
307.23	BIOTITE HORNBLende GNEISS, pinkish grey, good to excellent rock quality, slightly weathered to unweathered.		8	NQ												
8.14			9	NQ												
			10	NQ												
303.97	END OF BOREHOLE															
11.40	Notes: 1) This borehole forms part of Highway 522 Underpass Foundation Investigation. 2) Borehole located at station 9+966.8, ~5.0 m right of centreline as referenced to Highway 522.															



# RECORD OF BOREHOLE BH-4CF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 093 646.7 N, 313 870.9 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow Stem Augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE May 13, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT		UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20    40    60    80		wp — w — wl			
								SHEAR STRENGTH: Cu, KPa UNCONFINED      QUICK TRIAXIAL      FIELD VANE 					



# RECORD OF BOREHOLE BH-5CF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 093 653.0 N, 313 805.6 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow Stem Augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE May 13, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20	40	60	80					
315.24	GROUND SURFACE															
314.90	SAND & GRAVEL FILL	F					315									
0.30	SAND FILL, traces of organics, brown, moist. (loose to compact)	F	1	SS	7		314									
313.24			2	SS	14		313									2% 91% 7%
2.00	SILT, occasional seams of firm clay, brown to grey. (loose)		3	SS	9		312									0% 22% 78%
			4	SS	9		311									
311.24							310									
4.00	SAND & GRAVEL, brown, moist. (dense)		5	SS	41											8% 61% 31%
309.97	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER															
5.27	Notes: 1) This borehole forms part of Highway 522 Underpass Foundation Investigation. 2) Borehole located at station 9+967.3, offset ~6.0 m left of centreline as referenced to Highway 522. 3) Borehole caved wet at ~4.8 m depth on completion.															



# RECORD OF BOREHOLE C-1CF

1 OF 1

METRIC

W.P. 774-93-00

LOCATION 5 093 652.5 N, 313 870.7 E

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Dynamic cone test / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE May 13, 1998

CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60					
315.14 0.00	GROUND SURFACE Dynamic cone test only.														
311.48 3.66	END OF CONE TEST DUE TO BOUNCING REFUSAL ON BEDROCK OR BOULDER  Notes: 1) This cone test forms part of Highway 522 Underpass Foundation Investigation. 2) Cone test located at station 10+033.0, on centreline as referenced to Highway 522.														





# RECORD OF BOREHOLE C-3CF

1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 093 651.0 N, 313 857.8 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Dynamic cone test / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE May 25, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)				CONE PENETRATION TEST			PLASTIC LIMIT			NATURAL MOISTURE CONTENT			LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60	80	20	40	60	80	wp	w	wl	10	20	30	40			KN/m <sup>3</sup>
315.18 0.00	GROUND SURFACE Dynamic cone test only.																								
310.07 5.11	END OF CONE TEST DUE TO BOUNCING REFUSAL ON BEDROCK OR BOULDER  Notes: 1) This cone test forms part of Highway 522 Underpass Foundation Investigation. 2) Borehole located at station 10+020.0, on centreline as referenced to Highway 522. 3) Augered first 0.3 m through dense fill before driving cone test.																								



# RECORD OF BOREHOLE AP-1CF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 093 645.6 N, 313 841.0 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard Augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE May 13, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20	40	60	80					
315.93	GROUND SURFACE															
0.00 315.56 0.37	ASSUMED SAND & GRAVEL FILL															
314.41 1.52	ASSUMED SAND						315									
							314									
							313									
	ASSUMED SILT						312									
311.05 4.88	ASSUMED FILL						311									
310.93 5.00	END OF PROBE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER															
Notes: 1) This auger probe forms part of Highway 522 Underpass Foundation Investigation. 2) Auger probe located at station 10+002.5, offset 5.0 m left of centreline as referenced to Highway 522.																



# RECORD OF BOREHOLE AP-2CF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 093 654.7 N, 313 835.1 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard Augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE May 13, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20	40	60	80					
314.93	GROUND SURFACE															
314.93 0.30	ASSUMED FILL															
	ASSUMED SAND						314									
313.10 1.83	ASSUMED SILT						313									
							312									
							311									
							310									
309.84 5.09	ASSUMED TILL															
309.60 5.33	END OR PROBE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER															
Notes: 1) This auger probe forms part of Highway 522 Underpass Foundation Investigation. 2) Borehole located at station 9+997.5 offset 5.0 m left of centreline as referenced to Highway 522.																





# RECORD OF BOREHOLE AP-3CF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 093 652.3 N, 313 875.4 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard Augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE May 13, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80								
								SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL      FIELD VANE LAB SHEAR 20 40 60 80								
								WATER CONTENT (%) wp ——— w ——— wl 10 20 30 40								
315.16 0.00	GROUND SURFACE ASPHALT, ~45 mm over						315									
	ASSUMED SAND						314									
313.64 1.52							313									
	ASSUMED SILT						312									
311.35 3.81																
311.08 4.08	ASSUMED SILTY SAND & GRAVEL															
	END OF PROBE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER															
	Notes: 1) This auger probe forms part of Highway 522 Underpass Foundation Investigation. 2) Borehole located at station 10+037.0, on centreline as referenced to Highway 522.															



# RECORD OF BOREHOLE AP-4CF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 093 647.2 N, 313 800.8 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard Augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE May 13, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80								
								SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL      FIELD VANE LAB SHEAR								
315.74	GROUND SURFACE															
0.00	ASPHALT, ~45 mm over															
							315									
							314									
	ASSUMED SAND						313									
							312									
311.17							311									
4.57							310									
	ASSUMED SILT															
309.49																
6.25																
309.03	ASSUMED COBBLES															
6.71	END OF PROBE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER															
	Notes: 1) This auger probe forms part of Highway 522 Underpass Foundation Investigation. 2) Borehole located at station 9+963.0, on centreline as referenced to Highway 522.															



S07524G/C

**TABLE 1-1**  
**ROCK CORE DESCRIPTION**

BH#	Core Recovery				Core Description	
	Core #	Depth (m)	% CR*	% RQD**	Depth (m)	Description
<b>HIGHWAY 522 BRIDGE FOUNDATION</b>						
1-CF	3	3.72 to 4.72	100	98	3.72 to 6.89	<b>Biotite Horneblende Gneiss</b> - light grey to pinkish white, fine to medium grained, strong, unweathered, fractures very widely spaced, dipped at 80 to 90° from vertical, planar, smooth
	4	4.72 to 5.88	100	100		
	5	5.88 to 6.89	100	100		
2-CF	6	5.18 to 5.94	100	66	5.18 to 9.63	<b>Biotite Hornblende Gneiss</b> - light grey to pinkish white, with pegmatitic quartz inclusions, medium to coarse grained, strong, unweathered, fractures moderate to very close spread, dipping at 45° from vertical planar, smooth
	7	5.94 to 6.71	100	75		
	8	6.71 to 7.47	100	93		
	9	7.47 to 9.63	100	82		
3-CF	8	8.14 to 9.05	100	75	8.14 to 11.40	<b>Biotite Hornblende Gneiss (Garnetiferous)</b> , pinkish white to light grey, medium grained, strong unweathered, fractures moderately spaced, dipping at 0 to 10° and 80 to 90° from vertical, planar, smooth
	9	9.05 to 10.52	100	76		
	10	10.52 to 11.40	100	94		

\*CR

Core Recovery %

\*\*RQD

Rock Quality Designation %

**APPENDIX B**

---

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT

SAND

GRAVEL

FINE

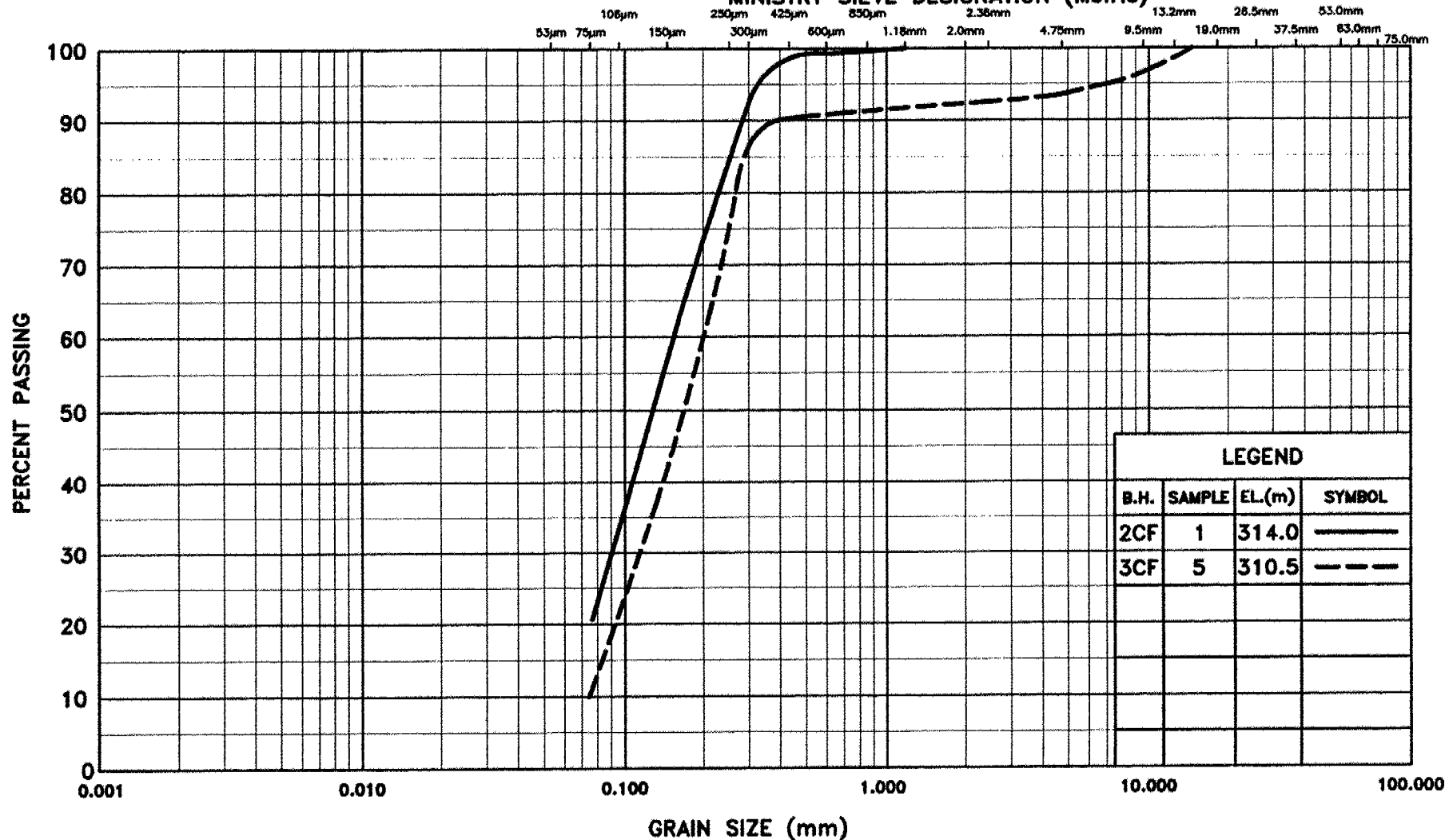
MEDIUM

COARSE

FINE

COARSE

MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION

FINE SAND

FIGURE 1

W.P. 774-93-00

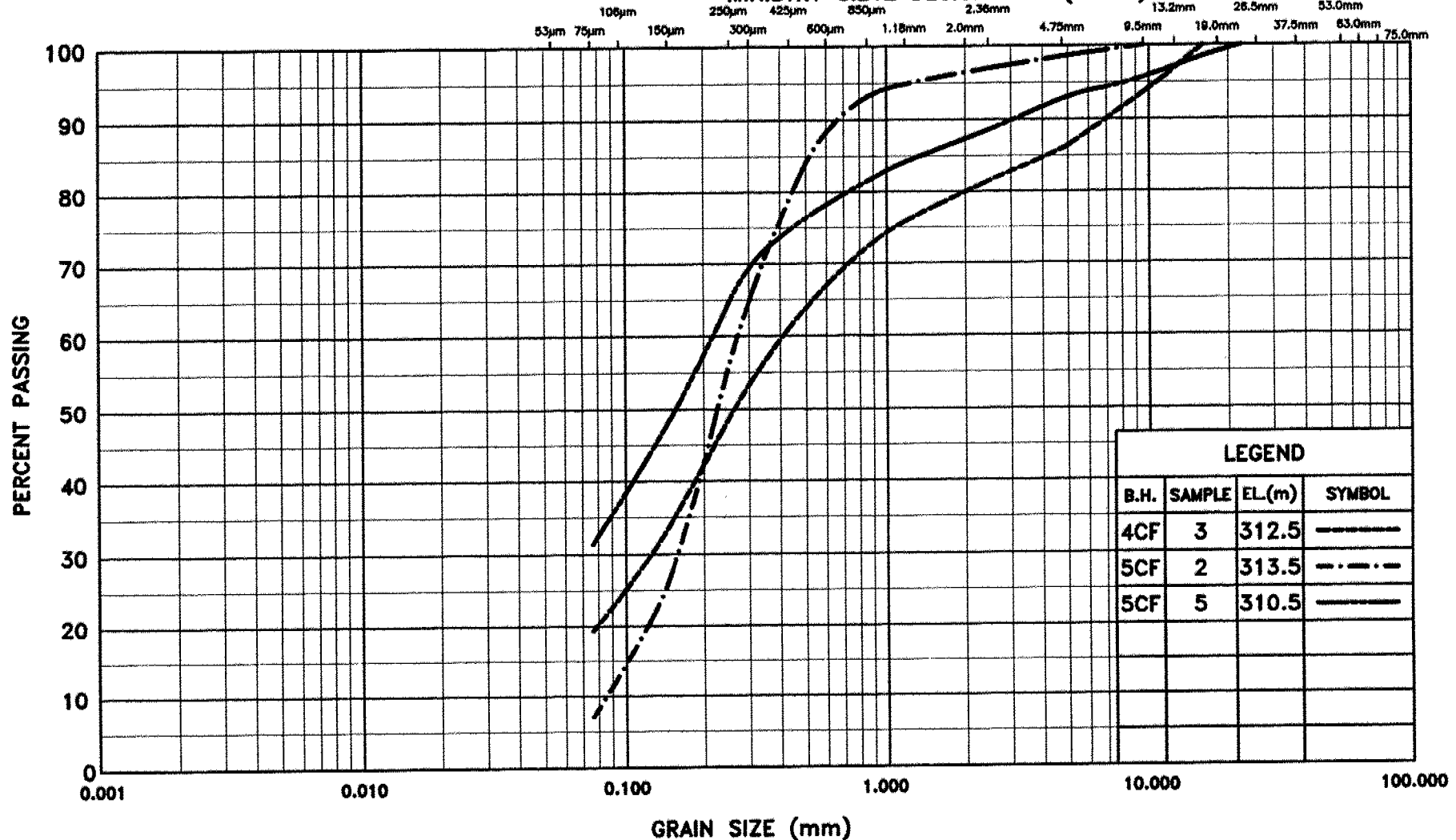


PROJ. No. S07524GCF

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION  
FINE/MEDIUM SAND, with gravel

FIGURE 2

W.P. 774-93-00

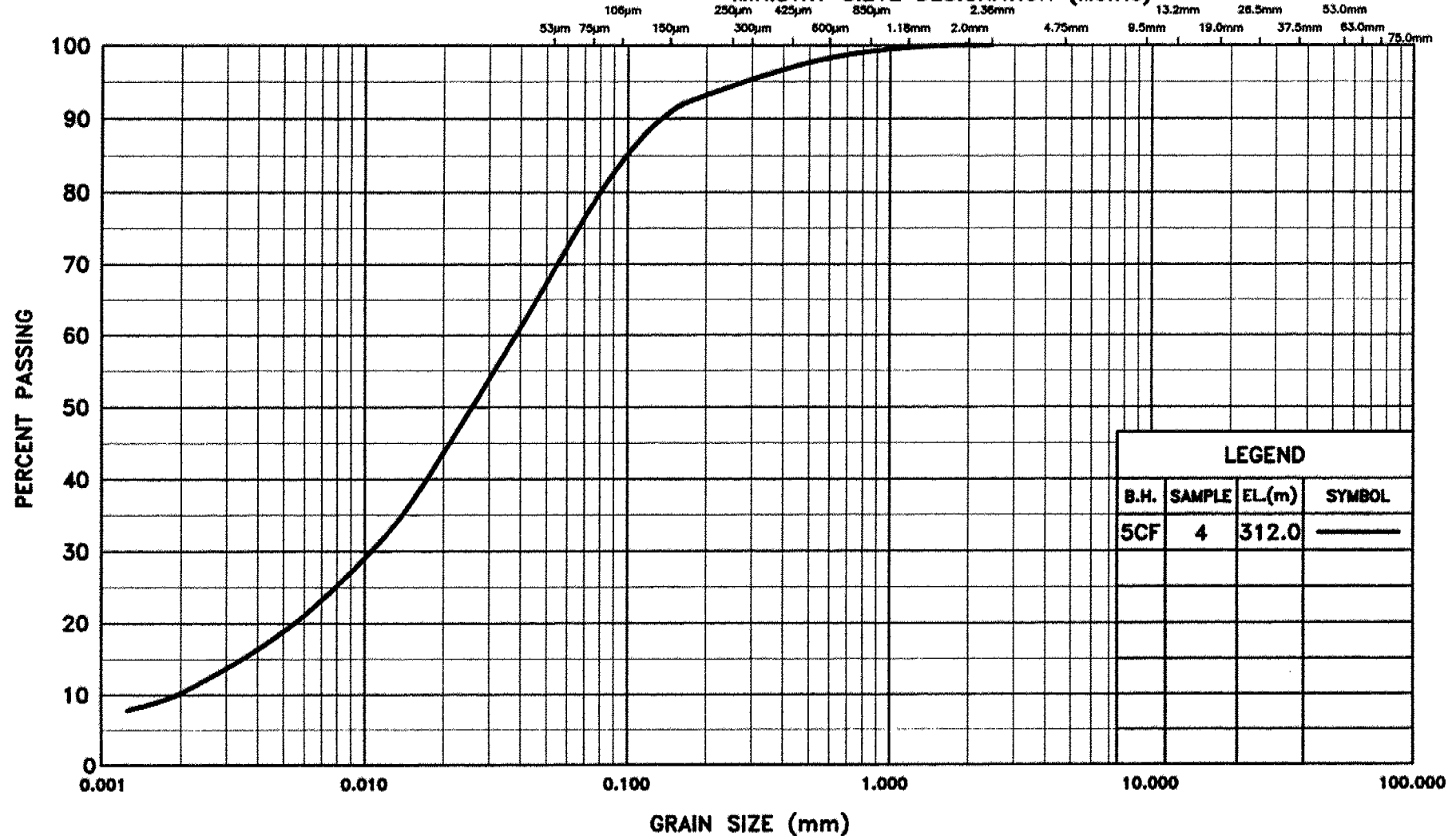


PROJ. No. S07524GCF

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

### GRAIN SIZE DISTRIBUTION

BH-5, SAMPLE 4: SILT, some sand, trace of clay

FIGURE 3

W.P. 774-93-00



PROJ. No. S07524GCF

# MEMORANDUM



To: E. Gallant, P. Eng.  
Senior Project Engineer  
Planning and Design, Northern Region

September 10, 1998

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

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

Re: Foundation Investigation Report Review  
Hwy 522, Bridge Structure  
Trout Creek By-Pass  
WP 774-93-00  
District 54, Sudbury

As requested we have reviewed draft reports prepared by Trow Consulting Engineers Ltd for the proposed Hwy 522 bridge structure and the Hwy 522 bridge structure approach embankments. 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. 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..

Two separate reports have been submitted: one for the bridge structure and one for the approach embankments. We have no comments for the Discussion and Recommendations component for the approach embankment report. Comments for the factual component of the report are applicable to both the bridge structure and approach embankment report.

## **Factual Component of Report(Hwy 522 Bridge Structure & Approach Embankment Reports)**

The Site Description and Geology, Investigation Procedure and Description of Subsurface and Groundwater Conditions have been partially addressed. The Consultant should, however, address the following:

1. Explanation of lab shear tests conducted on selected soil samples.
2. Borehole backfilling procedures.
3. Borehole Plan Drawing, stratigraphical profiles and drawings are not to MTO standards.
4. Section 1.4.1 - the report states that "...it is also believed to be fill or possibly alluvial in origin,...". The Consultant shall conclude whether the material is fill or not.(applicable to bridge report only)
5. Section 1.5 - Groundwater Conditions. Seasonal fluctuations are not addressed.



**Foundation Investigation & Design Report  
Bridge Structure & Approaches  
Trout Creek (Site 44-371S)  
SOUTHBOUND LANES  
Trout Creek By-Pass, King's Highway 11  
District 54, Sudbury, Ontario  
GWP No. 774-93-00**

Prepared For:

Marshall Macklin Monaghan  
80 Commerce Valley Drive East  
Thornhill, Ontario  
L3T 7N4

**Trow Consulting Engineers Ltd.**

Brampton, Calgary, Cambridge, Hamilton, Kingston, London, Markham, Montréal  
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F98179-C/G  
November 24, 1999

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## Preface

Work Project GWP 774-93-00 is one of a series of projects for the four lane expansion of Highway 11. This project involves the four lane design of Highway 11, from 4.0 km south of Highway 522, northerly for 7.9 km. It will result in the construction of a by-pass of the existing Highway 11 to the west of the Town of Trout Creek.

This highway section is located in the Townships of Laurier and South Himsworth, within the geographic District of Parry Sound, as shown on the Key Plan, Figure A1 in Appendix A. The project requires geotechnical input for the following major components:

- New pavement design for the entire length of the four lane by-pass, including associated service roads.
- New structure, Trout Creek South Interchange (underpass), Site 44-372.
- New structure, Trout Creek Northbound Lanes, Site 44-371N.
- **New structure, Trout Creek Southbound Lanes, Site 44-371S.**
- New structure, Highway 522 (underpass), Site 44-370.
- New structure, Trout Creek North Interchange (underpass), Site 44-369.

This report deals with the new bridge structure for the **southbound lanes** at the proposed Trout Creek crossing, Site 44-371S, as well as the approach fills. Separate reports will be submitted for the additional components.

Part 1 of this report, Foundation Investigation, describes the geotechnical investigation methodology and the results derived, as pertaining to the design parameters required for the bridge foundations and related earthworks.

Part 2 of this report, Engineering Discussion and Recommendations, addresses the geotechnical engineering design considerations pertaining to the bridge foundations and the approaches.

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## **Appendix A**

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## **Appendix B**

Borehole Logs and Rock Descriptions

## **Appendix C**

Results of Laboratory Testing (grain size, Atterberg Limits, consolidation tests)

## **Appendix D**

Stability Analysis Printouts

## **Rear Pockets**

- Drawing No. 1. Bridge Site Plan & Profile
- Drawing No. 2. Bridge Site Plan & Sections

## **Part 1 Foundation Investigation**

### **1.1 Introduction**

This report presents the results of a geotechnical foundation investigation by Trow Consulting Engineers Ltd. (Trow) for the proposed bridge structure and approaches for the southbound lanes (SBL) at the Highway 11, Trout Creek crossing at Site 44-371S.

An original investigation was completed at this site in June and July of 1998 for an originally proposed 105 m long ( $\pm$ ) three span structure. However, for technical reasons based on the results of the original investigation, as discussed more fully in Part 2 of this report, a longer (242 m), five span bridge was selected. Accordingly, a supplementary investigation was completed during September and November, 1998 which investigated the subsurface conditions at the revised pier and abutment locations of the new proposed five span structure, where conditions were not previously investigated as part of the original work.

This report contains the results of the supplementary investigation, plus the relevant results of the original investigation, compiled for the five span structure arrangement of the Highway 11, southbound lane crossing of Trout Creek. It is Trow's understanding that the 5 span structure will be located with the central span crossing Trout Creek. The structure will include an approximately 14 metre high south abutment, 22 metre high centre span, and 7 metre high north abutment. The span lengths will be approximately 40, 54, 54, 54, 40 metres in length, creating a bridge with a length of about 242 metres.

### **1.2 Site Description and Geological Setting**

The site is located in Lot 30, Concession 2, Township of South Himsworth, District of Parry Sound, along the banks of Trout Creek, about 2 km west of the Town of Trout Creek, and 750 metres south of Highway 552, as shown on Figure A1, in Appendix A.

Generally, the terrain at the site is sloping towards the creek and is well drained. However several drainage gullies run parallel and perpendicular to the creek, with steep embankments on either side of the gullies, creating a highly variable terrain in the vicinity of the proposed structure. The relief

can vary at slopes steeper than 1H:1V within the site. There are mature trees with heavy underbrush across the site.

According to OGS Maps 2544 and 2556, as well as Ontario Geological Survey Map P.3160 (Quaternary Geology, South River Area), the site is located in what is known as the Central Gneiss Belt, with mainly felsic igneous rocks of the Mesoproterozoic Group.

The overburden soils have been mapped as glaciolacustrine deposits consisting of laminated, rhythmically bedded (also referred to as varved) to massive silts and clays. The southernmost portion of the site edges onto an area mapped as a bedrock drift complex with relatively thin, primarily cohesionless soil cover (till).

## 1.3 Investigative Procedures

### 1.3.1 General

Part 1 of this report describes the investigative procedures used for the geotechnical assessment of the southbound lanes crossing of Trout Creek, within the Trout Creek By-Pass, King's Highway 11. Properties of the overburden soils were obtained by *in situ* and laboratory testing and the procedures employed during the investigation are described below.

### 1.3.2 Field Investigation

The initial field investigation, or explorations, for the originally proposed three span arrangement was carried out between June 9 and July 2, 1998. Originally these explorations investigated only the pier and abutment locations, as well as the immediate approach embankments. With the discovery of clay at this site additional explorations were advanced, while initially on site, to outline the vertical and areal extent of the clay within the approach embankments. A further investigation of the north approach was completed September 23 and 24, 1998. The supplementary investigation of the revised, five span, arrangement occurred between November 10 and 24, 1998.

The locations of the boreholes, cones and probes, completed as part of these investigations are shown on Figures A2a and A2b, in Appendix A, as well as on Drawings, No. 1 and No. 2, located in the pockets at the end of this report. These locations, as well as the surface elevations, were established

from the terrain model for the project, and/or a survey crew from Marshall Macklin Monaghan, and are referenced to geodetic datum.

The investigation of the piers and abutments of the five span arrangement includes 12 boreholes (BH-1OF to BH-10OF, BH-4EF, BH-20EF), eight dynamic cone penetration tests (C-1EF, C-6EF, C-11OF to C-14OF, C-4EF, C-7EF), and nine auger probes (AP-1OF to AP-8OF, AP-1EF). All of these explorations were advanced to auger refusal or to refusal of the penetration cone ( $N > 100$ ). Six of the boreholes were advanced into the bedrock to obtain core, as described below.

The investigation of the approaches included two additional boreholes (BH-17EP, BH-18EP) at the south approach and four additional boreholes (BH-20EP to BH-23EP) at the north approach. All of these boreholes were advanced to auger refusal, except for BH-23EP, which was terminated at a depth of about 14 m.

Other explorations completed in the vicinity of the SBL structure as part of the original three span investigation included nine boreholes, three dynamic cone penetration tests, and one auger probe, in addition to those explorations completed as part of the NBL investigations (see foundation report for the NBL structure).

The boreholes, cones and probes were advanced through the overburden soils using a Bombardier mounted CME-55 drill and a Dietric 50 drill, equipped with solid and hollow stem augers, and supplied by a soils drilling contractor, Master Soil Investigations Limited. At some of the borehole locations, a bulldozer was used to prepare the site for drill access.

Soil samples were obtained by using a 51 mm OD split-spoon sampler in conjunction with standard penetration tests at approximately 0.75 m and 1.5 m intervals. The standard penetration ( $N$ ) values, together with the blows from the dynamic cone penetration tests, were recorded and used to provide an assessment of the compactness or consistency of the overburden soils. Sampling and testing procedures were in general accordance with ASTM D1586.

Several undisturbed, nominal 50 mm diameter, 'Shelby' tube samples were also obtained in the cohesive deposits. Field vane testing was completed in the boreholes and the auger probes throughout the cohesive soils to measure the *in situ* undrained shear strength of the cohesive soils. The field vane used had dimensions of 152 mm long by 70 mm diameter, not including the pointed 45° point. Torque measurement was by using two calibrated scales on a calibrated lever arm threaded to the drill rods. The testing was in general accordance with ASTM D2573.

The recovered soil samples were taken to Trow's Sudbury laboratories for additional examination, identification and laboratory testing.

At each bridge foundation element, conventional rock coring techniques were used to advance one of the explorations approximately 3 metres into the underlying bedrock. At Pier WP5, two boreholes were cored. A BQ size core barrel and casing was used and core samples of the bedrock were retrieved for rock quality determination and classification. The seven boreholes advanced into the underlying bedrock included BH-2OF, BH-3OF, BH-5OF, BH-7OF, BH-8OF, BH-4EF, and BH-9OF.

Details of the soil and bedrock conditions encountered in the boreholes are included on the logs in the attached Appendix B.

### **1.3.3 Laboratory**

The laboratory testing program for selected soil samples consisted of the following:

- natural moisture content
- unit weight
- grain size distribution
- laboratory shear tests (lab vane, penetrometer)
- Atterberg limits
- 1-d consolidation test

The laboratory test results are summarized on the attached borehole logs in Appendix B and are also presented in Appendix C. Many of the results are also discussed in the following descriptive sections.

## **1.4 Subsurface Conditions**

The borehole locations are shown on the site plans, Figures A2a and A2b in Appendix A, as well as on Drawings, No. 1 and No. 2. Centerline soil profiles are also shown on the Figures and Drawing No. 1 and the soil cross sections at the locations of the foundation elements are shown on Drawing No. 2.





The borehole, auger probe and dynamic cone penetration (CPT) logs are provided in Appendix B. In general, the following main soil layers were encountered, with increasing depth:

- topsoil
- sand
- silty sand
- silty clay
- silty sand and gravel
- bedrock

A summary of the soil strata encountered in the boreholes, and inferred from the probes and dynamic cone penetration tests, is presented below.

#### **1.4.1 Topsoil**

The majority of boreholes encountered a surficial layer of up to about 300 mm of topsoil. At some of the borehole locations, a bulldozer was used to prepare the site for drill access, thus the topsoil in these areas was removed in the process and the logs do not indicate the presence of topsoil.

#### **1.4.2 Sand**

A 300 mm to 600 mm thick layer of loose to compact, fine to medium grained brown sand was encountered beneath the topsoil at Boreholes, BH-9OF and BH-10OF, at the location of the north abutment, and was underlain by silty clay, as described below.

#### **1.4.3 Silty Sand**

Brown to grey, very loose to loose (N-values of 1 to 5) silty sand was encountered beneath the topsoil in Boreholes, BH-6OF and BH-7OF at the location of Pier WP4, adjacent to the north bank of Trout Creek. The soil contained organics such as roots and pieces of wood and may be a recent stream alluvium. At BH-6OF, the thickness was about 4 m and was underlain by the silty sand and gravel, described below. In BH-7OF, the silty sand was underlain by silt clay at a depth of about 2.3 m.

#### **1.4.4 Silty Clay**

Beneath the upper sand, topsoil, or silty sand, a stratum of silty clay was encountered as the principal soil in all boreholes at the locations of the foundation elements except at the south abutment (WP1),



BH-4OF at Pier WP2, and BH-6OF at Pier WP4. At the south abutment, the native soil consists of silty sand and gravel, described below. Silty sand and gravel was also encountered in BH-4OF (Pier WP2). In BH-6OF (Pier WP4), silty sand, as described above, was encountered.

The silty clay, where encountered, is relatively thin (1.5 m) in BH-3OF at the location of Pier WP2, and increases in thickness to between about 12 m to 14 m on the north side of Trout Creek, in the area of the north abutment and approach.

Generally, the silty clay is thinly laminated with silt (varved). The individual layers, or laminations, varied in thickness from a few millimetres to a few centimetres, but in general were less than about one centimetre. The proportions of clay to silt and clayey silt varied also, but in general the clay portion dominated.

The natural moisture content of the clay varies from about 20% to over 50% (depending on the silt content) and consistency. Atterberg limit determinations provided the following approximate ranges: Plastic Limit, 18 to 23; Liquid Limit, 28 to 50; Plasticity Index, 8 to 30. These data indicate that, in general, the clay can be described as a medium plasticity silty clay (CI). Locally, clayey silt soil (CL) was encountered. Typically, the silty clay is drier near the top and bottom, but there does not appear to be a depth relationship relative to the Atterberg Limits. The laboratory test data are shown on the borehole logs, on Figure A3 in Appendix A, and in Appendix C.

Standard penetration test (SPT) values ranging from about 1 to over 20 were obtained in the silty clay. The higher values were generally obtained within the upper metre, or so. *In situ* field vanes and laboratory shear vane testing, as well as the SPT values, indicate that the silty clay has a stiff to very stiff crust with undrained shear strengths exceeding 100 kPa. The silty clay consistency, however, reduces with depth to soft to firm to stiff, with undrained shear strengths of about 20 kPa to 70 kPa, at depths of about 3 m to 5 m. Sensitivity, the measure of peak shear strength to remoulded shear strength, ranged from about 2 to 16, with an average of about 7, indicating the clay is moderately sensitive. The undrained shear strength profile which includes shear strength data from all boreholes is shown on Figure A4. The strength profile shown on Figure A3 is based on the strength data from boreholes at the locations of the abutments and the design profile assumed is based on the actual vane test results and the SPT values.

Based on all the above, and with reference to Figure A3, it is evident that the clay is heavily overconsolidated in the upper 3 m to 4 m, becoming moderately to lightly overconsolidated with increasing depth. The preconsolidation pressure near the top of the stratum is estimated at about 400 kPa, on average. The overconsolidation ratio (OCR) is estimated as ranging from over 30 near

the top to about 3 at a depth of about 3 m. Thereafter with increasing depth, the OCR decreases gradually to about 1.6 at a depth of 14 m.

A one-dimensional consolidation test was performed on a sample of the silty clay extruded from a thin walled Shelby tube, obtained from BH-21EP. The results are presented graphically and numerically in Appendix C. The data are also summarized below in Table 1-1, along with the value ranges used in our subsequent analyses.

<b>Table 1-1. Consolidation Parameters for Silty Clay</b>		
	<b>BH-21EP, 3 m</b>	<b>Values for Analyses</b>
Compression ratio, $C_c' (= C_c/(1+e_0))$	0.08	0.08 - 0.20
Recompression ratio, $C_r' (= C_r/(1+e_0))$	0.006	0.008 - 0.02
Coefficient of consolidation (recompression), $C_{vr}$	40	25 - 60 (avg 40)
Coefficient of consolidation (virgin), $C_v$	7	5 - 12 (avg 8)
Coefficient of secondary compression, $C_{\alpha\epsilon}^*$	0.003 - 0.004	0.003 - 0.005
Notes: Coefficients of consolidation in units of $m^2/year$		
* $C_{\alpha\epsilon} = \Delta H/H_0$ (= secondary compression ratio, in some literature)		

#### 1.4.5 Silty Sand and Gravel

A basal deposit of loose to dense, brown silty sand and gravel was encountered above the bedrock (or refusal) surface in almost all boreholes. Standard penetration indices (N-values) ranged from about 5 to over 100. Its thickness varied from less than about 0.5 m to over 3.5 m. Where bedrock is relatively shallow, such as at the south abutment, the sand and gravel was encountered as the uppermost soil overlying the bedrock.

#### 1.4.6 Bedrock

The presence of bedrock was established by retrieving "BQ" size cores at each of the six foundation element locations, for depths of between about 3.1 m and 3.5 m. Detailed descriptions of the rock are presented in Table 1-1 in Appendix B, following the borehole logs. Generally, the bedrock can be described as a pink and light grey, biotite-hornblende gneiss. The rock is strong and unweathered for the most part.

Rock core recovery was 100% for all runs and the Rock Quality Designation (RQD) values for individual core runs ranged from about 35% to 96%. The average RQD for the rock core recovered was about 80%, based on the 23 core runs. In accordance with the MTO classification system, the rock quality can be described as poor to excellent, with an average of fair. It is noted that the RQD values are likely conservative; it is expected that higher values would be obtained using NQ core.

Table 1-2, below, lists the bedrock depths and elevations as well as those of refusal, at the locations of each of the six foundation elements. It can be seen that the bedrock and refusal depths and elevations are quite variable, even within short distances at the individual element locations. Refusal (to augering or dynamic cone penetration testing (CPT)) is inferred to be due to probable bedrock, but it is noted that refusal may be due to cobbles, boulders, or very dense soil. The bedrock depths and elevations have been positively established only at the locations where the bedrock has been cored.

<b>Table 1-2. Bedrock and Refusal Depths and Elevations at Boreholes</b>					
<b>Location</b>	<b>Borehole</b>	<b>Orig. Gr. Elev. (m)</b>	<b>Bedrock or Refusal Elev (m)</b>	<b>Overburden Thickness (m)</b>	<b>Comment RQD (by run)</b>
<b>WP1 South Abutment</b>	BH-1OF	310.76	307.56	3.20	auger refusal
	BH-2OF	311.74	308.62	3.12	B/R cored 80%, 80%, 90%
	AP-1OF	312.05	309.15	2.90	auger refusal
	AP-2OF	312.05	310.17	1.68	auger refusal
	AP-3OF	311.24	308.56	2.68	auger refusal
	AP-4OF	311.83	309.09	2.74	auger refusal
<b>WP2 Pier</b>	BH-3OF	305.90	302.55	3.35	B/R cored 75%, 97%
	BH-4OF	306.84	303.94	2.90	auger refusal
	AP-5OF	306.65	302.54	4.11	auger refusal
	AP-6OF	307.39	304.71	2.68	auger refusal
	AP-7OF	306.10	302.38	3.72	auger refusal
	AP-8OF	305.77	302.72	3.05	auger refusal
<b>WP3 Pier</b>	BH-5OF	303.79	298.52	5.27	B/R cored 35%, 50%, 60%
	AP&C-1EF	301.02	295.84	5.18	auger/cone refusal
	C-6EF	300.50	295.90	4.60	CPT refusal

Table 1-2. Bedrock and Refusal Depths and Elevations at Boreholes					
Location	Borehole	Orig. Gr. Elev. (m)	Bedrock or Refusal Elev (m)	Overburden Thickness (m)	Comment RQD (by run)
WP4 Pier	BH-6OF	299.89	293.95	5.94	auger refusal
	BH-7OF	300.22	293.90	6.32	B/R cored 70%, 92%
	C-11OF	300.01	293.33	6.68	CPT refusal
	C-12OF	300.16	293.94	6.22	CPT refusal
	C-13OF	300.57	293.96	6.61	CPT refusal
	C-14OF	300.12	294.06	6.06	CPT refusal
WP5 Pier	BH-8OF	303.23	294.94	8.29	B/R cored 65%, 78%
	BH-4EF	305.36	295.45	9.91	B/R cored 80%, 90%
	BH-20EF	303.20	293.90	9.30	auger refusal
	C-7EF	303.70	294.51	9.19	CPT refusal
WP6 North Abutment	BH-9OF	312.07	297.65	14.42	B/R cored 82%, 93%
	BH-10OF	311.62	297.96	13.66	auger refusal

## 1.5 Groundwater Conditions

Information regarding the groundwater levels at the site was obtained by measuring the water levels in the open boreholes after completion of drilling and by tactile examination of the recovered samples. The measured or inferred groundwater levels are shown on the borehole logs and the stratigraphic sections. In general, the groundwater table at the times of the field work was between about 1 m and 4 m in depth. It appears to follow the topography and this suggests that local subsurface drainage would be towards Trout Creek.

## Part 2 Engineering Discussion and Recommendations

### 2.1 Introduction

The following subsection addresses the geotechnical design considerations pertaining to the proposed five span bridge for the Southbound Lanes crossing of Trout Creek, as well as the approaches.

An original investigation was completed at this site in June and July of 1998 for an originally proposed 105 m long ( $\pm$ ) three span structure and revealed significant clay deposits on either side of Trout Creek which caused concern for the stability of the approach embankments, which were about 20 m high. Various design alternatives were considered including removal of the clay to the underlying bedrock, extensive berming of the embankments, the use of lightweight fill within the embankments, retaining walls, and a lengthening of the structure to limit the impact of the clay deposits. A subsequent cost benefit analysis of the design alternatives proposed by Trow, performed by Marshall Macklin Monaghan, indicated the preferred alternative was to lengthen the structure.

It was considered that the lengthened structure would effectively span the clay deposit, so that the approach embankments could either be located on a thinner or absent clay layer, or alternatively the lengthened structure would extend to the point where the height of the approach embankments could be reduced, and thus reduce or eliminate the complications encountered by placing high approach embankments on thick clay deposits.

Upon MTO acceptance of the lengthened structure alternative, a supplementary investigation was completed during September and November, 1998 as described in Part 1 of this report. The supplementary investigation examined the soil conditions at the revised pier and abutment locations of the new proposed five span structure, where conditions were not previously investigated as part of the original work.

The five span bridge is proposed to carry southbound Highway 11 traffic over Trout Creek and its valley. It is Trow's understanding that the bridge will be located with the central span crossing Trout Creek. The structure will include an approximately 14 metre high south abutment, 22 metre high centre span, and 7 metre high north abutment. The span lengths will be approximately 40, 54, 54, 54, 40 metres in length, creating a bridge with a length of about 242 metres.

It is understood that the project is scheduled for completion within a two year time frame. Accordingly, some of the discussions and design recommendations presented herein are based on this time frame.

## **2.2 Foundations**

In general, because of the presence of loose to compact sand and silt and relatively weak and compressible clay at the locations of the foundation elements, it is not considered feasible to support the proposed bridge foundations on spread footings constructed on the native mineral soils. For all foundation elements, driven steel H-piles are considered to be the preferred alternative. Alternate types of foundations may, however, be considered for the support of the bridge piers and abutments. The alternate types that are considered applicable to the site and proposed layout include large diameter reinforced concrete caissons and spread footings on rock or structural fill. Not all of these foundation types are applicable to all six foundation elements.

The following sections present the foundation design recommendations for the six foundation elements of the proposed bridge.

### **2.2.1 Steel H-Piles (all locations)**

All abutments and piers are recommended to be supported on steel H-piles driven to the bedrock surface, using the ULS capacities for HP310x110 and HP310x132 sections, as given in Table 2-1, below.

Current design practice requires that consideration be given to downdrag loads that may be generated as a result of soil consolidation due to fills, groundwater table fluctuations and the soil stresses induced by pile installation. We consider that, even after primary consolidation of the clay soils due to the fill placement is complete, the potential exists for downdrag loads due to events such as significant groundwater lowering and the long term process of secondary compression. At this site, secondary compression is the likely mechanism. Secondary compression of the clay foundation soils is discussed further in a subsequent section of this report. Accordingly, we have considered the effects of downdrag loads on the pile capacities given in the following sections, where applicable.

**Table 2-1. H-Pile Design Pile Capacities (kN)**

	HP 310x110			HP 310x132		
Factored Structural Capacity (OHBDC)	3285			3890		
Factored Axial Resistance (MTO*)	2000			2300 (est)		
Pile Location ---->	South Abutment W1	Piers W2, W3, W4, W5	North Abutment W6	South Abutment W1	Piers W2, W3, W4, W5	North Abutment W6
Factored Downdrag Load	-	-	1200	-	-	1500
Factored Axial Capacity at ULS**	2000	2000	2000	2300	2300	2300
Notes: MTO* = Structural Office Policy Memo 98-01, April 15, 1998 ** Factored axial capacity at ULS is the lesser of, a) factored structural capacity minus factored drag load, or b) factored axial resistance. SLS capacity not applicable to piles driven to bedrock						

The axial capacity at SLS is not applicable to steel piles driven to bedrock because very high loads, in excess of the ULS capacity, are required to produce unacceptable deformations. The pile structural capacity will govern.

The capacities given are axial capacities, for both vertical and inclined piles. No reduction is required for inclined piles. Notwithstanding the above, the horizontal and vertical loads acting on all piles must be resolved axially, such that the vector sum of these loads does not exceed the ULS capacity provided above.

For design purposes, the modulus of horizontal subgrade reaction for steel H-piles can be taken as 8,000 kN/m<sup>3</sup> for the cohesive soils (silty clays) and 40,000 kN/m<sup>3</sup> for the cohesionless soils. It is expected, however, that inclined piles will be required to accommodate the lateral loads. These can be designed using the same axial capacities given in Table 2-1.

A minimum soil embedment depth of 3 m below the pile cap is recommended. Pile caps should be provided with at least 2 m of soil cover for frost protection. Local grade raises may be required in order to provide this cover.

If the underside of the pile caps cannot be provided with a minimum of 2 m earth cover, insulation will be required. Insulation should consist of rigid board extruded polystyrene, meeting CAN/CGSB-51.20-M87 (Type 4), such as DOW SM™. The insulation is recommended to be placed



beneath the pile caps, prior to placement of concrete. Since the insulation will not carry any significant loads, high strength/low compressibility insulation (such as *DOW HI40™*, etc.) is not required. Products other than those made by *DOW CORNING* may be used, provided they meet the above noted specification.

The insulation thickness and lateral extension beyond the edges of the pile caps will depend on the depth of placement (i.e., underside of pile cap), in accordance with Table 2-2, below. A minimum soil cover of 300 mm is recommended over the top of the insulation.

Table 2-2. Pile Cap Insulation Dimensions		
Depth (mm)	Thickness (mm)	Lateral Extension (mm)
500	90	1500
1000	50	1000
1500	25	500

As discussed in following subsections of this report, substantial settlements of the north approach fill will occur as will some associated lateral strains. Accordingly, piles should not be installed until after the majority of the fill settlement has occurred, based on monitoring during construction. This is particularly important for inclined piles, that would be subjected to bending stresses.

All piles should be driven to bedrock. Pile tip elevations can be estimated from Table 1-2 which provides the bedrock or refusal elevations encountered at the boreholes drilled at the various foundation elements. The boreholes indicate that the bedrock elevations are quite erratic and the potential for irregular steeply sloping bedrock at the foundation locations is considered to be high at most locations. As such, problems may arise during pile seating. At some locations, the piles may have a tendency to skip over the bedrock surface resulting in alignment problems and deeper penetration. In the event that this problem occurs, somewhat longer piles may be required and, in some cases, piles may have to be added or replaced.

To minimize seating difficulties, rock injector points should be used to facilitate proper seating. All piles must be fitted with reinforcing plates welded to the flanges as per OPSD 3301 to minimize pile damage. It is recommended that, during pile driving and upon initial contact with the bedrock, the

pile driving energy should be reduced and subsequently increased incrementally until the piles have been sufficiently seated.

### 2.2.2 Concrete Caissons (all locations)

As an alternative foundation system, concrete caissons installed on or into the bedrock can be considered for all locations. However, they will likely only be practical for the foundations at the south abutment (WP1) and Piers WP2 and WP3, where the bedrock surface is generally within about 1.7 m to 5.3 m below original grade. The load capacity will be derived by end bearing, in accordance with the values given in Table 2-3. As for steel H-piles, the effects of downdrag loads must be considered.

<b>Table 2-3. Concrete Caisson Design Capacities</b>	
Factored Downdrag Load (north abutment only)*	3800 kN per m of pile diameter
Factored Bearing Resistance at ULS	8000 kPa
Notes: SLS resistance not applicable to caissons on bedrock * Factored Downdrag Load to be applied to the factored dead loads	

In order to provide an adequate socket, the caisson should be installed at least one pile diameter into the bedrock, or be heavily dowelled. While these units can provide high capacities, because of the irregular and potentially steeply sloping bedrock surface expected at this site, caisson installation may prove difficult, thus expensive. This is complicated by the fact that in most cases the bedrock is overlain by silty sand and gravel that may create dewatering and stability problems during work at the base of the caissons.

### 2.2.3 Spread Footings

Spread footings for the south abutment and Piers, WP2 and WP3, can be considered for design and construction on bedrock or on structural fill, as described in the following sections. We do not consider spread footings to be feasible alternatives for the remaining foundation elements, either because of a great depth of excavation or low SLS bearing resistance for footings near the ground surface.

### 2.2.3.1 Spread Footings on Bedrock (WP1, WP2, WP3)

An alternate foundation for the south abutment (WP1) and Piers WP2 and WP3 is a spread footing constructed on the bedrock. The elevations of footings can be estimated from Table 1-2. This alternative is not considered practical for the other locations because of the excavation depths exceeding 6 m. The factored bearing resistance at ULS for footings on unweathered bedrock is **8,000 kPa**. The bearing resistance at SLS does not apply because of the much higher pressures required to produce unacceptable deformations.

In order to evaluate the sliding resistance of spread footings on bedrock, the unfactored coefficients of friction for mass concrete on clean bedrock can be taken as 0.7. If the factored resistance against sliding failure is inadequate based on friction only, steel dowels will be required for footings on bedrock.

The ULS capacity of spread footings must be reduced for the effects of inclined loads. The reduction factors given in Table 2-4, below, can be used for footings on bedrock. Interpolation is possible. These factors must be applied to the ULS bearing resistance given previously.

<b>Table 2-4. ULS Reduction Factors for Inclined Loads on Spread Footings</b>	
<b>Ratio of Horizontal to Vertical Load</b>	<b>Footings on Rock</b>
0.1	0.86
0.2	0.76
0.3	0.66
0.4	0.58
The ULS reduction factors for inclined loads have been taken from Figure 6-8.4.2 of the OHBDC	

### 2.2.3.2 Spread Footings on Structural Fill (WP1 - South Abutment)

Spread footings can be designed for construction on structural fill at the south abutment. Structural fill should be constructed after removal of the overburden soils, where shallow, or it can be placed on the stripped native soils, as described below. For spread footing support, it is recommended that the structural fill consist of OPSS Granular A, placed in small lifts and adequately compacted (100%

standard Proctor). Alternatively, a relatively fine well graded rockfill, with a maximum size of 300 mm can be used. This finer graded rockfill should be placed in lifts limited to about 500 mm and adequately compacted with heavy vibratory rollers (minimum 6 passes, 10 tonnes).

At the south abutment, a spread footing abutment foundation, if considered, should be designed for construction in the approach fill, with a depth of about 2 m below the slope face. This would place the base of the footing at a distance of about 7 m to 8 m above original ground, at an elevation of about 318 m.

If the native granular soils are left in place, the structural fill supporting the foundation should have a thickness at least equal to the width of the footing. In addition, the structural fill should be constructed to occupy a zone, down and out from the footing edges at a slope of no steeper than 1H:2V, in order to accommodate the footing stresses.

The ULS resistance values given in Table 2-5, below, can be used for design. For 25 mm immediate settlement, the SLS resistance is greater than at ULS. Accordingly, the ULS resistance governs the design. Consolidation settlement is not considered an issue at this location since cohesive soils were not encountered in the investigation.

<b>Table 2-5. Spread Footing ULS and SLS Bearing Resistance</b>	
Factored Bearing Resistance at ULS on Unweathered Bedrock <sup>1</sup>	8000 kPa
Factored Bearing Resistance at ULS on Structural Fill <sup>2</sup>	1000 kPa
Bearing Resistance at SLS - Initial Elastic Settlement - Structural fill <sup>3</sup>	> 1000 kPa
Notes: 1. SLS resistance not applicable to footings on bedrock 2. Thickness of structural fill greater than footing width 3. Structural fill placed on native granular soils after removal of organics/topsoil, and bottom of footing about 7 m to 8 m above original ground (approx el. 318 m).	

For the determination of the sliding resistance of spread footings, the unfactored coefficient of friction for mass concrete on granular structural fill can be taken as 0.6. If the factored resistance against sliding failure is inadequate based on friction only, a soil key can be considered for footings on structural fill, making use of the passive soil resistance. Passive earth pressure coefficients are provided in Section 2.3.

The ULS resistance of spread footings must be reduced for the effects of inclined loads. For footings on granular structural fill, Figure 6-8.4.2 of the OHBDC may be used for the applicable footing depth to effective width ratio. These factors must be applied to the ULS resistance given previously in Table 2-5.

## 2.3 Backfill

Backfill for abutments or retaining walls should consist of free draining granular materials such as Granular A, Granular B, or rock fill. Computation of earth pressures shall be in accordance with Section 6.7.4 of the Ontario Highway Bridge Design Code. Unfactored properties for backfill materials are provided in the following Table 2-6.

<b>Table 2-6. Fill Types and Unfactored Geotechnical Properties</b>					
<b>Material</b>	<b>Friction Angle, <math>\phi'</math></b>	<b><math>\gamma</math> (kN/m<sup>3</sup>)</b>	<b><math>K_A</math></b>	<b><math>K_P</math></b>	<b><math>K_0</math></b>
Granular A	35 degrees	22	0.27	3.7	0.43
Granular B	30 degrees	21	0.33	3	0.5
Rock Fill	42 degrees	20	0.2	5	0.33
Note: Values given for $K_A$ and $K_P$ are for horizontal backfill, and will vary for other configurations. $K_A$ is the earth pressure coefficient corresponding to the active state. $K_P$ is the earth pressure coefficient corresponding to the passive state. $K_0$ is the earth pressure coefficient at rest.					

The mobilization of full active or passive resistance requires a measurable and perhaps significant wall movement or rotation, in general accordance with that indicated in Figure C6-7.1 of the OHBDC. Therefore, unless the structural element can tolerate these deflections, the at rest earth pressure should be used in design. Alternatively, partially mobilized active and passive earth pressure coefficients may be used, in accordance with the deflections given in the Figure.

The effects of compaction surcharge should be taken into account in the calculation of active and at rest earth pressures. In accordance with Section 6-7.4.3 of the OHBDC, the lateral pressure due to compaction should be taken as at least 16 kPa at the surface, and its magnitude should be assumed to diminish linearly with depth to zero at the depth where the active (or at rest) pressure is equal to

16 kPa. This pressure distribution should be added to the calculated active (or at rest) pressure. Notwithstanding, lighter compaction equipment and smaller lifts should be used adjacent to walls to prevent overstressing.

If rock fill is used as backfill behind abutments, the particle size should be limited to no greater than 300 mm and the backfill must be placed carefully in a manner that does not cause damage to the abutments or other structural components of the bridge.

Weep holes should be provided in the abutments and a filter wrapped, 150 mm diameter perforated PVC drainage pipe should be installed behind the abutments, at or slightly above the base.

## 2.4 Excavations and Dewatering

All work associated with design and construction relative to excavations shall be in accordance with Part III of Ont. Reg. 213/91 of the Occupational Health and Safety Act. Where the width of the base of the excavation is less than twice its depth, conformance with this regulation is required.

The existing organic zone (topsoil) with a depth ranging to about 300 mm will have to be removed from beneath the approach fill footprints. For pile caps at the pier locations, excavations of at least 2 m depth will likely be required, in order to provide the recommended frost cover. Since the ground surface is uneven at the pier locations, deeper excavations will be required to place the pile cap at a common elevation, unless the finished site grades are raised with fill or the caps are provided with insulation. Based on the borehole information, excavations would generally proceed through the loose to compact upper sand and into the stiff to firm silty clay at most locations.

At the locations of Piers WP2, WP3 and WP5, excavations should be relatively straight forward, since they will likely terminate above the prevailing groundwater level. The upper sand and silty clay is considered a Type 3 soil and excavations should be cut back to at least 1H:1V. If minor groundwater seepage occurs and loosens/softens the soil, flatter slopes will be required. Dewatering of the excavations should be possible by pumping from sumps within the excavations.

At the location of Pier WP4, on the north side of Trout Creek, the excavation will likely be carried out entirely within the loose to compact silty sand and would terminate close to or below the prevailing water level of Trout Creek. These soils would then be classified as Type 4 soils and excavations should be sloped back at 3H:1V. Pumping from sumps within the excavation should



suffice here as well, in order to maintain a safe and workable area, although more aggressive effort will likely be required. In order to stabilize the base of the excavation if it becomes loosened due to groundwater infiltration, a 300 mm layer of crushed clear stone may be required to improve working conditions.

If the native soils are to be removed and replaced with structural fill at the south abutment, or to place foundations on the bedrock at Piers WP2 and WP3, excavations of between about 1.7 m and 5.3 m will be required, based on the results of the investigation. The soils to be excavated will consist of sand, and silty sand and gravel, as well as silty clay in some locations. The water table would be encountered well above the 5.3 m depth. Accordingly, excavations in this area should be sloped back at 3H:1V, or flatter. Aggressive pumping from sumps will likely be required. Alternatively, sheeted and braced excavations could be considered, but this may prove problematic because of the variable bedrock surface.

Excavations carried out within granular structural fill in the approaches can likely be completed using a 1H:1V cut since it will be above the water table.

It is recommended that a non-standard special provision (NSSP) for dewatering be provided in the contract documents.

## **2.5 Bridge Approach Fills**

The construction of the bridge approaches will require embankment fills of up to about 14 m height at the south abutment and about 6 m height at the north abutment. The soils at the south abutment are predominantly granular, with a maximum thickness of about 3.5 m, based on the results of the investigation. At the north abutment, the principal soils consist of stiff to firm silty clay, to depths of up to about 14 m. The two principal design and construction considerations are embankment stability and consolidation settlement. These two issues are discussed in the following sections.

In all of the following discussions, it is assumed that all organic material (topsoil) is removed from beneath the embankments and the embankments are constructed on the native mineral soils. Fill heights should be measured from the top of the native mineral soil.

## 2.5.1 Embankment Stability

Highway embankments can be constructed using structural fill of various acceptable soil materials. Typically, however, in this part of the province they are constructed using rockfill as the principal component. A typical MTO section will consist of a minimum (steepest) slope of 1.25H to 1V to a maximum height of 6 m. For embankment sections greater than 6 m height, a 2 m wide horizontal bench is required before the next 6 m, 1.25H to 1V lift is constructed. A 14 m crest width has also been assumed, based on the drawings provided.

Slope stability analyses have been performed using SLOPE/W (ver. 3.x), based on Bishop's Method, using total stress parameters, for the cohesive soils. This analysis would apply to rapid construction (short term stability) and factors of safety can be expected to increase with time. The undrained shear strength profile shown on Figure A3 was used to provide the shear strength parameters for the silty clay soils. Table 2-7, below, lists the parameters used.

<b>Table 2-7. Geotechnical Parameters for Slope Stability Analyses</b>			
	$\gamma_{total}$ (kN/m <sup>3</sup> )	$c_u$ (kPa)	$\phi'$
Rockfill	20	0	42°
Sand	20	0	32°
Silty clay	19.5	variable (see Fig. A3)	0
Sand and Gravel	21.5	0	35°
Notes: Embankment crest width 14 m.			

Appendix D contains many of the SLOPE/W graphical printouts, for the various analyses performed and discussed below.

### 2.5.1.1 South Approach Stability

The south approach embankment will rest on essentially cohesionless soils. The results of the analyses performed on the embankment cross-sections, for an approach height of 14 m above original ground, indicates that the factor of safety against a foundation failure is about 1.7. Accordingly, the section as proposed is acceptable.



### 2.5.1.2 North Approach Stability

The north approach will rest on essentially cohesive soils. The results of the total stress analyses performed on the embankment cross-sections, for heights of 5 m to 8 m, resulted in safety factors ranging from about 2.9 to 2.0, which are considered more than adequate.

An analysis was also performed on the longitudinal design section (front slope) as shown on Figure A2b and Drawing No. 2. The calculated factor of safety for this slope is 1.40, which is considered adequate.

### 2.5.2 Consolidation Settlement of Approach Embankments (North Approach)

Long term consolidation settlement will result only at the north approach since cohesive soils are present as the principal foundation soils. The soils at the south approach are primarily cohesionless and consolidation settlement will not occur. Accordingly, the following discussion will apply only to the north approach.

#### 2.5.2.1 Magnitudes of Consolidation Settlement

For the north approach embankment, consolidation settlement calculations have been performed using the effective stress profiles shown on Figure A3 and compression ratios ( $C_c' = C_c/[1+e_0]$ ) ranging from 0.08 to 0.20. The values used were established from the consolidation test data, previous experience at the north and south interchanges of this project, as well as from geotechnical literature. Recompression indices ( $C_r' = C_r/[1+e_0]$ ) ranging from about 0.008 to 0.02 were used. The stress increases in the foundation soils due to the embankment loadings have been calculated based on the Boussinesq elastic solutions. Recompression and virgin consolidation has been calculated based on the assumed existing overburden pressure and preconsolidation stress profiles shown on Figure A3.

The results of the calculations for embankment centerline and crest settlement using rockfill to construct the embankments are provided in Table 2-8, below and are shown graphically in the top panel of Figure A5. The thickness of the compressible silty clay soil ranges up to about 14 m. Because of the unevenness of the original ground surface at the approaches, the height of fill may vary slightly. Accordingly, the analyses have been performed taking this into account by using a varying fill height.

**Table 2-8. Estimated Embankment Consolidation Settlement - North Approach**

Embankment Ht	Centerline Settlement (mm)	Crest Settlement (mm)
5 m	35	30
6 m	40	35
7 m	70	45
8 m	115	75

Notes: Embankment crest width 14 m. Values rounded to nearest 5 mm

The loadings imposed by the fill will approach and may exceed the indicated preconsolidation pressure, such that the silty clay soils will be normally consolidated following the consolidation process due to the fill. Examination of the top panel of Figure A5 indicates that the settlement is expected to increase significantly as the embankment height exceeds about 6 m, where the settlement curve steepens. This will result in greater settlements due to any future additional loadings, such as grade changes, for example. Accordingly, it may be prudent to overbuild the embankment by 1 or 2 m, allow consolidation to take place and subsequently remove the excess fill height (*i.e.* preloading). The preload should be left in place for about a year, unless monitoring indicates that it can be removed sooner. This will result in a slight overconsolidation of the foundation silty clay soils.

If consideration is to be given to the use of light weight fill, for preliminary design purposes, the total consolidation settlements can be reduced **approximately** by proportion of rockfill replaced by lightweight polystyrene (eg. 30% replacement of fill height with polystyrene will result in about 70% of the indicated settlement).

### 2.5.2.2 Time Rate of Consolidation Settlement

The time rate of consolidation settlement has been calculated for the various cases of embankment height for vertical drainage only. It is also assumed for the purposes of calculation, all embankments are constructed to full height in about a one month construction period. Coefficients of consolidation (virgin),  $C_v$ , of 8 m<sup>2</sup>/year, and (recompression),  $C_{vr}$ , of 40 m<sup>2</sup>/year, have been used in the analyses, based on the results of the consolidation tests and the geotechnical literature.

The bottom panel of Figure A5 shows the calculated consolidation rate for the various fill heights at the north approach. Primary consolidation should be complete anywhere between about 18 months to over 24 months, depending on the fill height.

### 2.5.3 Secondary Compression of Clays

Calculations have been performed to estimate the magnitudes of secondary compression of the foundation clays, following the completion of primary consolidation. For purposes of analysis and discussion, the primary consolidation is assumed to be essentially complete within one to two years from the start of construction of the embankments. The calculations are based on use of a coefficient of secondary compression,  $C_{\alpha\epsilon}$ , of 0.004, based on the results of the consolidation tests, previous experience, and the geotechnical literature.

The calculations indicate that the secondary settlement may be about 30 mm to 50 mm in the first 10 years.

### 2.5.4 Rockfill and Rockfill Settlement

Rockfill used in the embankments and approaches should consist of hard, durable, angular quarried rock. Rockfill should be tested for acid drainage potential and checked against the appropriate regulatory requirements for acceptability.

For the bridge abutments that are to be supported by driven steel H-piles, as discussed in a previous section, these will have to be installed after all the embankment fill has been placed. The driving of steel piles through rockfill will be difficult, if not impossible, unless care is taken to ensure that the rockfill meets a tight size range, generally smaller than 75 mm. For the majority of the embankments and approach fills, however, larger rockfill can be used.

The use of large rockfill, loosely placed or end dumped and compacted by conventional construction traffic only may result in long term settlements of up to about 2% of the fill height. Accordingly, a 10 m high rockfill may settle up to 200 mm following completion of construction. This is primarily due to the rockfill "adjusting" under highway service conditions.

In order to minimize the long term settlements associated with the rockfill itself, consideration can be given to specifying the use of a finer graded rockfill, with a maximum size of 400 mm. This finer graded rockfill should be placed in lifts limited to about 800 mm and compacted with heavy vibratory rollers. While the production of the finer graded rockfill and its placement, as described

above, carries with it a cost premium, the long term performance of the finished highway pavement will be improved.

It is recommended that an NSSP for rockfill material and placement requirements be included in the contract documents.

## **2.6 Instrumentation and Construction Monitoring**

Construction of embankments should be monitored during construction using a properly designed and effective system of instrumentation, consisting primarily of settlement points. Lateral slope movements should also be monitored both during and following construction of the embankments. This will provide indications of the rate of settlement, such that construction timing of the foundations can be modified, if required.

## **2.7 Closing Comments**

The recommendations made in this report are in accordance with our present understanding of the project and are provided solely for the use of Marshall Macklin Monaghan and its design team for the design of the bridge foundations and approach fills for the southbound lanes, five span bridge to be constructed over Trout Creek, as part of the Highway 11 Trout Creek By-Pass Project. We request that we be retained to review the design and our recommendations as the design proceeds, to ensure that the final design is in general agreement with our recommendations and that our recommendations have been interpreted as intended.

A subsurface investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those reported at the test locations, we require that we be notified immediately in order to allow reassessment of our recommendations. It may then be necessary to carry out additional field work and analyses.

Contractors bidding on or undertaking the works should, relative to the subsurface conditions, decide on their own investigations, if deemed necessary, as well as the their own interpretations of the factual results provided herein, so they may draw their own conclusions as to how the subsurface conditions may affect them.

The information presented in this report is based on a limited investigation designed to provide information to support an overall assessment of the current geotechnical conditions at the site of the proposed southbound lanes bridge over Trout Creek. The conclusions presented in this report reflect

site conditions existing at the time of the investigation. It is noted that the soil boundaries indicated on the logs are inferred from discontinuous sampling and observations during drilling. These boundaries are intended to reflect transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change.

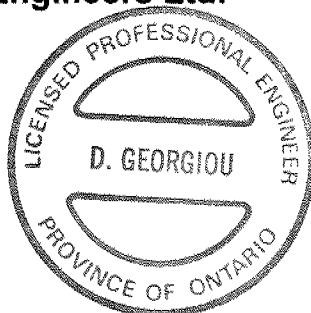
This report has been prepared by Mr. D.N. Georgiou, P.Eng. and was reviewed by Messrs. S.E. Gonsalves, P.Eng., and E. Gonneau, P.Eng. The field investigation was performed by Messrs. I. Dumpis, C.E.T., R. Moore, C.E.T., and S. McAuliffe, and was supervised by Mr. E.A. Gonneau, P.Eng.

We trust this report is satisfactory for your purposes. Should you have any questions, please do not hesitate to contact us.


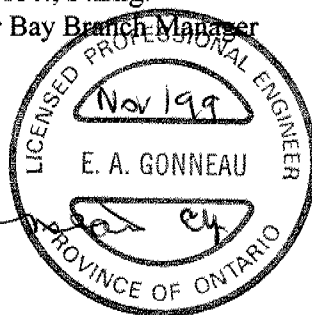
All the foregoing and attachments respectfully submitted,  
**Trow Consulting Engineers Ltd.**



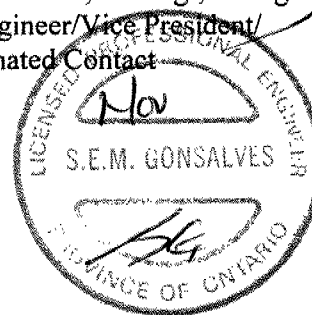
Demetri N. Georgiou, M.A.Sc., P.Eng.  
Principal Engineer/Thunder Bay Branch Manager



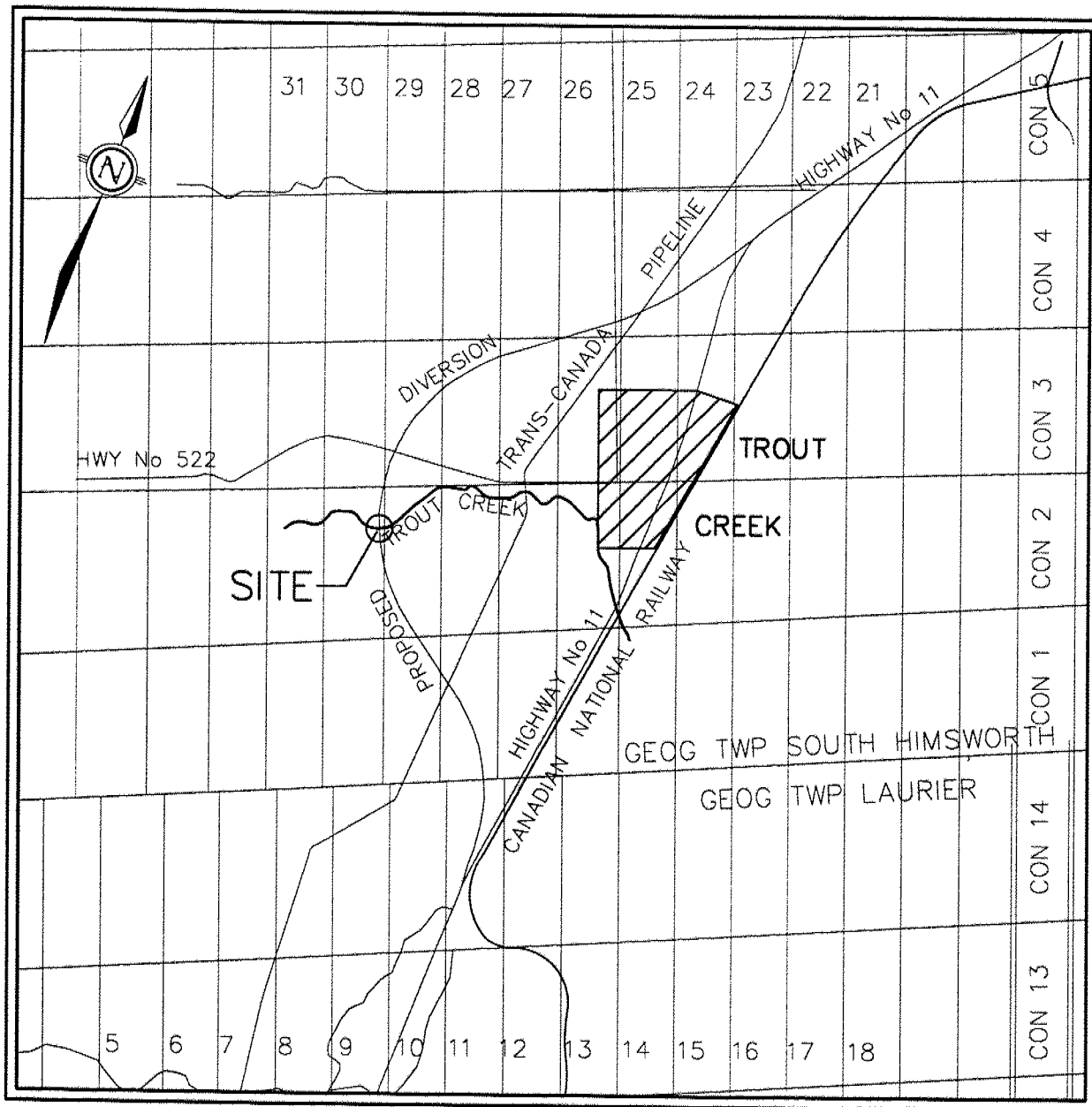
Eric A. Gonneau, P.Eng.  
Project Manager



Stan E. Gonsalves, M.Eng., P.Eng.  
Principal Engineer/Vice President/  
MTO Designated Contact



A



Trow Consulting Engineers Ltd.  
Thunder Bay, Ontario

FIGURE  
A1

### KEY PLAN

Trout Creek By Pass  
Trout Creek Bridge-Southbound Lanes

PROJECT NO.: F-98179-C/G

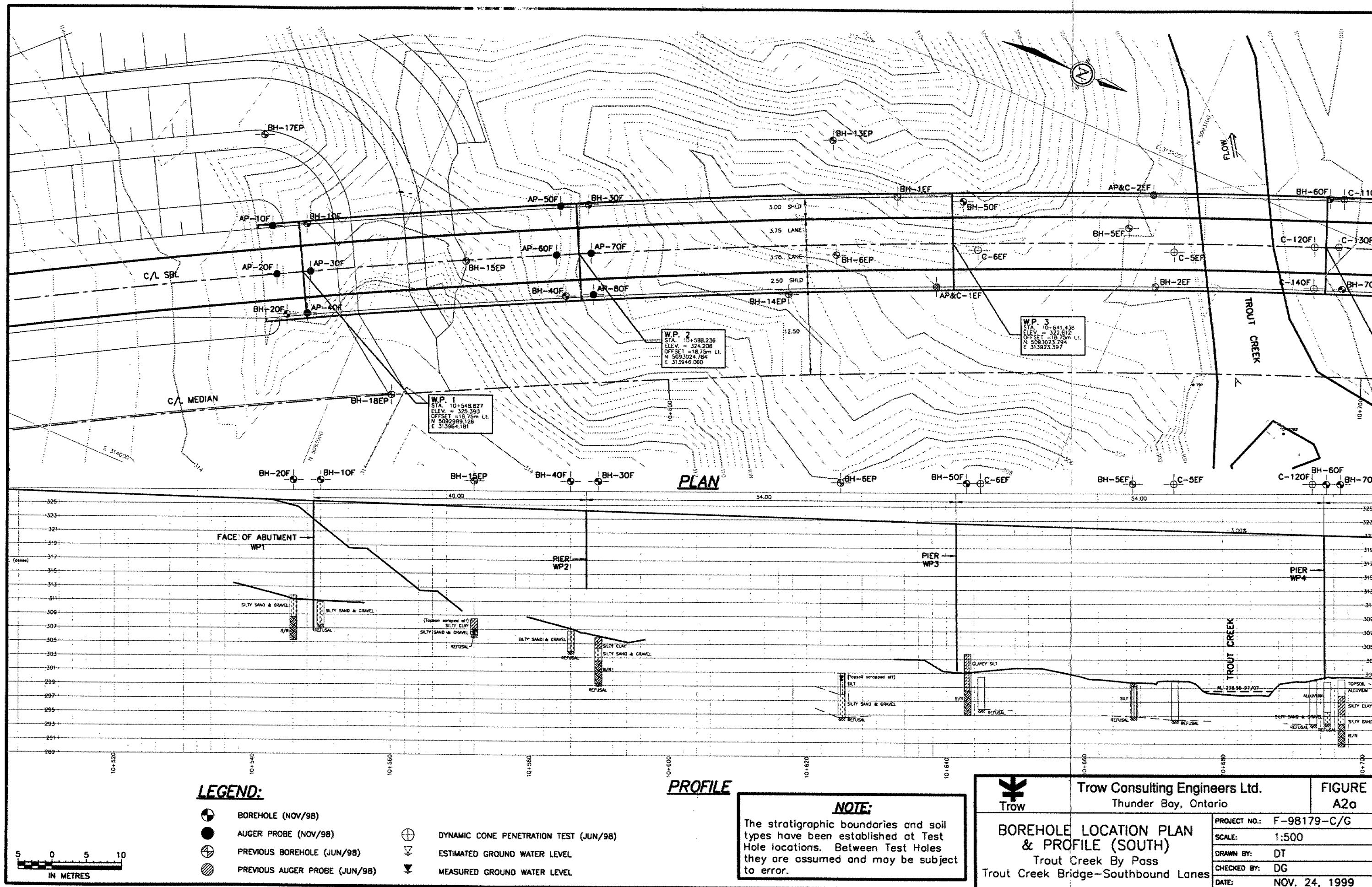
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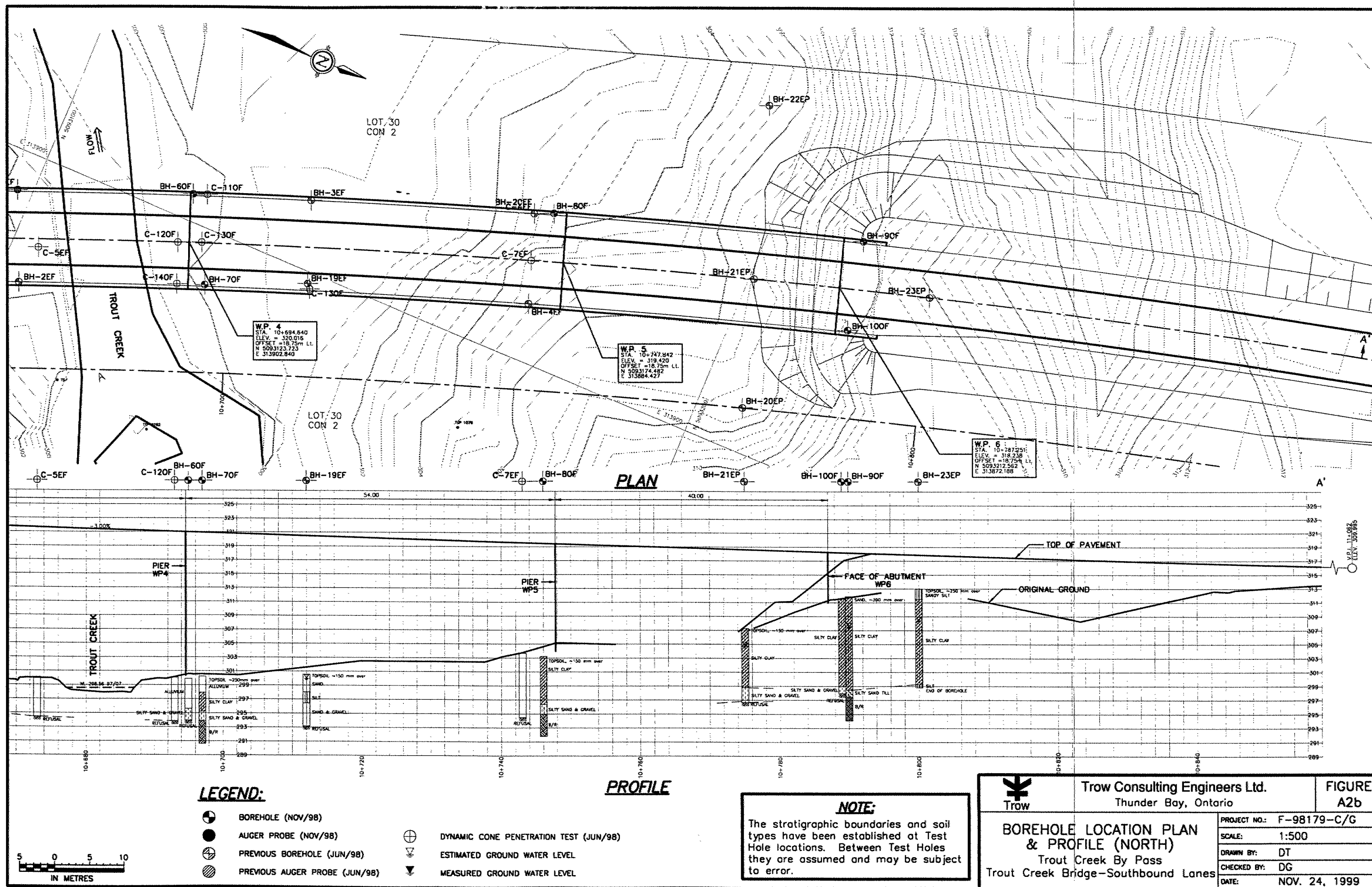
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DATE: MARCH 12, 1999

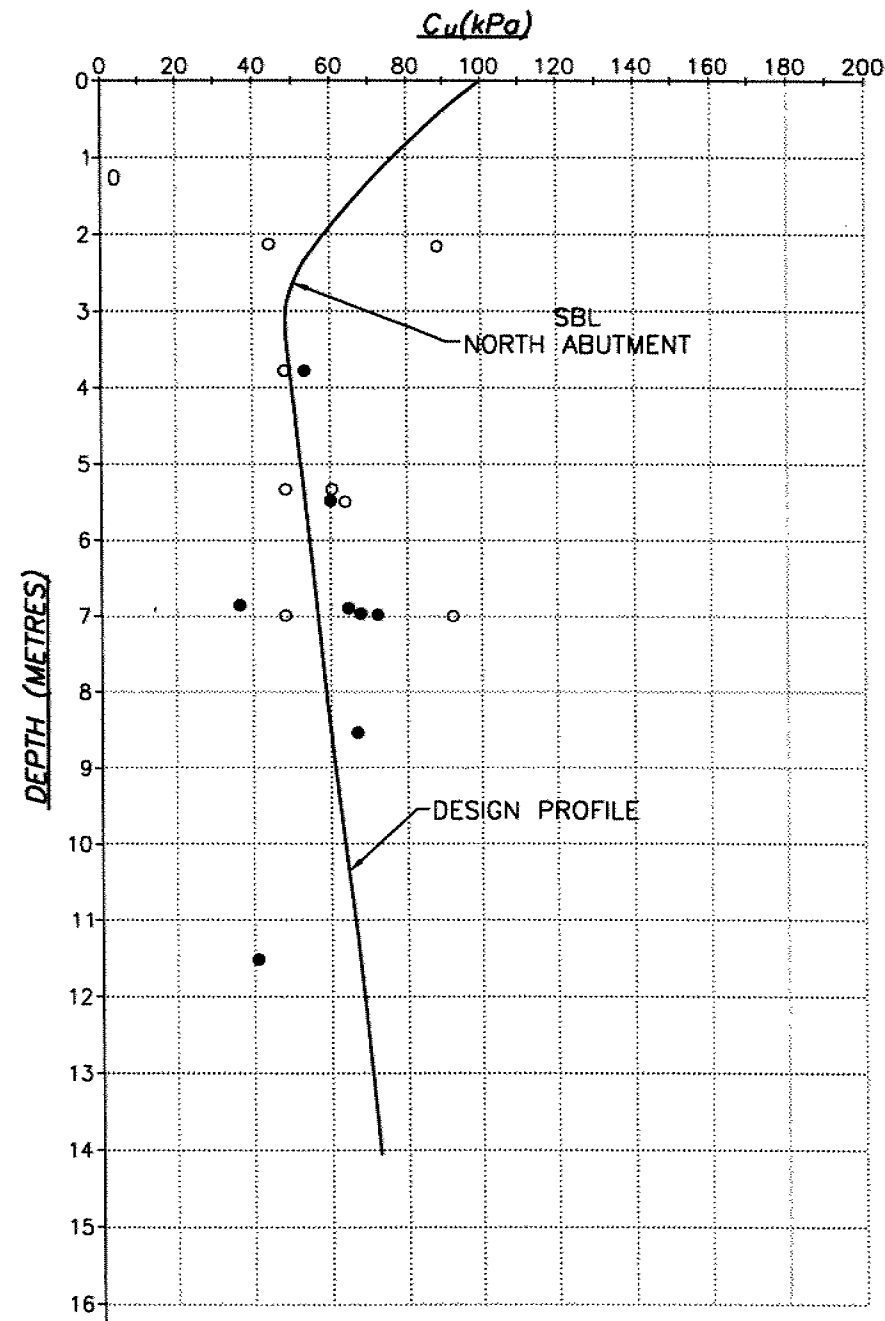
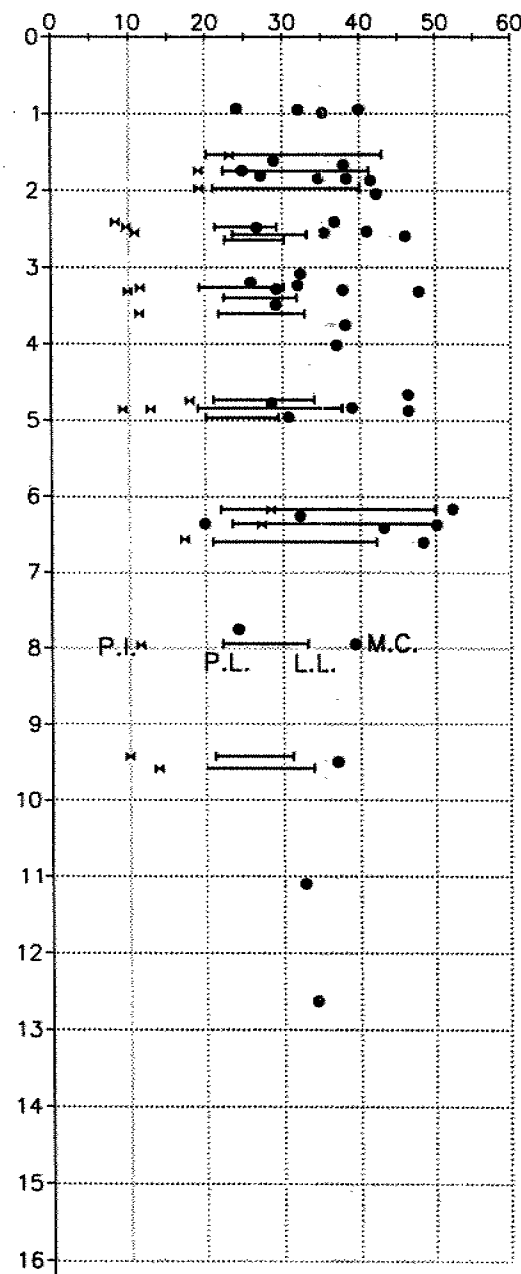
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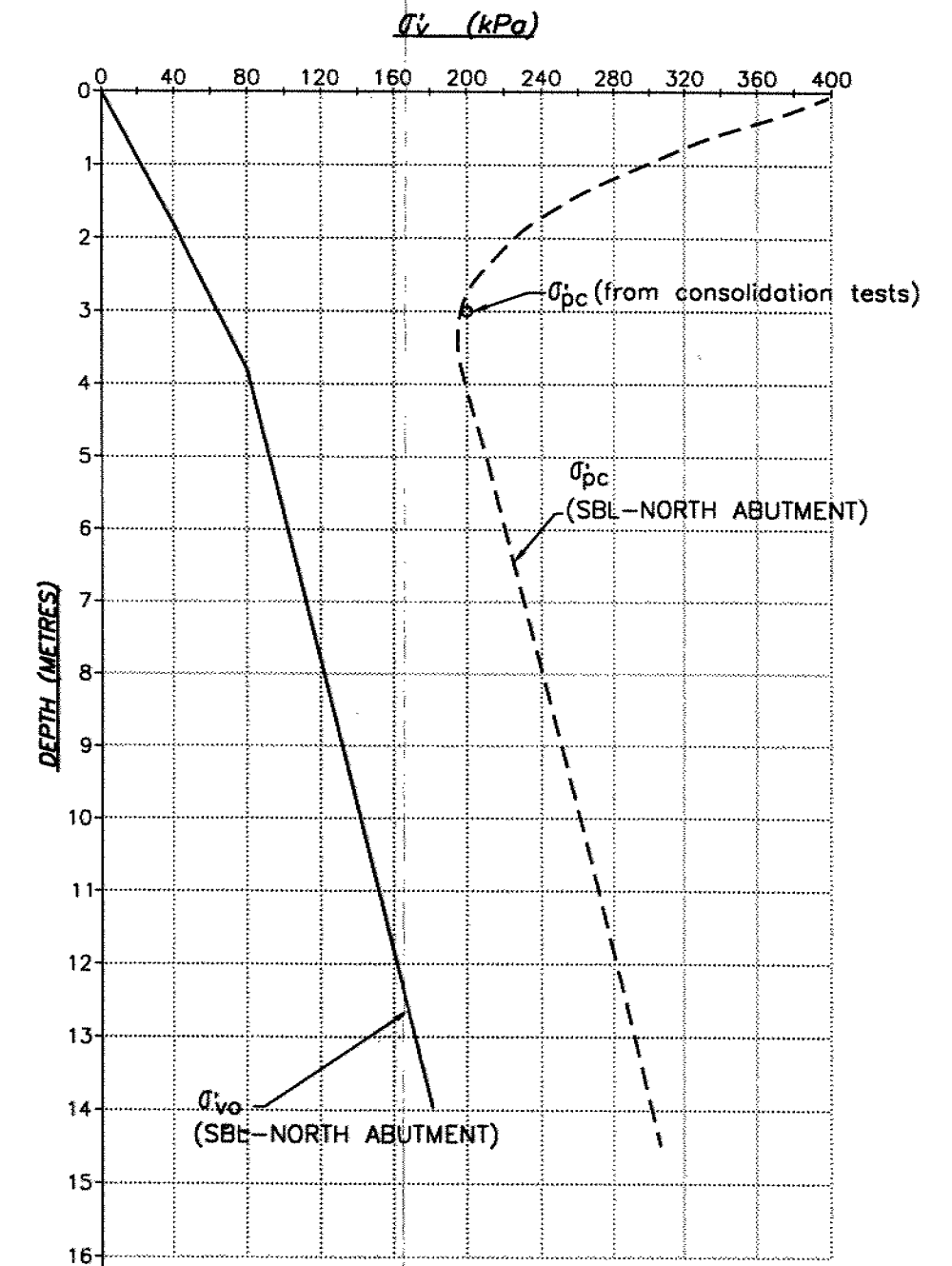




# MOISTURE CONTENT & ATTERBERG LIMITS



• NBL - NORTH ABUTMENT  
○ SBL - NORTH ABUTMENT



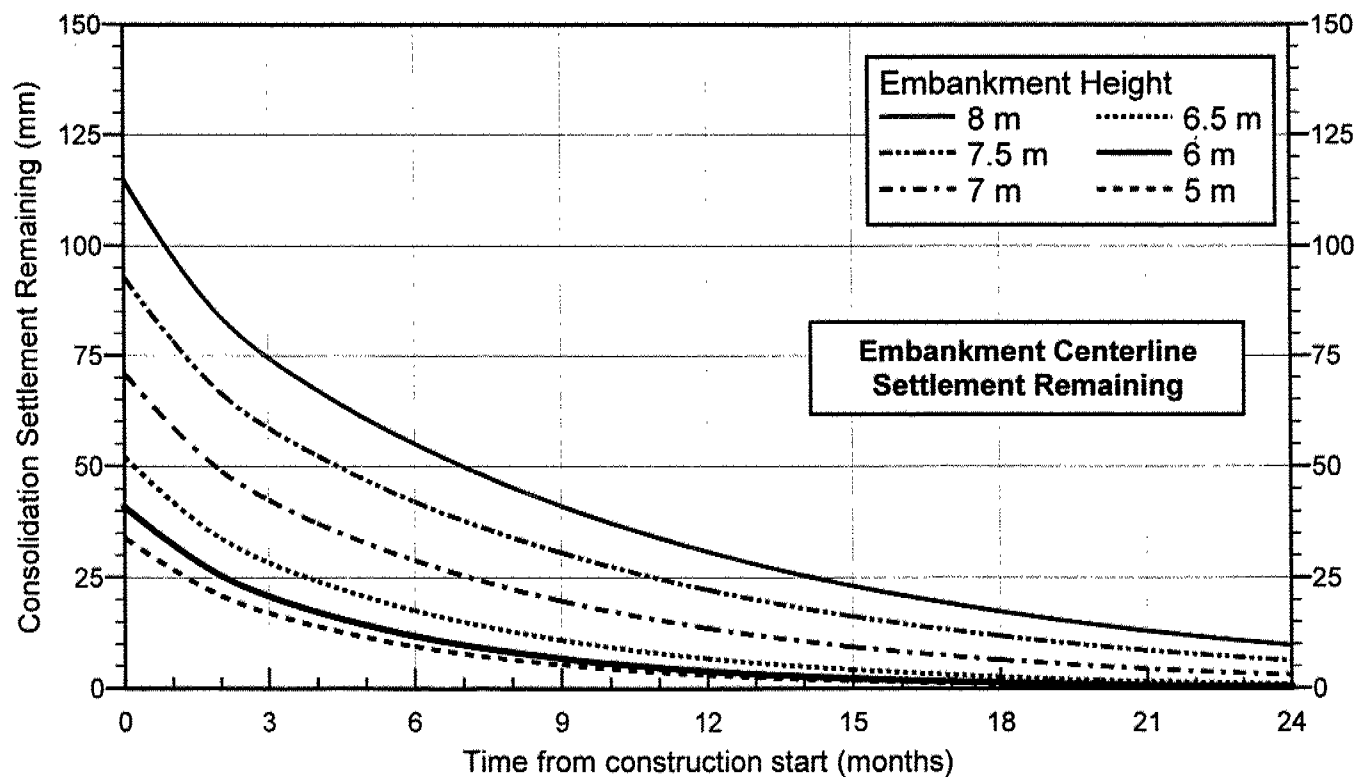
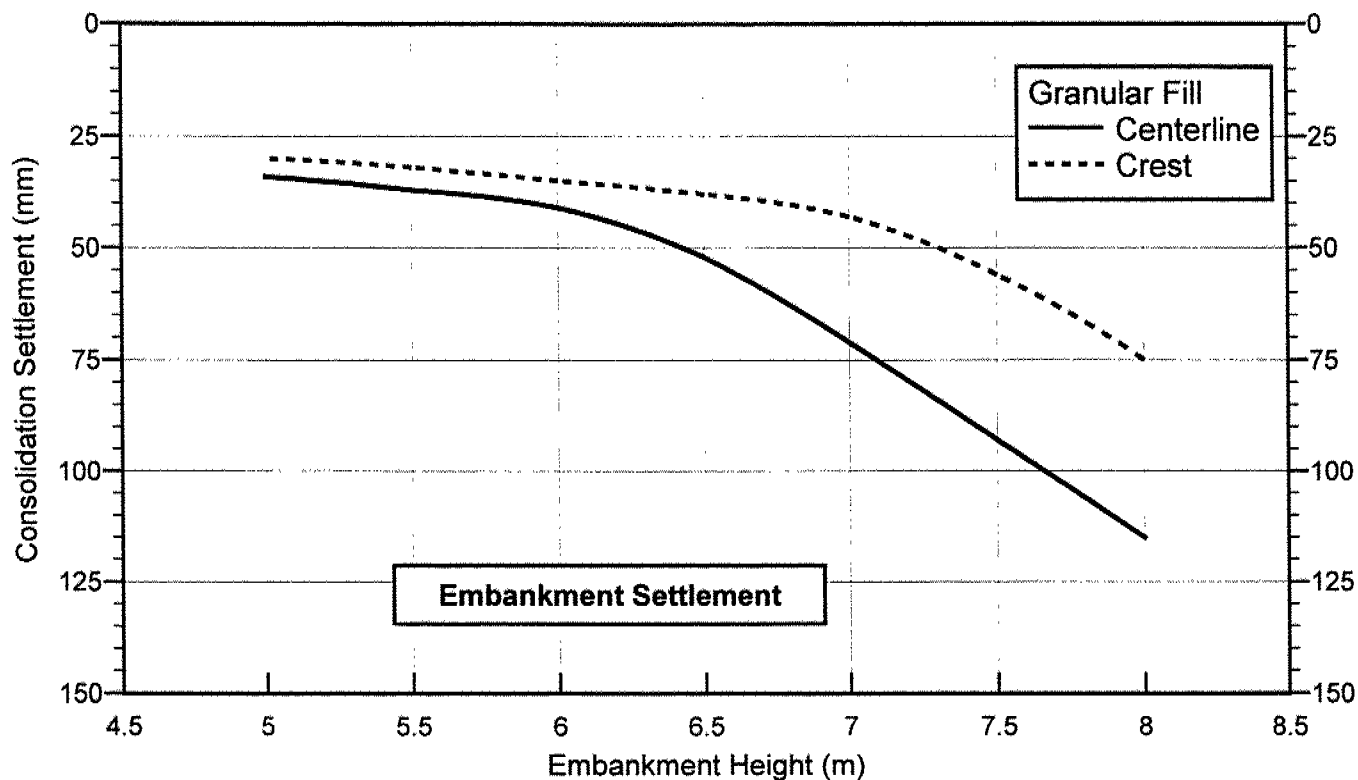
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Thunder Bay, Ontario

FIGURE  
A3

UNDRAINED SHEAR STRENGTH,  
ATTERBERG LIMITS &  
EFFECTIVE STRESS PROFILES  
Trout Creek By Pass  
Trout Creek Bridge-Southbound Lanes

PROJECT NO.: F-98179-C/G  
SCALE: AS SHOWN  
DRAWN BY: DT  
CHECKED BY: DG  
DATE: MARCH 12, 1999





Trow Consulting Engineers Ltd.  
Thunder Bay, Ontario

### Estimated Consolidation Settlement NORTH APPROACH

F98179-C/G

Mar 3/99

Marshall Macklin Monaghan

Trout Creek Bridge - SOUTHBOUND LANES

Figure A5

B

# RECORD OF BOREHOLE BH-10F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station +10+550, offset 7 m left of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 12, 1998 CHECKED BY E.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION			
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	SHEAR STRENGTH: Cu, KPa		WATER CONTENT (%)					
								UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR	wp			w	wl	
310.76	GROUND SURFACE					20	40	60	80	10	20	30	40	kN/m <sup>3</sup>	GR   SA   (SI & CL)
0.00	SILTY SAND & GRAVEL, some cobbles, brown, wet. (compact)		1	SS	6										
			2	SS	24										
			3	SS	21										7%   65%   28%
307.56	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER		4	SS	60			/Bouncing							
3.20	Notes: 1) This borehole forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 092 986.9 N, 313 957.4 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model. 4) Water level was at ~2.6 m & hole was open to ~2.8 m depth on completion. 5) This area was levelled with a bulldozer prior to advancing borehole.														



# RECORD OF BOREHOLE BH-20F 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station ~10+546, offset ~6 m right of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / B.Q. core COMPILED BY M.D.  
 DATUM Geodetic DATE November 13, 1998 CHECKED BY E.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE		20	40	60	80	wp	w	wl	WATER CONTENT (%)		
311.74	GROUND SURFACE														
0.00	SILTY SAND & GRAVEL, some cobbles, brown, wet, till-like structure below ~1.5 m depth. (compact to dense)		1	SS	22										
			2	SS	50										
			3	WS											
308.62			4	SS	60										
3.12	BIOTITE HORNBLende GNEISS		5	BQ											Rec. 100%R.Q.D. 90%
			6	BQ											Rec. 100%R.Q.D. 80%
			7	BQ											Rec. 100%R.Q.D. 80%
305.28															
6.48	END OF BOREHOLE														
<b>Notes:</b> 1) This borehole forms part of the Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 092 989.4 N, 313 970.8 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model. 4) Water level was at ~1.1 m & hole was open to ~2.2 m depth on completion. 5) This area was levelled with a bulldozer prior to advancing borehole. 6) Drill encountered auger refusal on a probable boulder at ~1.6 m depth. Drill moved ~0.5 m from BH-20F and advanced borehole to completion.															



# RECORD OF BOREHOLE BH-30F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+590, offset 7 m left of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / B.Q. core COMPILED BY M.D.  
 DATUM Geodetic DATE November 12, 1998 CHECKED BY E.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR	wp	w	wl	WATER CONTENT (%)		
305.90	GROUND SURFACE														
0.00	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (stiff to firm)		1	SS	13										3% 30% 67%
304.38			2	SS	26										
1.52	SILTY SAND & GRAVEL, some cobbles, till-like below 3 m depth, brown, wet. (compact then dense below 3 m depth)		3	SS	10										
302.55			4	SS	27										
3.35	BIOTITE HORNBLende GNEISS		5	BQ											Rec. 100% R.Q.D. 75%
299.35			6	BQ											Rec. 100% R.Q.D. 97%
6.55	END OF BOREHOLE														
<p>Notes:</p> <p>1) This borehole forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation.</p> <p>2) Borehole located at U.T.M. coordinates 5 093 023.3 N, 313 939.0 E.</p> <p>3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model.</p> <p>4) Water level was at 2.0 m &amp; hole was open to 2.7 m depth on completion.</p> <p>5) This area was levelled with a bulldozer prior to advancing borehole.</p>															





# RECORD OF BOREHOLE BH-40F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+586, offset 6 m right of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 12, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20      40      60      80		wp ——— w ——— wl				
								SHEAR STRENGTH: Cu, KPa		WATER CONTENT (%)				
							● UNCONFINED QUICK TRIAXIAL	● FIELD VANE LAB SHEAR						
							20      40      60      80		10      20      30      40					
306.84	GROUND SURFACE													
0.00	SILTY SAND & GRAVEL, some cobbles, brown, wet. (compact to dense)		1	SS	23		306							
			2	SS	60		305	/Bouncing						
			3	SS	60		304	/Bouncing						
303.94	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER													
2.50	Notes: 1) This borehole forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 025.4 N, 313 952.4 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model. 4) Borehole caved wet at ~2.7 m depth on completion. 5) This area was levelled with a bulldozer prior to advancing borehole.													



# RECORD OF BOREHOLE BH-50F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10 - 643, offset 6 m left of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / B.Q. core COMPILED BY M.D.  
 DATUM Geodetic DATE November 11, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)				
								UNCONF. TRIAXIAL	FIELD VANE	LAB SHEAR		wp	w			wl
303.79	GROUND SURFACE					20	40	60	80	10	20	30	40	GR	SA	(SI & CL)
0.00	CLAYEY SILT, with SILT bands, brown to grey, wet. (loose)		1	SS	8									0%	0%	100%
			2	SS	6											
			3	SS	6											
			4	SS	7											
			5	SS	7									0%	0%	100%
298.52	BIOTITE HORNBLende GNEISS		6	BQ										Rec. 100%	R.Q.D. 60%	
5.27			7	BQ										Rec. 100%	R.Q.D. 50%	
295.01	END OF BOREHOLE		8	BQ										Rec. 100%	R.Q.D. 35%	
8.78	<b>Notes:</b> 1) This borehole forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 072.8 N, 313 917.3 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model. 4) Water level was at ~1.2 m & hole was open to ~4.6 m depth on completion.															



# RECORD OF BOREHOLE BH-60F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+695, offset 7 m left of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 18, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20 40 60 80		wp — w — wl				
							SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL X FIELD VANE LAB SHEAR 20 40 60 80		WATER CONTENT (%) 10 20 30 40				
299.89 0.00	GROUND SURFACE												
	TOPSOIL, 300 mm over ALLUVIUM, SILTY SAND, brown to grey, some organics, wood chunks. (loose)		1	SS	0								
			2	SS	2								
			3	SS	2								
			4	SS	5								
295.62 4.27	SILTY SAND & GRAVEL, some cobbles, grey. (compact)		5	SS	9								
293.95 5.94	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER												
<p>Notes:</p> <p>1) This borehole forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation.</p> <p>2) Borehole located at U.T.M. coordinates 5 093 121.5 N, 313 896.2 E.</p> <p>3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model.</p> <p>4) Water level was at 0.6 m &amp; hole was open to 0.7 m depth on completion.</p>													



# RECORD OF BOREHOLE BH-70F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+697, offset ~6 m right of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / BQ core COMPILED BY M.D.  
 DATUM Geodetic DATE November 18, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20   40   60   80				wp       w       wl				
								SHEAR STRENGTH: Cu, KPa UNCONFINED   QUICK TRIAXIAL   FIELD VANE   LAB SHEAR				WATER CONTENT (%) 10   20   30   40				
300.22	GROUND SURFACE															
0.00	TOPSOIL, ~250 mm over ALLUVIUM, SILTY SAND, brown to grey, some organics, wood chunks. (very loose to loose)		1	SS	4											
			2	SS	1											
297.92																
2.30	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		3	SS	4											
			4	SS	6											
295.22			5	SS	4											
5.00	SILTY SAND & GRAVEL, brown to grey, wet. (compact)															
293.90			6	SS	60											
6.32	BIOTITE HORNBLende GNEISS		7	BQ												
			8	BQ												
290.62																
9.60	END OF BOREHOLE															
Notes: 1) This borehole forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 128.1 N, 313 907.6 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model. 4) Water level was at ~0.5 m & hole was open to ~0.6 m depth on completion.																



# RECORD OF BOREHOLE BH-80F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+746, offset 7 m left of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / BQ core COMPILED BY M.D.  
 DATUM Geodetic DATE November 19, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80		wp — w — wl			
								SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL X FIELD VANE LAB SHEAR		WATER CONTENT (%)			
							20 40 60 80			10 20 30 40		GR SA (SI & CL)	
303.23	GROUND SURFACE												
0.00	TOPSOIL, ~150 mm over SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		1	SS	8								0% 0% 100%
			2	SS	4								
			3	SS	3								
			4	SS	5								
			5	SS	6								
			6	SS	6								
296.37	SILTY SAND & GRAVEL, some cobbles. (loose to compact)		7	SS	8								26% 58% 16%
294.94	BIOTITE HORNBLENDE GNEISS		8	BQ									Rec. 100% RQD 78%
			9	BQ									Rec. 100% RQD 65%
291.80	END OF BOREHOLE												
11.43	Notes: 1) This borehole forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 170.4 N, 313 878.4 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan from their terrain model. 4) Water level was at ~3.2 m & hole was open to ~4.2 m depth on completion. 5) This area was levelled with a bulldozer prior to advancing borehole.												



# RECORD OF BOREHOLE BH-90F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+790, offset ~7 m left of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / BQ core COMPILED BY M.D.  
 DATUM Geodetic DATE November 23, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION			
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60	80			wp	w	wl
								SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL FIELD VANE LAB SHEAR						WATER CONTENT (%)		
						20	40	60	80	10	20	30	40	GR	SA	(SI & CL)
312.07	GROUND SURFACE															
0.00	SAND, brown, fine, ~300 mm over SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (stiff becoming firm)		1	SS	25											
			2	SS	19											
			3	SS	9											
			4	SS	6											
			5	SS	2											
			6	SS	3											
			7	SS	6											
			8	SS	7											
			9	SS	11											
			10	SS	8											
298.66			11	SS	7											
13.41	SILTY SAND & GRAVEL, some cobbles, grey, till-like structure, wet. (compact)		12	BQ												
297.65	BIOTITE HORNBLENDE GNEISS		13	BQ												
14.42	Notes: 1) This borehole forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 213.2 N, 313 864.7 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan from their terrain model.		14	BQ												
294.27	END OF BOREHOLE															
17.80	Notes: (cont'd) 4) Borehole caved wet at ~11.1 m depth on completion. 5) This area was levelled with a bulldozer prior to advancing borehole.															



# RECORD OF BOREHOLE BH-100F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station +10+789, offset 6 m right of centreline of Southbound Lane ORIGINATED BY J.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 23, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20    40    60    80				wp    —    w    —    wl				
								SHEAR STRENGTH: Cu, KPa UNCONFINED      FIELD VANE QUICK TRIAXIAL    X    LAB SHEAR				WATER CONTENT (%) 10    20    30    40				
311.62	GROUND SURFACE															
0.00	SAND, fine, brown, ~600 mm over SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (stiff becoming firm)		1	SS	7		311								0%   0%   100%	
			2	SS	13		310									
			3	SS	11		309									
			4	SS	6		308									
			5	SS	2		307									
			6	SS	2		306								0%   0%   100%	
			7	SS	4		305									
			8	SS	6		304									
			9	SS	8		303									
			10	SS	8		302									
298.97	SILTY SAND & GRAVEL, grey, till-like structure, wet, (compact)						301									
12.65							300									
297.96							299									
13.66	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER						298									
<p>Notes:</p> <p>1) This borehole forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation.</p> <p>2) Borehole located at U.T.M. coordinates 5 093 216.0 N, 313 855.4 E.</p> <p>3) Borehole elevation obtained by Marshall Macklin Monaghan from their terrain model.</p> <p>4) Borehole caved wet at ~11.7 m depth on completion.</p> <p>5) This area was levelled with a bulldozer prior to advancing borehole.</p>																

Notes:  
 1) This borehole forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation.  
 2) Borehole located at U.T.M. coordinates 5 093 216.0 N, 313 855.4 E.  
 3) Borehole elevation obtained by Marshall Macklin Monaghan from their terrain model.  
 4) Borehole caved wet at 11.7 m depth on completion.  
 5) This area was levelled with a bulldozer prior to advancing borehole.



# RECORD OF BOREHOLE BH-1EF 1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station +10+634, offset 7 m left of centreline of Southbound Lane

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Hollow stem augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE June 10, 1998

CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80		wp  ---  w  ---  wl			
								SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL X FIELD VANE LAB SHEAR 20 40 60 80		WATER CONTENT (%) 10 20 30 40			
303.95	GROUND SURFACE												
0.00	(Topsoil scraped off) SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (stiff becoming firm)		1	SS	10		303						
			2	SS	8		302		S=9				
			3	SS	4		301						0% 0% 100%
			4	SS	7		300		S=7	X			
			5	SS	5		299		S=5	X			
			6	SS	7		298		S=10				
296.13			7	SS	40		297						
7.82	BIOTITE HORNBLENDE GNEISS, good to excellent rock quality, slightly weathered to unweathered.		8	BQ			296		/150 mm				Rec. 100% RQD 81%
			9	BQ			295						
293.13							294						Rec. 100% RQD 92%
10.82	END OF BOREHOLE												
Notes: 1) This borehole forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 064.0 N, 313 920.0 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Water level was at ~1.5 m depth on completion.													





# RECORD OF BOREHOLE BH-2EF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+670, offset ~6 m right of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 10, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80				wp — w — wl					
								SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL X FIELD VANE LAB SHEAR 20 40 60 80				WATER CONTENT (%) 10 20 30 40					
299.90	GROUND SURFACE																
0.00	(Topsoil scraped off) SAND, with gravel inclusions & pieces of wood, grey/brown, wet. (very loose)		1	SS	3		299										
298.40	ALLUVIUM																
1.50	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		2	SS	5		298			S=12							
			3	SS	3				X								
			4	SS	3		297			X						0% 10% 90%	
							296										
			5	SS	9		295			S=8							
294.72	BIOTITE HORNBLende GNEISS, good rock quality, unweathered.		6	BQ			294									Rec. 100% RQD 83%	
5.18							293										
			7	BQ			292									Rec. 100% RQD 86%	
291.58	END OF BOREHOLE																
8.32	Notes: 1) This borehole forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 102.8 N, 313 917.6 N. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Water level was at ~1.4 m & hole was open to ~1.6 m depth on completion.																



# RECORD OF BOREHOLE BH-3EF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+712, offset ~7 m left of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 25, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80				wp — w — wl					
								SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)					
								UNCONFINED QUICK TRIAXIAL		FIELD VANE LAB SHEAR							
300.36	GROUND SURFACE																
0.00	TOPSOIL, ~200 mm over SAND, some gravel sizes & root inclusions, traces of clay & silt, brown, wet. (compact)		1	SS	14											0% 66% 34%	
298.36			2	SS	18												
2.00	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		3	SS	5												
			4	SS	6												
295.36			5	SS	8											0% 16% 84%	
5.00	SAND & GRAVEL, some cobbles, brown, wet. (loose then dense at base)		6	SS	5											14% 69% 17%	
			7	SS	35												
292.23	BIOTITE HORNBLende GNEISS, excellent rock quality, unweathered.		8	BQ												Rec. 100% RQD 95%	
8.13			9	BQ												Rec. 100% RQD 95%	
289.29	END OF BOREHOLE																
11.07	Notes: 1) This borehole forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 137.8 N, 313 890.0 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Drill met auger refusal at ~5.5 m depth. Drill moved ~0.6 m south of BH-3EF & carried out borehole to completion. 5) Water level was at ~0.9 m & hole was open to ~6.4 m depth on completion.																



# RECORD OF BOREHOLE BH-4EF 1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station +10+743, offset ~6 m right of centreline of Southbound Lane

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Hollow stem augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE June 30, 1998

CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20	40	60	80					
305.36	GROUND SURFACE														
0.00	(Topsoil scraped off) CLAYEY SILT, some layering, brown to grey, moist. (loose)		1	SS	7										
			2	SS	5										
			3	SS	4										
			4	SS	7										
301.36	4.00 SILT, occasional thin clay seams, grey, wet. (loose)		5	SS	7										
			6	SS	7										
298.36	7.00 SAND & GRAVEL, some cobbles & possible boulders, grey, wet. (compact)		7	SS	9										
			8	SS	12										
295.45	9.91 BIOTITE HORNBLENDE GNEISS, fair to excellent rock quality, slightly weathered to unweathered.		9	BQ											
			10	BQ											
292.38	12.98 END OF BOREHOLE														
Notes: 1) This borehole forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 171.8 N, 313 891.7 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Borehole caved wet at ~8.1 m depth on completion.															



# RECORD OF BOREHOLE BH-5EF 1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station 10+667, offset 2 m left of centreline of Southbound Lane

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Hollow stem augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE June 11, 1998

CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE		20	40	60	80	wp	w	wl			
299.76	GROUND SURFACE														
0.00	TOPSOIL, 150 mm over SILT, some sand, grey, traces of clay, wet below 600 mm depth. (loose)		1	SS	3										
			2	SS	5										
			3	SS	5										
			4	SS	5										
294.88	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER		5	SS	40										
4.88	Notes: 1) This borehole forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 096.9 N, 313 911.4 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Water level was at 3.8 m & hole was open to 4.3 m depth on completion.														



# RECORD OF BOREHOLE BH-6EP 1 OF 1

**METRIC**

W.P. 774-93-00 LOCATION Station +10+625, offset ~2 m right of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 15, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60	80		
300.97	GROUND SURFACE												
0.00	(Topsoil scraped off) SILT, occasional thin clay seams, grey, wet. (very loose to loose)		1	SS	2								
			2	SS	8								
			3	SS	9								
297.97			4	SS	36								
3.00	SILTY SAND & GRAVEL, some cobble sizes, brown, wet. (dense to very dense)		5	SS	105								
			6	SS	60								
294.63													
6.34	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER												
Notes: 1) This borehole forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 053.4 N, 313 932.0 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Water level was at ~0.9 m & hole was open to ~2.4 m depth on completion.													



# RECORD OF BOREHOLE BH-12EP 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station -10+499, on centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 18, 1998 CHECKED BY I.G.

SOIL PROFILE				SAMPLES			SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT				UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION  GR   SA   (SI & CL)	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m	GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	20      40      60      80				wp ——— w ——— wl					
								SHEAR STRENGTH: Cu, KPa ● UNCONFINED      ■ FIELD VANE QUICK TRIAXIAL      X LAB SHEAR				WATER CONTENT (%) 10      20      30      40					
316.75	GROUND SURFACE																
0.00	TOPSOIL, ~150 mm over																
316.08	SAND & GRAVEL																
	(dense)																
0.67	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER																
Notes: 1) This borehole forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 032 944.9 N, 313 988.7 E 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Borehole was dry & open to full depth on completion. 5) Drill made 4 other attempts to drill adjacent BH-12EF & met auger refusals from ~0.4 m to ~0.7 m depths.																	



# RECORD OF BOREHOLE BH-13EP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+625, offset ~15 m left of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 19, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80				wp — w — wl				
								SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)				
							UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR						kn/m³	GR SA (SI & CL)	
307.33	GROUND SURFACE															
0.00	(Topsoil scraped off) SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		1	SS	4											
			2	SS	5											
			3	SS	6											
			4	SS	6											
			5	TW												
			6	SS	7											
300.33																
7.00	SILT, traces of fine sand, some gravel sizes at base, wet. (compact)		7	SS	9											
299.25	SAND & GRAVEL (dense)															
8.08																
298.34																
8.99	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER															
	Notes: 1) This borehole forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 052.4 N, 313 916.5 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Borehole caved dry at ~4.8 m depth on completion.															



# RECORD OF BOREHOLE BH-14EP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+618, offset 7 m right of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 19, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)					
								UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR	wp	w	wl					
303.60	GROUND SURFACE																
0.00	(Topsoil scraped off) SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		1	SS	4												
			2	SS	5												
301.10			3	SS	20												
2.50	SILTY SAND & GRAVEL, some cobbles, grey, wet. (compact)		4	SS	24												
300.25																	
3.35	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER																
Notes: 1) This borehole forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 055.0 N, 313 939.5 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Borehole caved dry at ~2.7 m depth on completion.																	





# RECORD OF BOREHOLE BH-15EP 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station +10+572, on centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 19, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m								
309.49	GROUND SURFACE														
0.00	(Topsoil scraped off) <b>SILTY CLAY</b> , grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		1	SS	8										
307.69			2	SS	26										
1.80	<b>SILTY SAND &amp; GRAVEL</b> (compact to dense)														
307.23															
2.26	<b>END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER</b>														
<b>Notes:</b> 1) This borehole forms part of Trout Creek Bridge Southbound Lane Approach Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 010.0 N, 313 953.4 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Borehole caved dry at ~2.2 m depth on completion.															



# RECORD OF BOREHOLE BH-17EP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+545, offset 20 m left of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 19, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)				
								UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR	wp	w	wl				
313.84	GROUND SURFACE															
0.00	TOPSOIL, ~180 mm over CLAYEY SILT, bands of SILT, brown to grey, wet silt seams below 2.0 m depth. (firm to stiff)															
			1	SS	6											
			2	SS	6											
309.34																
4.50	SILTY SAND & GRAVEL, cobbles. (dense)		3	SS	37										20% 61% 19%	
308.51																
5.33	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER															
Notes: 1) This borehole forms part of Trout Creek Bridge Southbound Lane Approach Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 092 976.3 N, 313 948.3 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Borehole caved wet at ~3.8 m depth on completion.																



# RECORD OF BOREHOLE BH-18EP 1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station ~10+560, on centreline of Median

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Hollow stem augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE June 19, 1998

CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE		20	40	60	80	wp	w	wl			
313.75	GROUND SURFACE														
0.00	TOPSOIL, ~125 mm over CLAYEY SILT, seams of SILT, brown to grey, wet below ~2.0 m depth. (stiff)														
			1	SS	6										
			2	SS	4										
	Cobbles at base.														
309.48	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER														
4.27	Notes: 1) This borehole forms part of Trout Creek Bridge Southbound Lane Approach Foundation investigation. 2) Borehole located at U.T.M. coordinates 5 093 007.8 N, 313 975.6 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Borehole caved dry at ~3.3 m depth on completion.														



# RECORD OF BOREHOLE BH-19EF 1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station ~10+712, offset ~5 m right of centreline of Southbound Lane

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Hollow stem augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE June 24, 1998

CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m	20	40	60	80	wp	w	wl		
300.52	GROUND SURFACE													
0.00	TOPSOIL, ~150 mm over SAND, trace of silt, brown, wet below ~600 mm depth. (compact)		1	SS	11									
298.02														
2.50	SILT, some thin clay seams, grey, wet. (loose)		2	SS	9									
296.42														
4.10	SAND & GRAVEL, some cobbles & possible boulders. (compact to dense)		3	SS	29									
			4	SS	31									
293.17														
7.35	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER													
Notes: 1) This borehole forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 141.9 N, 313 901.3 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Water level was at ~2.0 m & hole was open to ~2.1 m depth on completion.														



# RECORD OF BOREHOLE BH-20EF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+742, offset 7 m left of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 26, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20	40	60	80	wp	w	wl		
303.20	GROUND SURFACE														
0.00	TOPSOIL, ~150 mm over SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		1	SS	9										
			2	SS	8										
			3	SS	6										
			4	SS	5										
			5	TW											
			6	SS	5										
296.70	SAND & GRAVEL, some cobbles & possible boulders, grey, wet. (loose)		7	SS	5										
6.50			8	SS	7										
293.90	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER														
9.30	Notes: 1) This borehole forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 166.6 N, 313 879.7 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Water level was at ~4.9 m & hole was open to ~5.94 m depth on completion.														



# RECORD OF BOREHOLE BH-21EP 1 OF 1

## METRIC

W.P. 774-93-00

LOCATION Station +10+775, on centreline of Southbound Lane

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Hollow stem augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE July 1, 1998

CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)		CONE PENETRATION TEST		PLASTIC LIMIT			NATURAL MOISTURE CONTENT			LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION		
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			SHEAR STRENGTH: Cu, KPa		WATER CONTENT (%)		wp	w	wl										
								UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR															
307.28	GROUND SURFACE																							
0.00	TOPSOIL, ~150 mm over SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		1	SS	3																			
			2	SS	5																			
			3	AS																				
			4	TW																				
			5	SS	5																			
			6	SS	5																			
			7	SS	10																			
298.78																								
8.50	SILTY SAND & GRAVEL, grey, wet. (compact)		8	SS	17																			
296.67																								
10.61	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER																							
<p>Notes:</p> <p>1) This borehole forms part of Trout Creek Bridge Southbound Lane Approach Foundation Investigation.</p> <p>2) Borehole located at U.T.M. coordinates 5 093 200.7 N, 313 875.9 E.</p> <p>3) Borehole elevation obtained from Marshall Macklin Monaghan survey crew.</p> <p>4) Borehole caved wet at ~8.8 m depth on completion.</p>																								



# RECORD OF BOREHOLE BH-22EP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station +10+775, offset 25 m left of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE July 1, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE		20	40	60	80	wp	w	wl	WATER CONTENT (%)		
305.85	GROUND SURFACE														
0.00	TOPSOIL, 150 mm over SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		1	SS	7										
			2	SS	5										
			3	SS	3										
			4	SS	3										
			5	TW											
299.85			6	SS	28										
6.00	SILTY SAND & GRAVEL, till-like structure, some cobbles, grey, wet. (compact to dense)		7	SS	15										
287.65															
8.20	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER														
Notes: 1) This borehole forms part of Trout Creek Bridge Southbound Lane Approach Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 193.2 N, 313 852.0 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Borehole caved wet at 5.3 m depth on completion.															



METRIC

W.P. 774-93-00

**LOCATION** Station ~10+800, on centreline of Southbound Lane

ORIGINATED BY S.M.

DIST 54 HWY 11

**BOREHOLE TYPE** Hollow stem augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE October 24, 1998

CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION			
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60	80			wp	w	wl
								SHEAR STRENGTH: Cu, KPa		WATER CONTENT (%)						
							● UNCONFINED QUICK TRIAXIAL	×	■ FIELD VANE LAB SHEAR							
							20	40	60	80	10	20	30	40		
313.04	GROUND SURFACE															
0.00	TOPSOIL, 250 mm over SANDY SILT, brown, moist. (compact)															
311.54	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (stiff becoming firm)		1	SS	18											
1.50			2	SS	9											
			3	SS	2											
			4	TW												
			5	SS	3											
			6	SS	9											
			7	SS	9											
			8	SS	12											
299.32	SILT, with SAND seams, grey brown, moist (compact)		9	SS	16											
13.72																
298.87	END OF BOREHOLE															
14.17	Notes: 1) This borehole forms part of Trout Creek Bridge Southbound Lane Approach Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 225.0 N, 313 868.5 E. 3) Borehole elevation obtained from Dearden and Stanton Ltd., O.L.S. 4) Borehole caved wet at ~10.3 m depth on completion.															





# RECORD OF BOREHOLE BH-24EP 1 OF 1

## METRIC

W.P. 774-93-00      LOCATION Station ~10+875, on centreline of Southbound Lane      ORIGINATED BY S.M.  
 DIST 54      HWY 11      BOREHOLE TYPE Hollow stem augers / CME-55      COMPILED BY M.D.  
 DATUM Geodetic      DATE October 24, 1998      CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60					
313.54	GROUND SURFACE														
0.00	TOPSOIL, ~250 mm over SILTY SAND, brown, moist. (compact)		1	SS	14										1% 95% 4%
311.10	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		2	SS	5										
2.44			3	SS	2										0% 0% 100%
			4	SS	1										
			5	TW											0% 0% 100%
			6	SS	6										
			7	SS	10										
301.35	SILT, trace of SAND, grey brown, moist. (compact)		8	SS	15										
12.19															
300.89															
12.65															
<b>END OF BOREHOLE</b> <b>Notes:</b> 1) This borehole forms part of Trout Creek Bridge Southbound Lane Approach Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 298.6 N, 313 849.3 E. 3) Borehole elevation obtained from Dearden and Stanton Ltd., O.L.S. terrain model. 4) Borehole caved wet at ~10.3 m depth on completion.															



# RECORD OF BOREHOLE AP-10F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+545, offset 7 m left of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 12, 1998 CHECKED BY E.G.


SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE		20	40	60	80	wp	w	wl			
312.05	GROUND SURFACE														
0.00															
	Probable SAND, GRAVEL & COBBLES														
309.15															
2.90	END OF PROBE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER														
	Notes: 1) This auger probe forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Auger probe located at U.T.M. coordinates 5 092 982.4 N, 313 959.8 E. 3) Probe elevation obtained from Marshall Macklin Monaghan terrain model.														



# RECORD OF BOREHOLE AP-20F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+545, on centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 12, 1998 CHECKED BY E.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) 				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	CONE PENETRATION TEST				WATER CONTENT (%)		
						20	40	60	80	wp	w	wl		
						SHEAR STRENGTH: Cu, KPa								
						UNCONFINED QUICK TRIAXIAL X FIELD VANE LAB SHEAR								
						20	40	60	80	10	20	30	40	
312.05	GROUND SURFACE													
0.00	Probable SAND, GRAVEL & COBBLES													
310.37														
1.68	END OF PROBE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER													
	Notes: 1) This auger probe forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Auger probe located at U.T.M. coordinates 5 092 985.7 N, 313 966.0 E. 3) Probe elevation obtained from Marshall Macklin Monaghan terrain model.													



# RECORD OF BOREHOLE AP-30F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+550, on centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 12, 1998 CHECKED BY E.G.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT				UNIT WEIGHT  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20   40   60   80				wp — w — wl					
								SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)					
							●	○	×	■							
							UNCONFINED	QUICK TRIAXIAL	FIELD VANE	LAB SHEAR							
							20	40	60	80							



# RECORD OF BOREHOLE AP-40F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+549, offset 6 m right of centreline of Southbound Lane ORIGINATED BY J.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 12, 1998 CHECKED BY E.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m	20	40	60	80	wp	w	wl	WATER CONTENT (%)		
311.83 0.00	GROUND SURFACE														GR SA (SI & CL)
310.00 1.83	Probable SANDY GRAVEL														
309.09 2.74	Probable SAND, GRAVEL & COBBLES														
END OF PROBE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER <b>Notes:</b> 1) This auger probe forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Auger probe located at U.T.M. coordinates 5 092 992.1 N, 313 969.4 E. 3) Probe elevation obtained from Marshall Macklin Monaghan terrain model.															



# RECORD OF BOREHOLE AP-50F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station +10+586, offset 7 m left of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 12, 1998 CHECKED BY E.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)				CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE															
306.65	GROUND SURFACE																		
0.00																			
	Probable SILTY SAND & GRAVEL																		
305.13																			
1.52																			
	Probable SAND, GRAVEL & COBBLES																		
302.54																			
4.11	END OF PROBE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER																		
	Notes: 1) This auger probe forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Auger probe located at U.T.M. coordinates 5 093 019.6 N, 313 940.8 E. 3) Probe elevation obtained from Marshall Macklin Monaghan terrain model.																		



# RECORD OF BOREHOLE AP-60F 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station 10+585, on centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 12, 1998 CHECKED BY E.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT				UNIT WEIGHT kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION GR   SA   (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	20	40	60	80	wp	w		
307.39 0.00	GROUND SURFACE														
	Probable SILTY SAND & GRAVEL														
305.56 1.83	Probable SAND, GRAVEL & COBBLES														
304.71 2.68	<b>END OF PROBE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER</b> <b>Notes:</b> 1) This auger probe forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 021.8 N, 313 947.5 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model.														



# RECORD OF BOREHOLE AP-70F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+590, on centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 12, 1998 CHECKED BY E.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m	20	40	60	80					
306.10 0.00	GROUND SURFACE													
	Probable SILTY CLAY													
303.66 2.44														
	Probable SAND, GRAVEL & COBBLES													
302.38 3.72	END OF PROBE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER													
	Notes: 1) This auger probe forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Auger probe located at U.T.M. coordinates 5 093 026.4 N, 313 945.3 E. 3) Probe elevation obtained from Marshall Macklin Monaghan terrain model.													





# RECORD OF BOREHOLE AP-80F 1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station 10+590, offset 7.6 m right of centreline of Southbound Lane

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Standard augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE November 12, 1998

CHECKED BY E.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	20	40	60	80	wp	w		
305.77 0.00	GROUND SURFACE														
305.16 0.61	Probable SILT														
	Probable SAND, GRAVEL & COBBLES														
302.72 3.05	END OF PROBE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER														
	Notes: 1) This auger probe forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Auger probe located at U.T.M. coordinates 5 093 029.0 N, 313 950.7 E 3) Probe elevation obtained from Marshall Macklin Monaghan terrain model.														



# RECORD OF BOREHOLE AP&C-1EF<sub>1</sub> OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+639, offset 6 m right of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Dynamic cone / Standard auger COMPILED BY M.D.  
 DATUM Geodetic DATE June 10, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST 20 40 60 80 SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL X FIELD VANE LAB SHEAR 20 40 60 80	PLASTIC LIMIT w <sub>p</sub> ——— w ——— w <sub>l</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT	UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER								
301.02 0.00	GROUND SURFACE (Topsoil scraped off)										
	Probable SILTY CLAY										
295.84 5.18	END OF CONE TEST DUE TO 'BOUNCING' REFUSAL ON PROBABLE BEDROCK OR POSSIBLE BOULDER  Notes: 1) This cone test forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Auger probe & cone test located at U.T.M. coordinates 5 093 073.9 N, 313 929.9 E. 3) Probe elevation obtained by Marshall Macklin Monaghan survey crew.										



# RECORD OF BOREHOLE AP&C-2EF<sup>1</sup> OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~ 10+670, offset ~7 m left of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Dynamic cone / Standard auger COMPILED BY M.D.  
 DATUM Geodetic DATE June 10, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR			wp
299.84	GROUND SURFACE (Topsoil scraped off)											
0.00												
	Assumed SILTY CLAY											
294.87	END OF CONE TEST DUE TO 'BOUNCING' REFUSAL ON PROBABLE BEDROCK OR POSSIBLE BOULDER											
4.97	Notes: 1) This cone test forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Auger probe located at U.T.M. coordinates 5 093 097.8 N, 313 905.6 E. 3) Probe elevation obtained by Marshall Macklin Monaghan survey crew.											



# RECORD OF BOREHOLE C-110F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10-697, offset 7 m left of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Dynamic cone / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 25, 1998 CHECKED BY E.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m					
300.01 0.00	GROUND SURFACE Dynamic cone test only.											
293.33 6.68	END OF CONE TEST DUE TO 'BOUNCING' REFUSAL ON PROBABLE BEDROCK OR POSSIBLE BOULDER  Notes: 1) This cone test forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Cone test located at U.T.M. coordinates 5 093 123.4 N, 313 895.4 E. 3) Cone elevation obtained from Marshall Macklin Monaghan terrain model.											



# RECORD OF BOREHOLE C-120F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+693, offset on centreline of Southbound Lane ORIGINATED BY J.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Dynamic cone / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 25, 1998 CHECKED BY E.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)				PLASTIC LIMIT			NATURAL MOISTURE CONTENT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA	NUMBER			TYPE	BLOWS/0.3m	CONE PENETRATION TEST				wp	w	wl	WATER CONTENT (%)		
						SHEAR STRENGTH: Cu, KPa											
						UNCONFINED	QUICK TRIAXIAL	FIELD VANE	LAB SHEAR								
						20	40	60	80	10	20	30	40				
300.16 0.00	GROUND SURFACE Dynamic cone test only.																
293.94 6.22	END OF CONE TEST DUE TO 'BOUNCING' REFUSAL ON PROBABLE BEDROCK OR POSSIBLE BOULDER  Notes: 1) This cone test forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 122.2 N, 313 903.4 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model.																



# RECORD OF BOREHOLE C-130F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+697, on centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Dynamic cone / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 25, 1998 CHECKED BY E.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT NUMBER	TYPE			BLOWS/0.3m	20	40	60	80	wp	w			wl
300.57 0.00	GROUND SURFACE Dynamic cone test only.														
293.86 6.61	END OF CONE TEST DUE TO 'BOUNCING' REFUSAL ON PROBABLE BEDROCK OR POSSIBLE BOULDER  Notes: 1) This cone test forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Cone test located at U.T.M. coordinates 5 093 126.0 3) Cone elevation obtained from Marshall Macklin Monaghan terrain model.														



# RECORD OF BOREHOLE C-140F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+693, offset 6 m right of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Dynamic cone / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 25, 1998 CHECKED BY E.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA	NUMBER			TYPE	BLOWS/0.3m	20	40	60	80	wp	w		
300.12	GROUND SURFACE														
0.00	Dynamic cone test only.														
294.06	END OF CONE TEST DUE TO "BOUNCING" REFUSAL ON PROBABLE BEDROCK OR POSSIBLE BOULDER														
6.06	Notes: 1) This cone test forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Cone test located at U.T.M. coordinates 5 093 124.3 N, 313 909.2 E. 3) Cone elevation obtained from Marshall Macklin Monaghan terrain model.														



# RECORD OF BOREHOLE C-3EF

1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+712, offset ~6 m right of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Dynamic cone / COMPILED BY M.D.  
 DATUM Geodetic DATE June 24, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	20	40	60	80	wp			w
300.59	GROUND SURFACE														
0.00	Dynamic cone penetration test only.														
293.40	END OF CONE TEST DUE TO 'BOUNCING' REFUSAL ON PROBABLE BEDROCK OR POSSIBLE BOULDER														
7.19	Notes: 1) This cone test forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Cone test located at U.T.M. coordinates 5 093 142.3 N, 313 902.2 E. 3) Cone elevation obtained by Marshall Macklin Monaghan survey crew.														





# RECORD OF BOREHOLE C-4EF

1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station +10+744, offset 7 m left of centreline of Southbound Lane

ORIGINATED BY J.D.

DIST 54 HWY 11

BOREHOLE TYPE Dynamic cone /

COMPILED BY M.D.

DATUM Geodetic

DATE June 26, 1998

CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m					
303.19	GROUND SURFACE											
0.00	Dynamic cone penetration test only.											
294.00	END OF CONE TEST DUE TO 'BOUNCING' REFUSAL ON PROBABLE BEDROCK OR POSSIBLE BOULDER											
9.19	Notes: 1) This cone test forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Cone test located at U.T.M. coordinates 5 093 168.5 N, 313 879.1 E. 3) Cone elevation obtained by Marshall Macklin Monaghan survey crew.											
	5093170.0N											
	313885.9E											



# RECORD OF BOREHOLE C-5EF

1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+673, offset 1 m right of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Dynamic cone / COMPILED BY M.D.  
 DATUM Geodetic DATE June 10, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	20	40	60	80	wp	w	wp	wl			10
300.08	GROUND SURFACE																	
0.00	Dynamic cone penetration test only.																	
294.44	END OF CONE TEST DUE TO 'BOUNCING' REFUSAL ON PROBABLE BEDROCK OR POSSIBLE BOULDER																	
5.64	Notes: 1) This cone test forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Cone test located at U.T.M. coordinates 5 093 103.7 N, 313 911.9 E. 3) Cone elevation obtained by Marshall Macklin Monaghan survey crew.																	



# RECORD OF BOREHOLE C-6EF

1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station +10+645, offset +1 m right of centreline of Southbound Lane

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Dynamic cone /

COMPILED BY M.D.

DATUM Geodetic

DATE June 10, 1998

CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)		CONE PENETRATION TEST		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR					
300.50	GROUND SURFACE													
0.00	Dynamic cone penetration test only.													
295.90	END OF CONE TEST DUE TO 'BOUNCING' REFUSAL ON PROBABLE BEDROCK OR POSSIBLE BOULDER													
4.60	Notes: 1) This cone test forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Cone test located at U.T.M. coordinates 5 093 077.5 N, 313 922.9 E. 3) Cone elevation obtained by Marshall Macklin Monaghan survey crew.													



## METRIC

ORIGINATED BY I.D.

COMPILED BY M.D.

CHECKED BY I.G.



SO7524G/O/F

**TABLE 1**  
**ROCK CORE DESCRIPTION**

BH#	Core Recovery				Core Description	
	RC #	Depth (m)	% CR*	% RQD**	Depth (m)	Description
<b>SOUND BOUND LANE</b>						
2-OF	1	3.14 to 4.66	100	78	3.14 to 6.46	<b>Biotite Hornblende Gneiss</b> - light grey to pink, medium to coarse grained, unweathered with sulfide inclusions, very close spacing of fractures dipping 45° to 90° from vertical, planar to slightly undulated, rough
	2	4.66 to 6.46	100	88		
3-OF	1	3.35 to 4.81	100	69	3.35 to 6.55	<b>Biotite Hornblende Gneiss</b> - light grey, medium to coarse grained, unweathered with sulfide inclusions, moderately spaced fractures and joints dipping 45° to 90° from vertical, rough and slightly undulating
	2	4.81 to 6.55	100	97		
5-OF	1	5.27 to 6.67	100	90	5.27 to 8.78	<b>Biotite Hornblende Gneiss</b> , light grey to pink, medium to coarse grained, slightly weathered, very close spacing of cemented fissures, joints and fissures dipping 0° to 90° from vertical, smooth and slightly undulating
	2	6.67 to 8.78	100	90		

SO7524G/O/F

**TABLE 1**  
**ROCK CORE DESCRIPTION**

BH#	Core Recovery				Core Description	
	RC #	Depth (m)	% CR*	% RQD**	Depth (m)	Description
7-OF	1	6.37 to 7.90	100	71	6.37 to 9.60	<b>Biotite Hornblende Gneiss (Garnetiferous)</b> , light grey to grey pink, medium to coarse grained, unweathered, moderate spacing of fractures dipping 90° from vertical, slightly undulated, rough
	2	7.90 to 9.60	100	86		
8-OF	1	8.29 to 9.84	100	95	8.29 to 11.43	<b>Biotite Hornblense Gneiss (Garnetiferous)</b> , light grey to grey-pink, medium to coarse grained, unweathered, moderate spacing of joints and fissures, some sulfide inclusions, close spacing of fractures dipping 45° to 90° from vertical, planar and smooth
	2	9.84 to 11.43	100	77		
9-OF	1	14.42 to 15.95	100	81	14.42 to 17.74	<b>Biotite Hornblende Gneiss (Garnetiferous)</b> , light grey to pink, unweathered, very close spacing of cemented fractures, inclusions of sulfides, fractures dipping 0° to 90° from vertical, rough and undulating
	2	15.95 to 17.74	100	100		
<div>*CR = Core Recovery</div> <div>**RQD = Rock Quality Designation</div>						

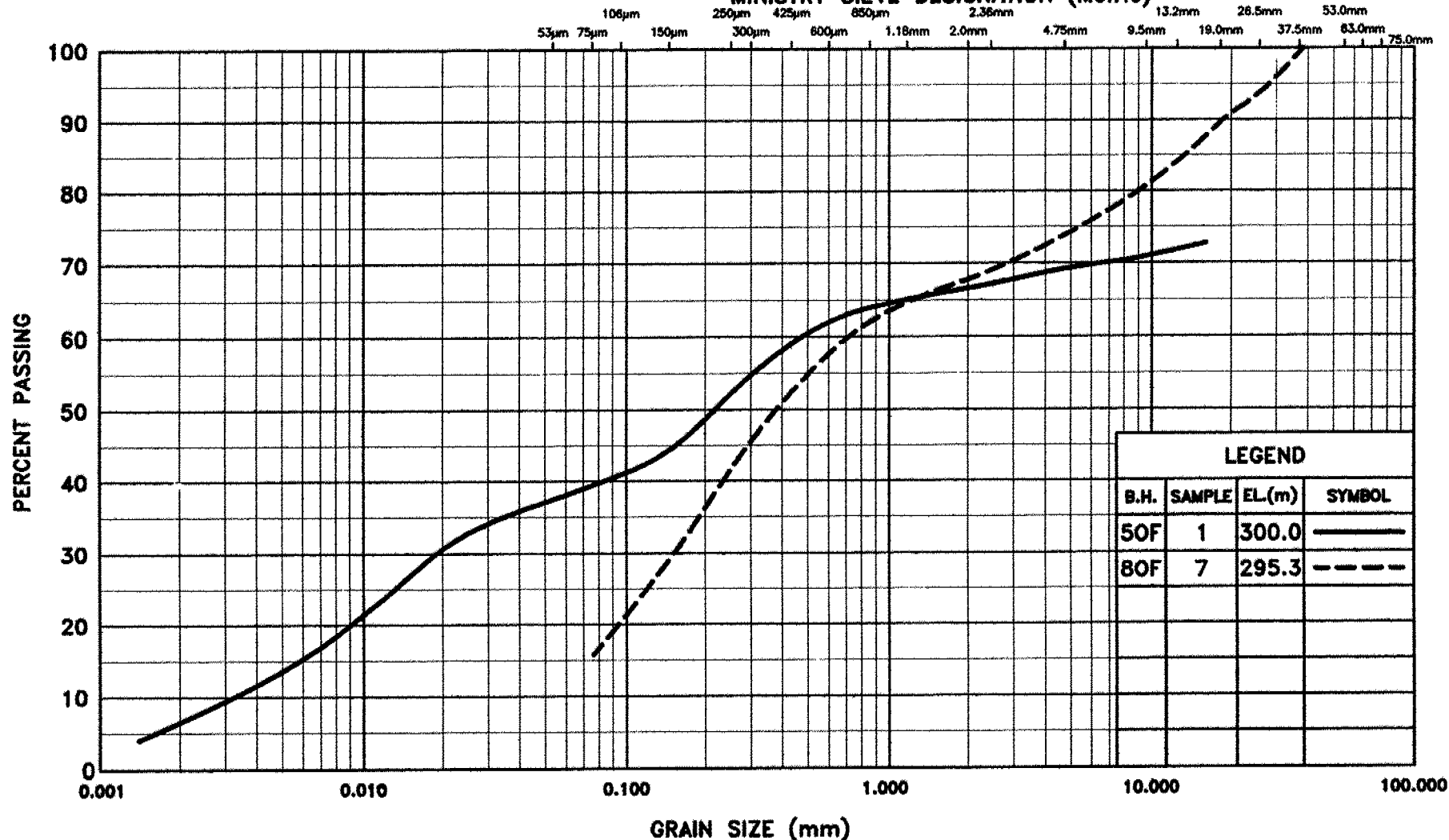
C

—

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



LEGEND			
B.H.	SAMPLE	EL.(m)	SYMBOL
50F	1	300.0	————
80F	7	295.3	-----

Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION  
SAND & GRAVEL

FIGURE C-1

W.P. 774-93-00



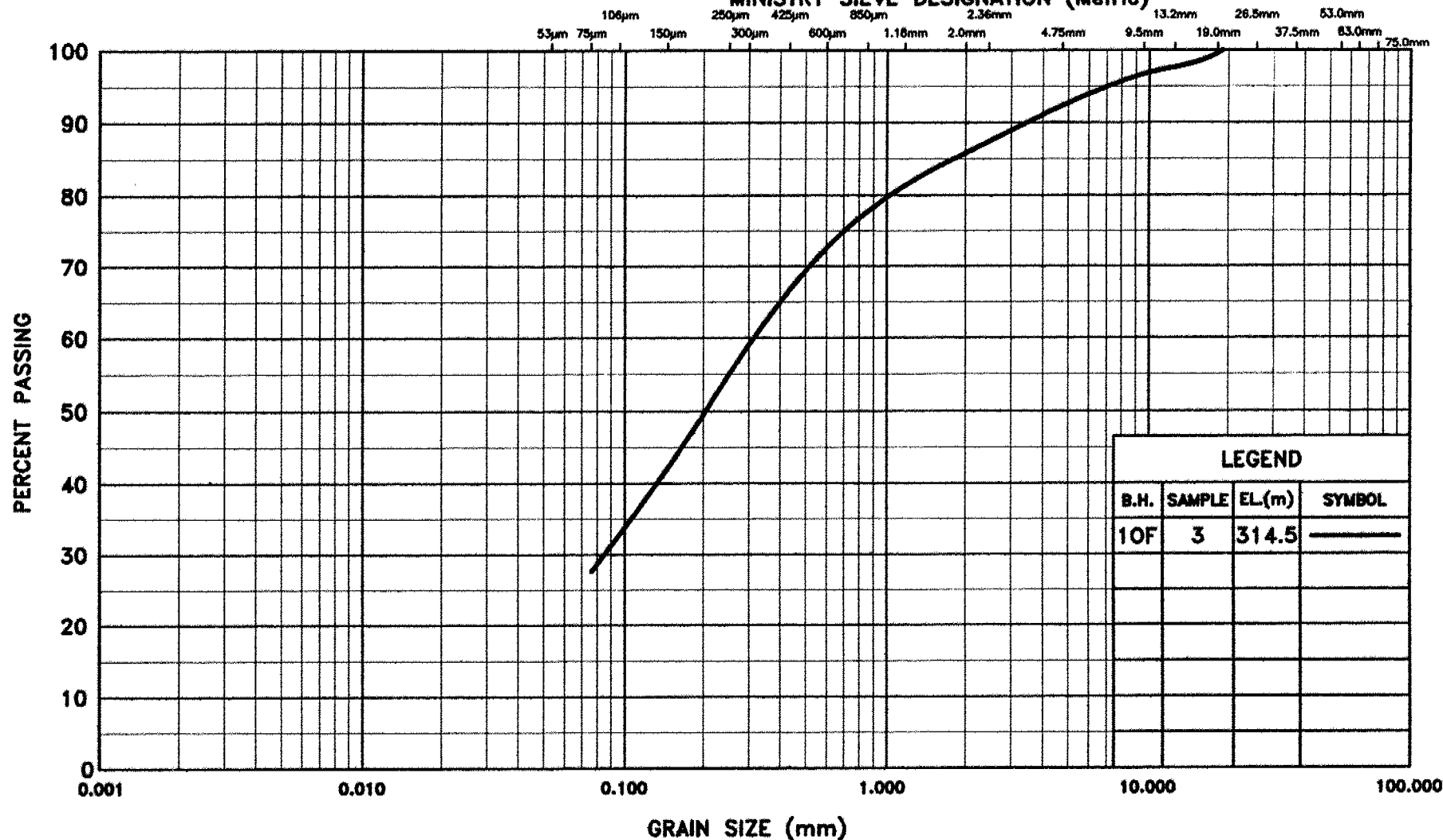
PROJ. No. S07524G0



# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION  
SAND

FIGURE C-2

W.P. 774-93-00

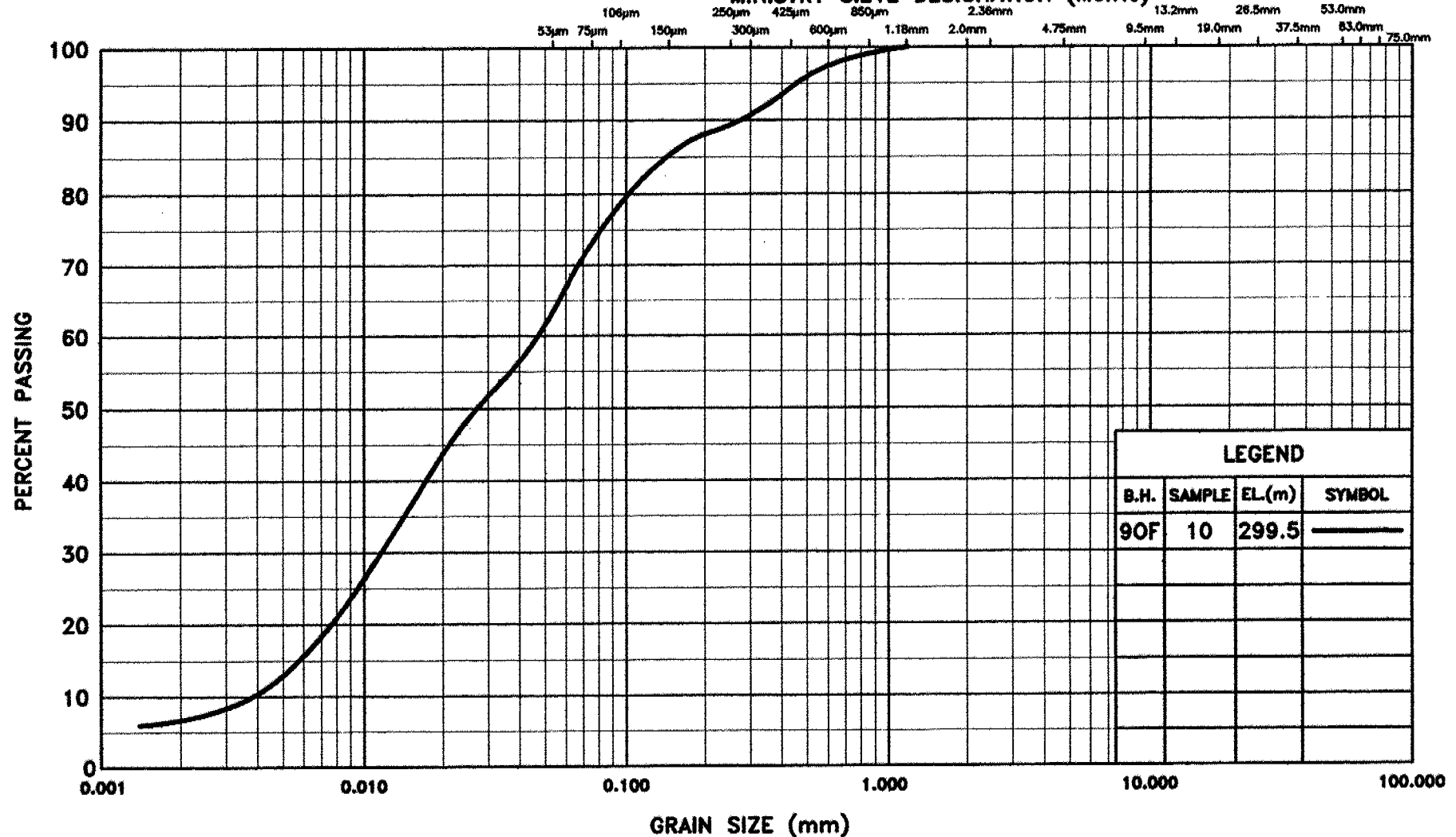


PROJ. No. S07524G0

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION

SANDY SILT

FIGURE C-3

W.P. 774-93-00



PROJ. No. S07524G0

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT

SAND

GRAVEL

FINE

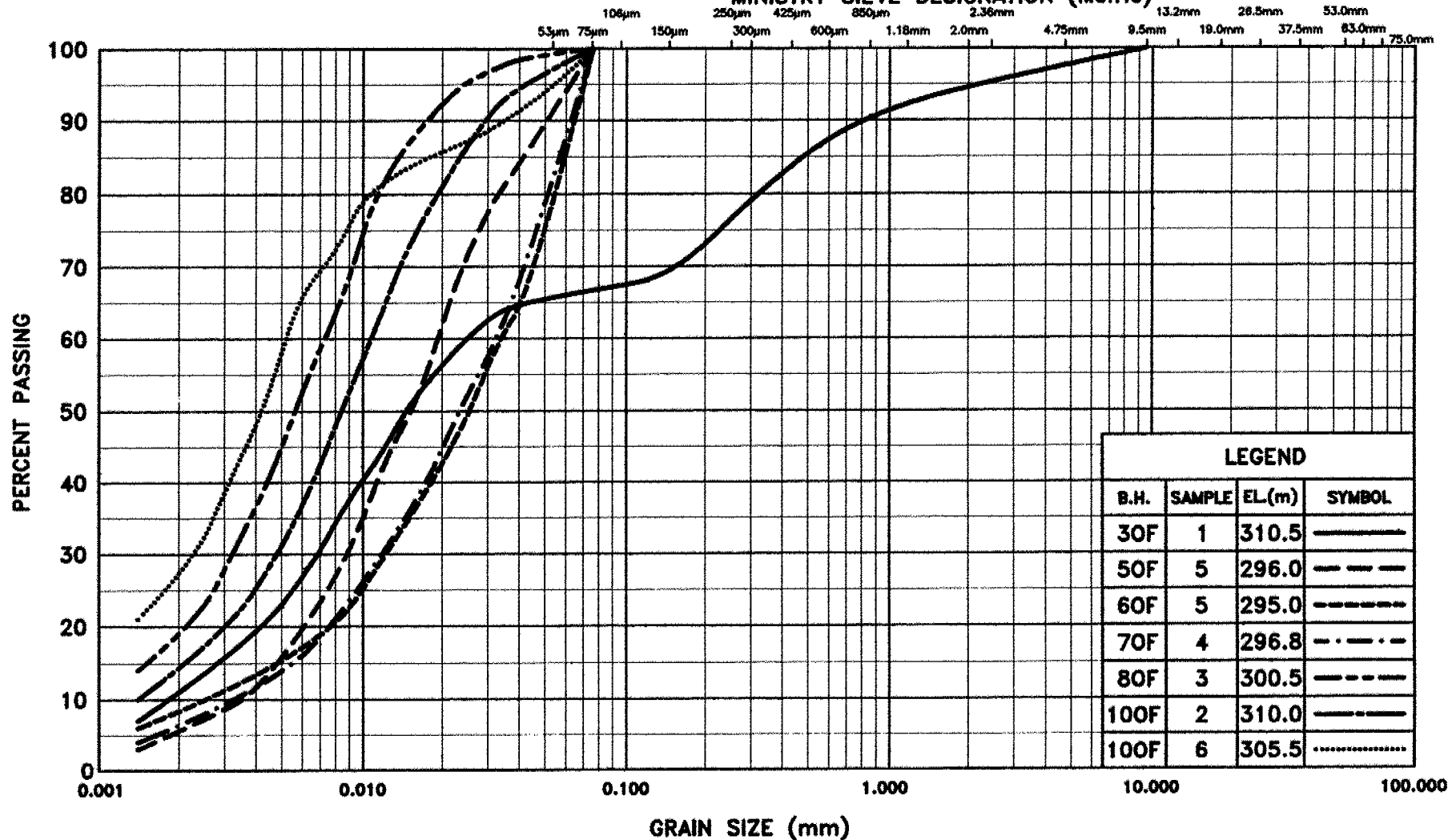
MEDIUM

COARSE

FINE

COARSE

MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION

SILTY CLAY

FIGURE C-4

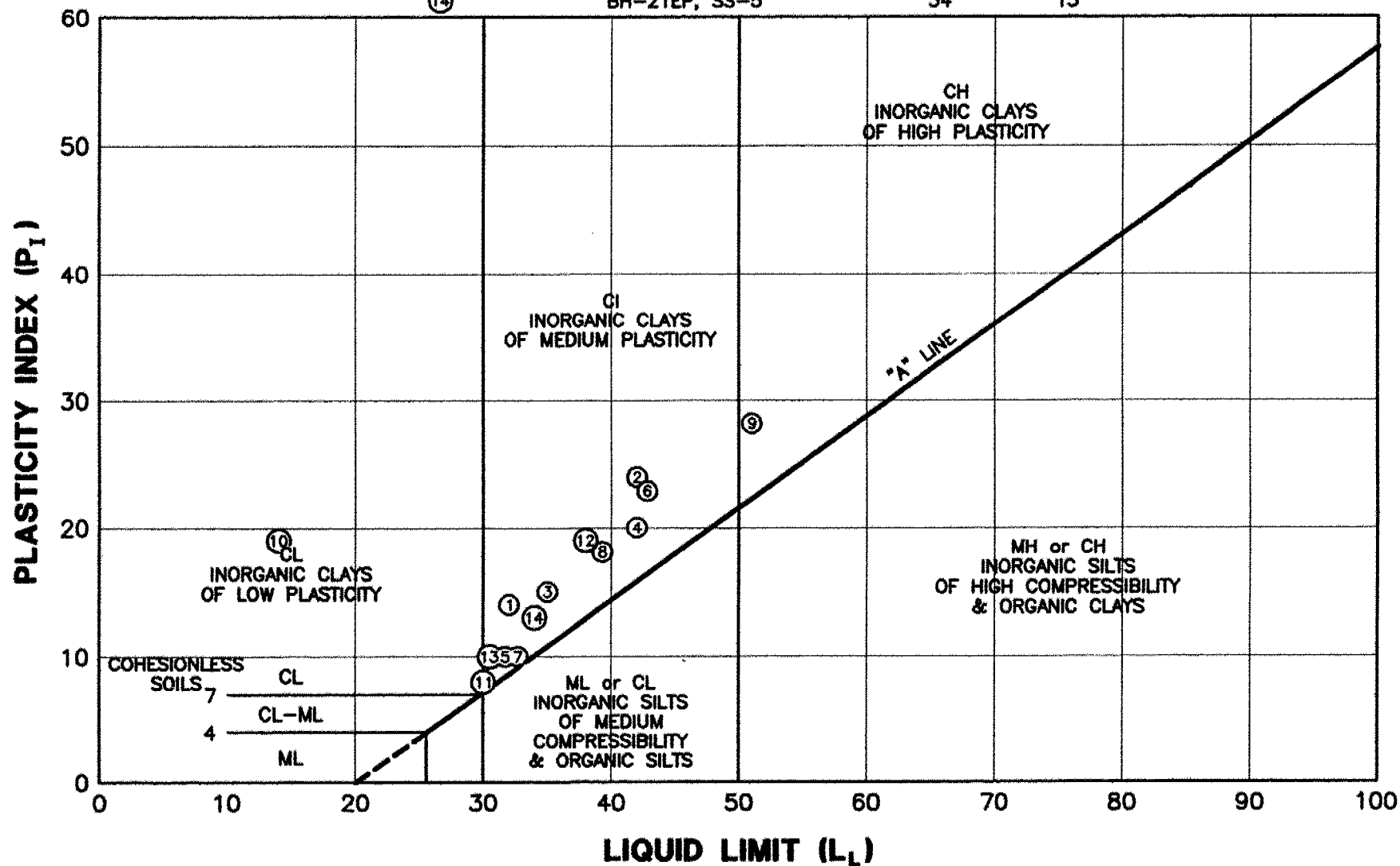
W.P. 774-93-00



PROJ. No. S07524G0

# ATTERBERG LIMITS - PLASTICITY CHART

SYMBOL	DESCRIPTION	LL	PI
①	BH-1EF, SS-5	32	14
②	BH-4EF, SS-2	42	24
③	BH-4EF, SS-4	35	15
④	BH-13EP, SS-2	42	20
⑤	BH-17EP, SS-2	32	10
⑥	BH-21EP, SS-2	43	23
⑦	BH-21EP, TW-4	33	10
⑧	BH-80F, SS-3	39	18
⑨	BH-100F, SS-6	51	28
⑩	BH-90F, SS-2	41	19
⑪	BH-100F, SS-3	30	8
⑫	BH-100F, SS-5	38	19
⑬	BH-100F, SS-8	31	10
⑭	BH-21EP, SS-5	34	13

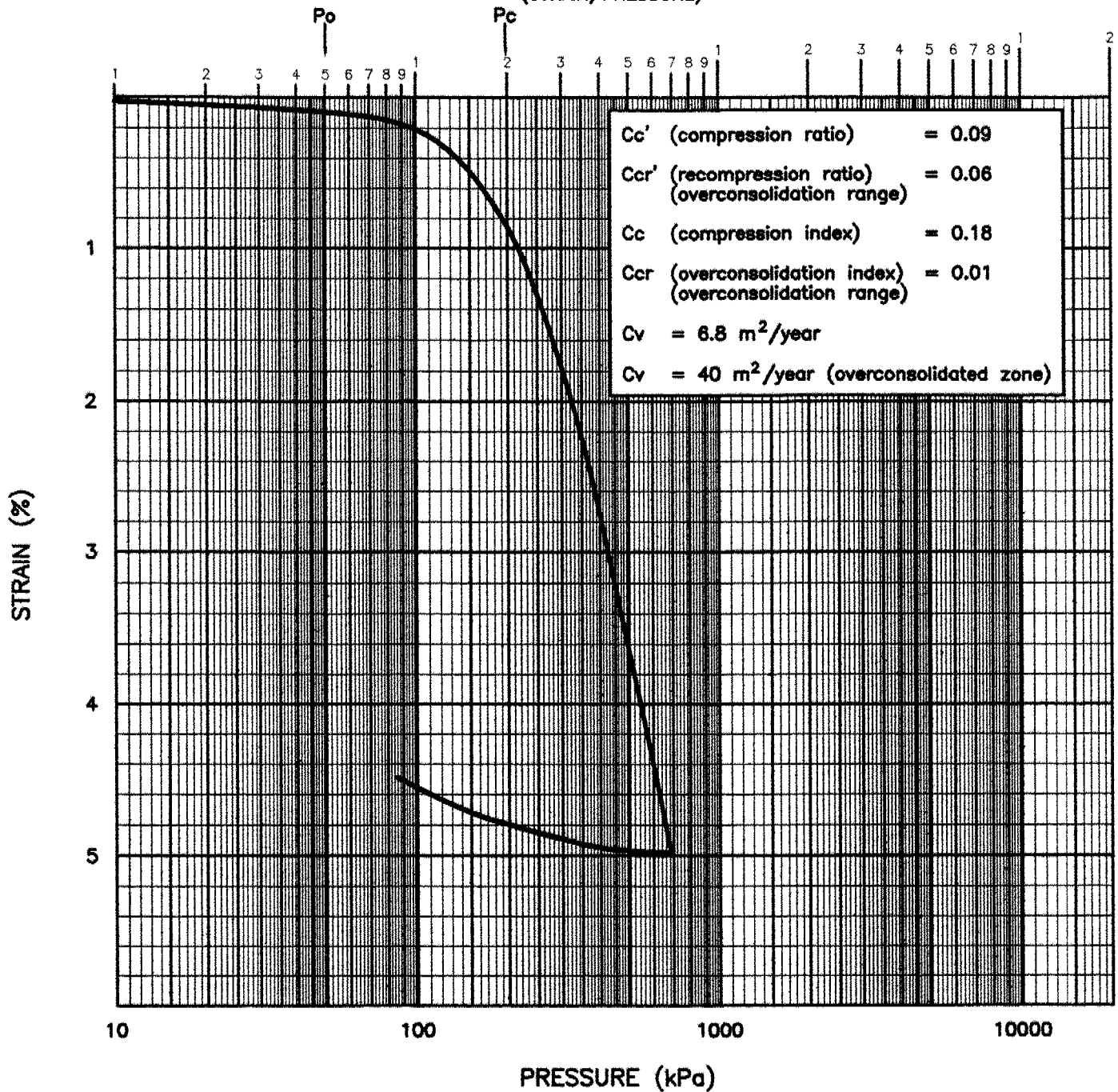


S0752460

FIG. No. C-5



# CONSOLIDATION TEST RESULTS (STRAIN/PRESSURE)

BOREHOLE No. 21EPDEPTH 3 mMOISTURE CONTENT 30.3 %LIQUID LIMIT 32 %PLASTIC LIMIT 22 % $e_s$  1.02UNIT WEIGHT 18.6 kN/m<sup>3</sup>

## SAMPLE DESCRIPTION

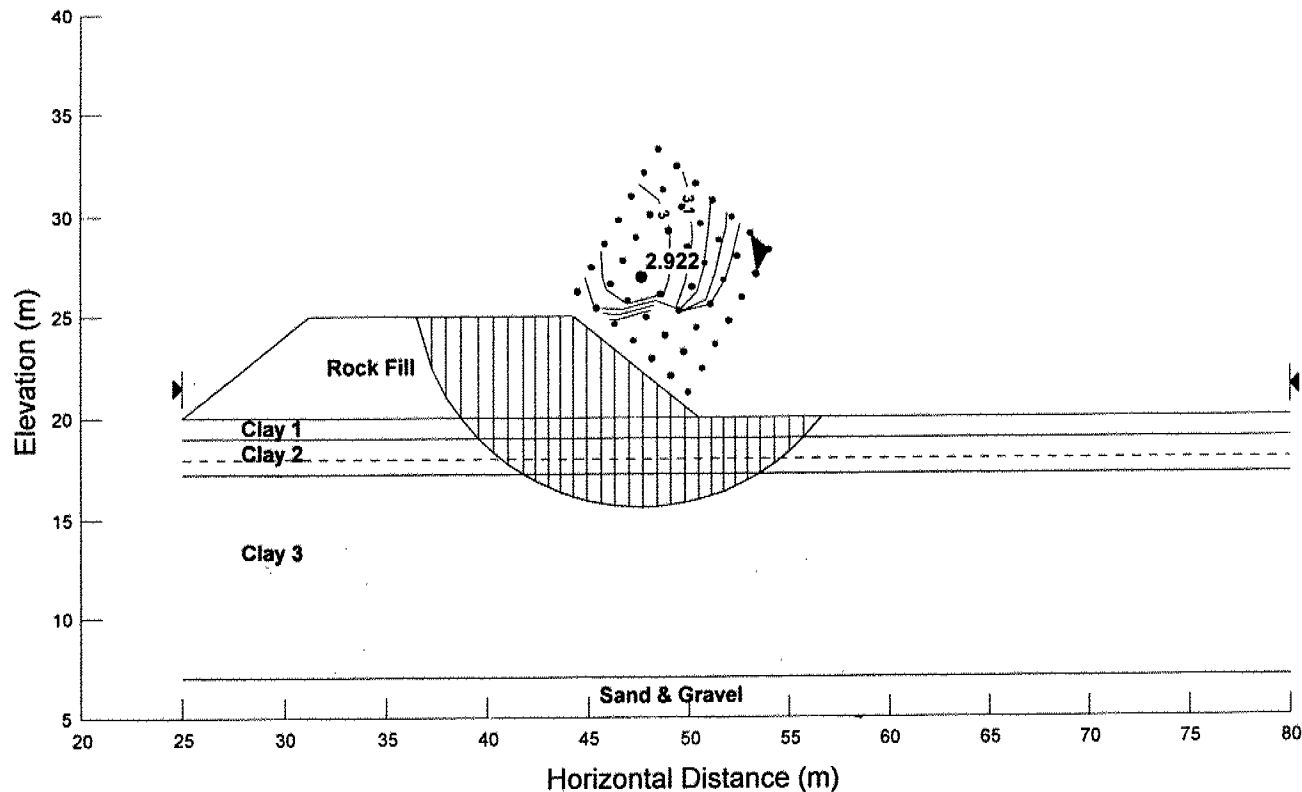
SILTY CLAY with firm to stiff, grey SILT layers

$$M_v = \frac{\Delta \text{ strain}}{\Delta \text{ pressure}} = 0.032 \text{ MPa}^{-1}$$

(COEFFICIENT OF VOLUME COMPRESSIBILITY)

D

Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-C/G)  
 North Abutment - Southbound Lanes  
 5 metre embankment height, 1.25:1 side slopes  
 N\_S5H.SLP



Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 100  
 Rate of Increase -30  
 Cv - Minimum 70  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 70  
 Rate of Increase -28.57  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Clay 3  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 50  
 Rate of Increase 1.95  
 Cv - Maximum 70  
 Ch/Cv Ratio 1

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39

Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-C/G)  
 North Abutment - Southbound Lanes  
 6 metre embankment height, 1.25:1 side slopes  
 N\_S6H.SLP

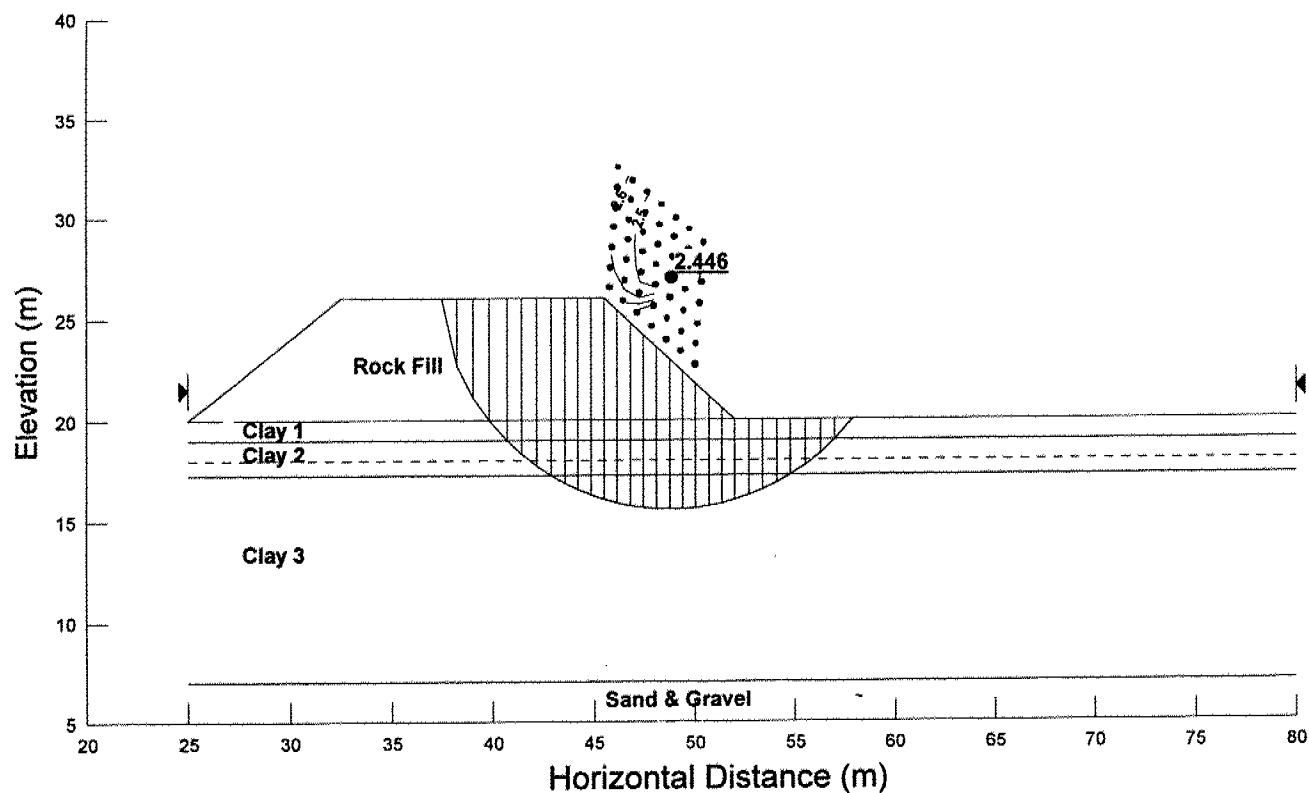
Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 100  
 Rate of Increase -30  
 Cv - Minimum 70  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 70  
 Rate of Increase -28.57  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Clay 3  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 50  
 Rate of Increase 1.95  
 Cv - Maximum 70  
 Ch/Cv Ratio 1

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39





Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-C/G)  
 North Abutment - Southbound Lanes  
 7 metre embankment height, 1.25:1 side slopes  
 4 metre high, 2 metre wide bench  
 N\_S7H.SLP

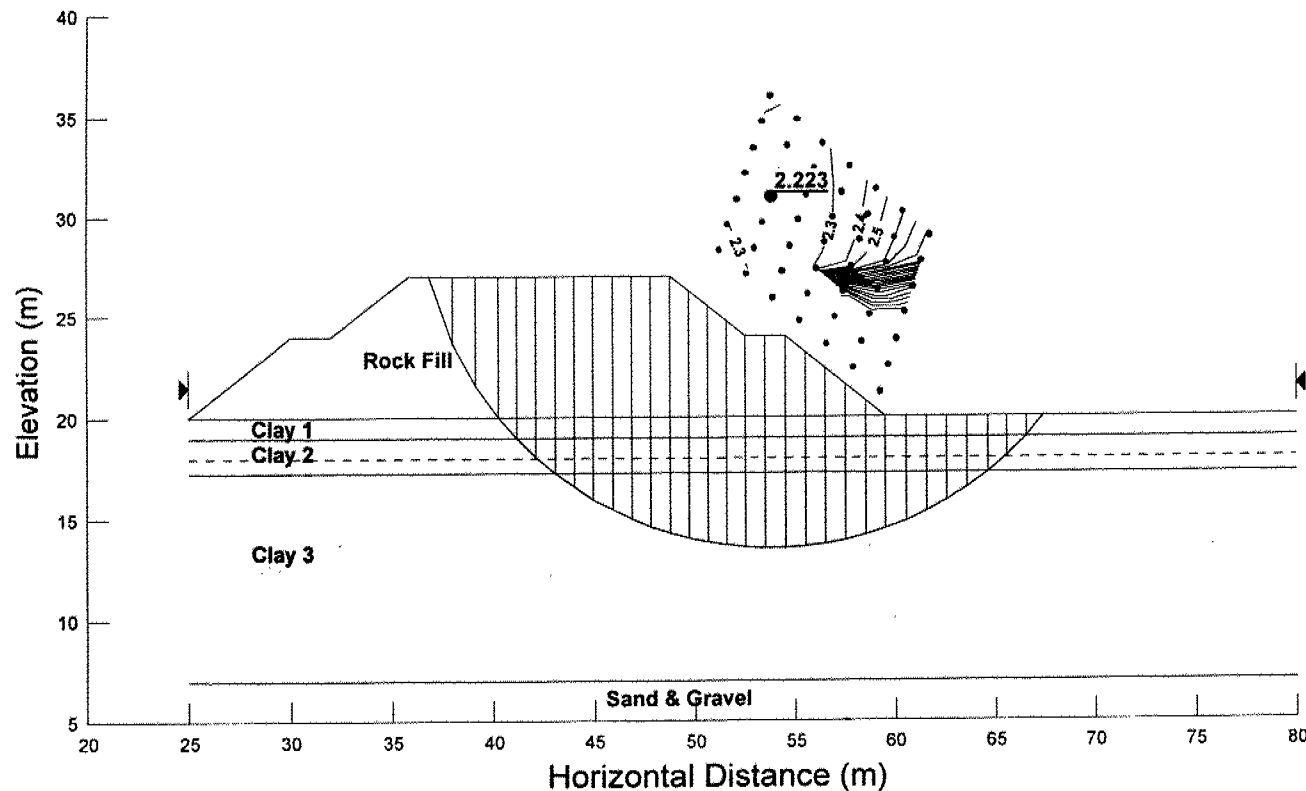
Rock Fill  
 Soil Model Mohr-Coulom  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 100  
 Rate of Increase -30  
 Cv - Minimum 70  
 Ch/Cv Ratio 1

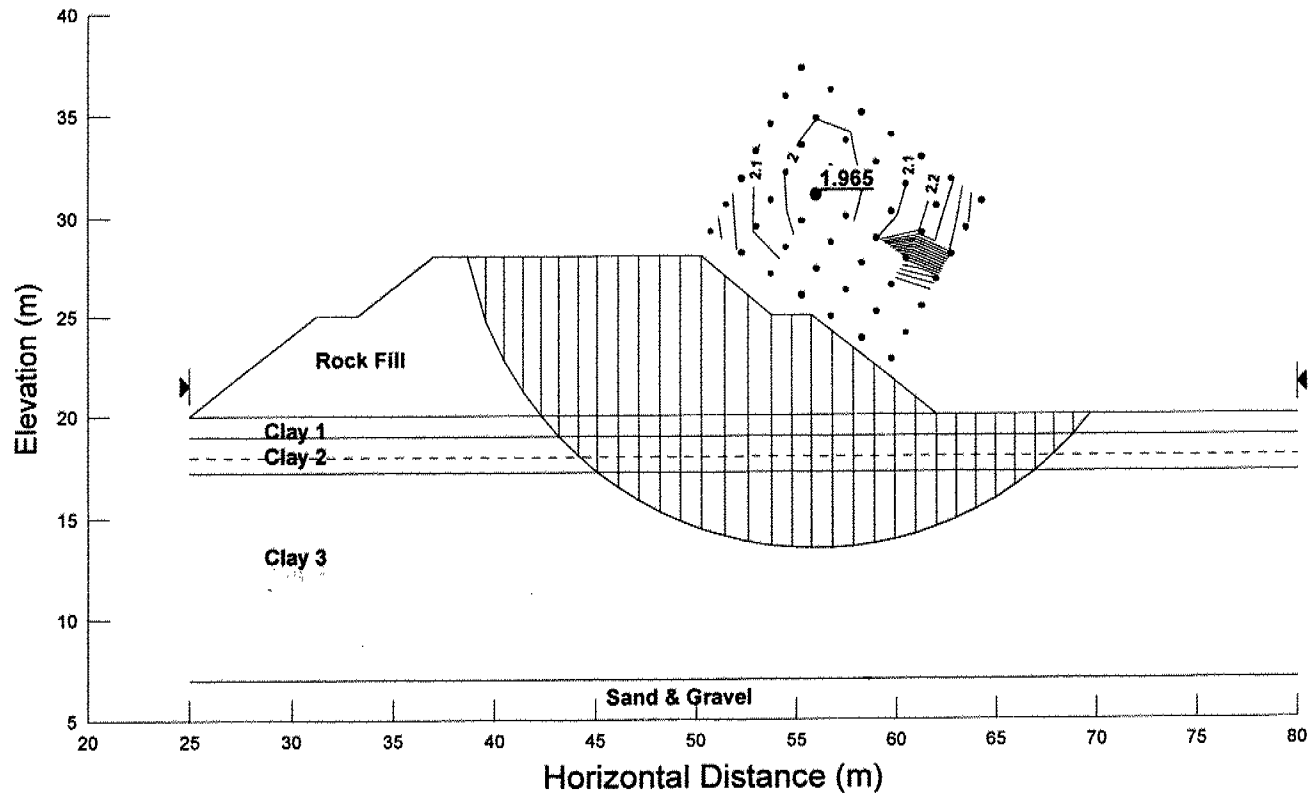
Clay 2  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 70  
 Rate of Increase -28.57  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Clay 3  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 50  
 Rate of Increase 1.95  
 Cv - Maximum 70  
 Ch/Cv Ratio 1

Sand & Gravel  
 Soil Model Mohr-Coulom  
 Unit Weight 20  
 Cohesion 0  
 Phi 39



Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-C/G)  
 North Abutment - Southbound Lanes  
 8 metre embankment height, 1.25:1 side slopes  
 5 metre high, 2 metre wide bench  
 N\_S8H.SLP



Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

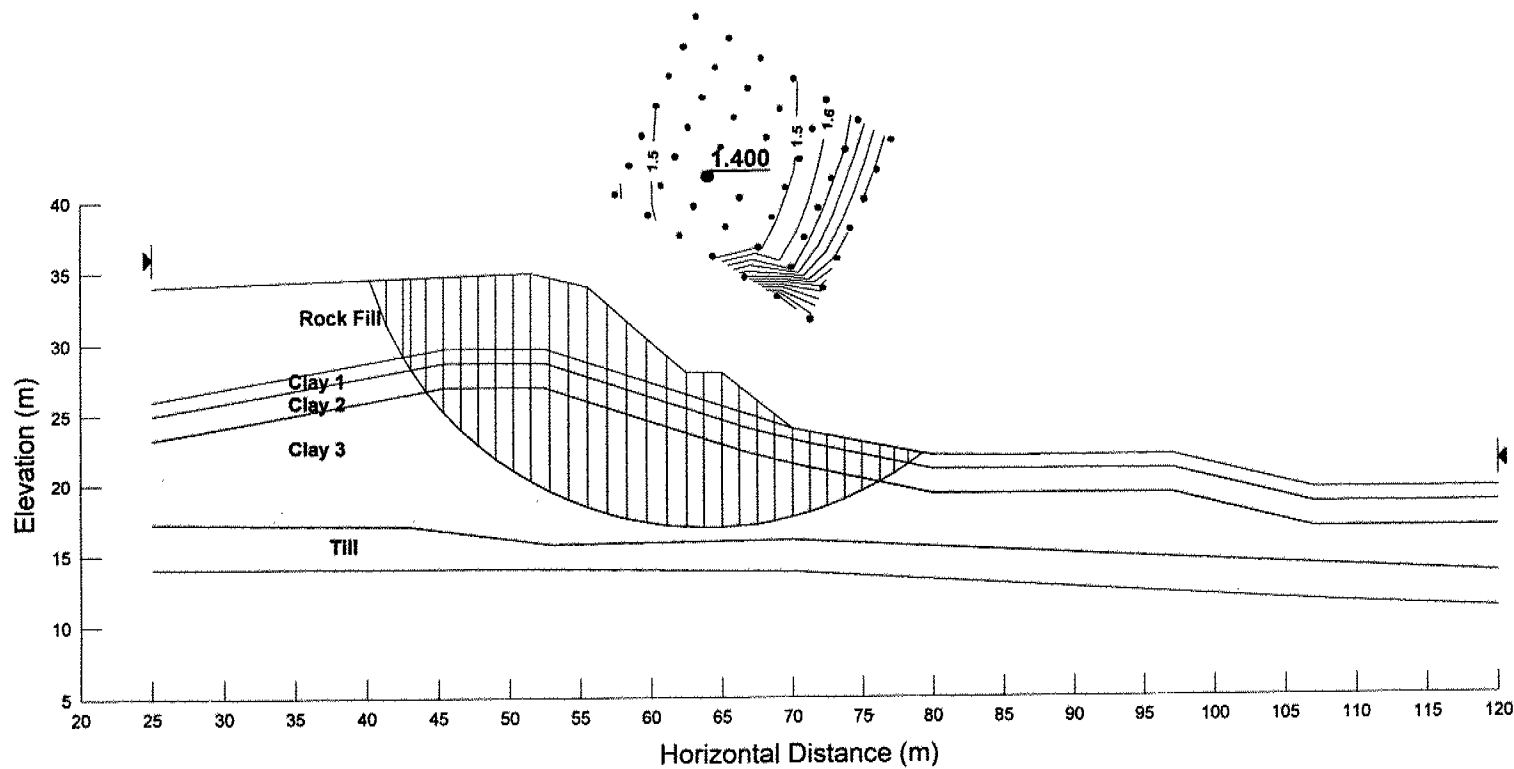
Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 100  
 Rate of Increase -30  
 Cv - Minimum 70  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 70  
 Rate of Increase -28.57  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Clay 3  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 50  
 Rate of Increase 1.95  
 Cv - Maximum 70  
 Ch/Cv Ratio 1

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39

Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-B/G)  
 Southbound Lane, North Abutment  
 With Extended Berm  
 SBL\_NALV.SLP



Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 100  
 Rate of Increase -30  
 Cv - Minimum 70  
 Ch/Cv Ratio 1

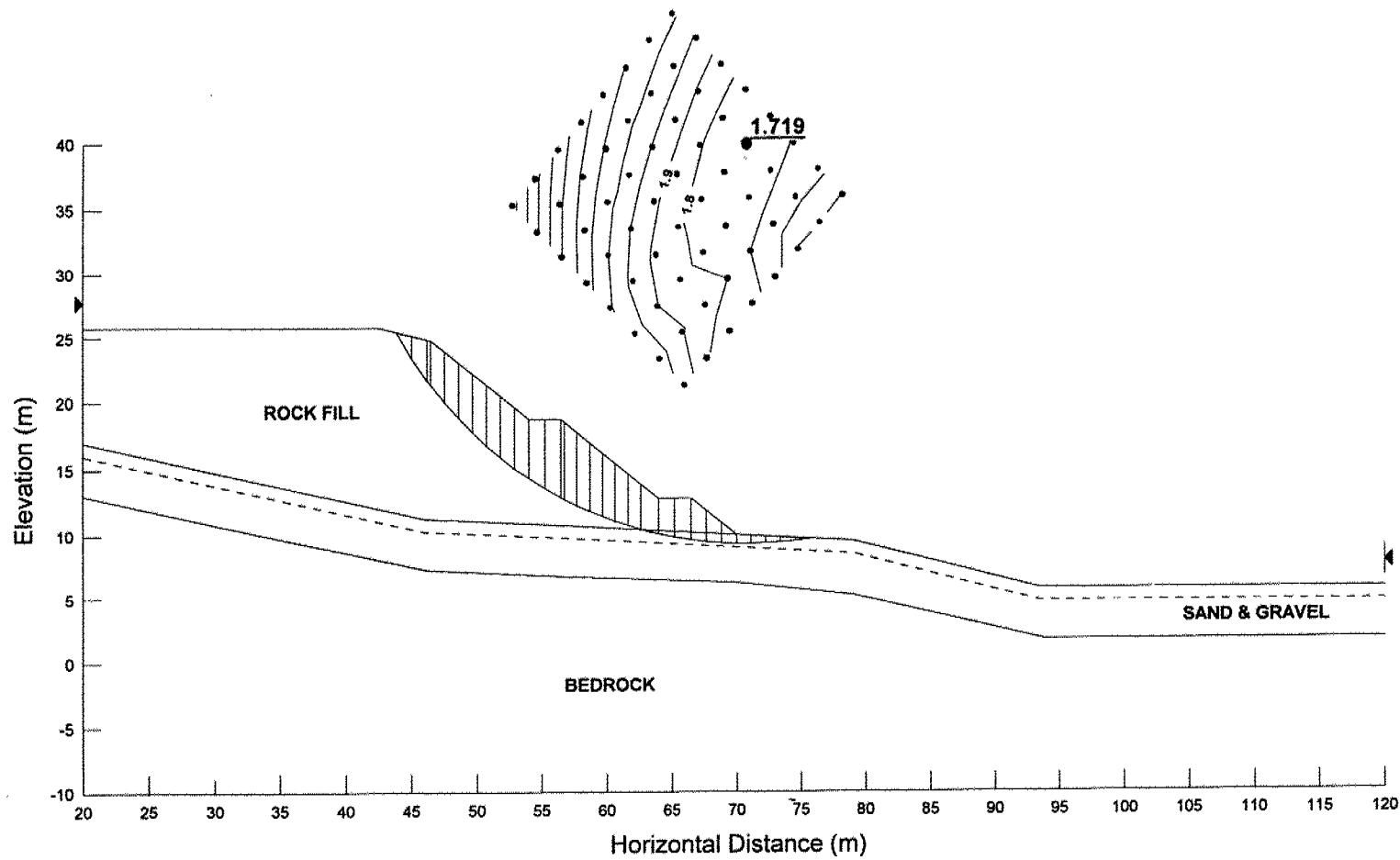
Clay 2  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 70  
 Rate of Increase -28.57  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Clay 3  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 50  
 Rate of Increase 1.95  
 Cv - Maximum 70  
 Ch/Cv Ratio 1

Till  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35  
 Unsaturated Phi B 0

Bedrock  
 Soil Model Bedrock  
 Unit Weight -1  
 Piezometric Line # 0  
 Pore-Air Pressure 0

Slope Stability - Total Stress Analysis  
Trout Creek - Highway 11 (F-98179-C/G)  
Southbound Lane, South Abutment  
With Extended Berm  
SBL\_SA.SLP



Rock Fill  
Soil Model Mohr-Coulomb  
Unit Weight 20  
Cohesion 0  
Phi 42

Sand and Gravel  
Soil Model Mohr-Coulomb  
Unit Weight 21  
Cohesion 0  
Phi 35

Bedrock  
Soil Model Bedrock

**Foundation Investigation & Design Report  
Bridge Structure & Approaches  
Trout Creek (Site 44-371N)  
NORTHBOUND LANES  
Trout Creek By-Pass, King's Highway 11  
District 54, Sudbury, Ontario  
GWP No. 774-93-00**

Prepared For:

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November 24, 1999

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## Preface

Work Project GWP 774-93-00 is one of a series of projects for the four lane expansion of Highway 11. This project involves the four lane design of Highway 11, from 4.0 km south of Highway 522, northerly for 7.9 km. It will result in the construction of a by-pass of the existing Highway 11 to the west of the Town of Trout Creek.

This highway section is located in the Townships of Laurier and South Himsworth, within the geographic District of Parry Sound, as shown on the Key Plan, Figure A1 in Appendix A. The project requires geotechnical input for the following major components:

- New pavement design for the entire length of the four lane by-pass, including associated service roads.
- New structure, Trout Creek South Interchange (underpass), Site 44-372.
- **New structure, Trout Creek Northbound Lanes, Site 44-371N.**
- New structure, Trout Creek Southbound Lanes, Site 44-371N.
- New structure, Highway 522 (underpass), Site 44-370.
- New structure, Trout Creek North Interchange (underpass), Site 44-369.

This report deals with the new bridge structure for the **northbound lanes** at the proposed Trout Creek crossing, Site 44-371N, as well as the approach fills. Separate reports will be submitted for the additional components.

Part 1 of this report, Foundation Investigation, describes the geotechnical investigation methodology and the results derived, as pertaining to the design parameters required for the bridge foundations and related earthworks.

Part 2 of this report, Engineering Discussion and Recommendations, addresses the geotechnical engineering design considerations pertaining to the bridge foundations and the approaches.

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## Appendix A

Figure A1.	Key Plan
Figure A2a.	Borehole Location Plan and Profile - South
Figure A2b.	Borehole Location Plan and Profile - North
Figure A3.	Undrained Shear Strength, Atterberg Limits and Effective Stress Profiles
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Figure A6.	Estimated Consolidation Settlement - SOUTH APPROACH
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## Appendix B

Borehole Logs and Rock Descriptions

## Appendix C

Results of Laboratory Testing (grain size, Atterberg Limits, consolidation tests)

## Appendix D

Stability Analysis Printouts

## Rear Pockets

Drawing No. 1. Bridge Site Plan & Profile  
Drawing No. 2. Bridge Site Plan & Sections



## **Part 1 Foundation Investigation**

### **1.1 Introduction**

This report presents the results of a geotechnical foundation investigation by Trow Consulting Engineers Ltd. (Trow) for the proposed bridge structure and approaches for the northbound lanes (NBL) at the Highway 1, Trout Creek crossing at Site 44-371N.

An original investigation was completed at this site in June and July of 1998 for an originally proposed 105 m long ( $\pm$ ) three span structure. However, for technical reasons based on the results of the original investigation, as discussed more fully in Part 2 of this report, a longer (242 m), five span bridge was selected. Accordingly, a supplementary investigation was completed during September and November, 1998 which investigated the subsurface conditions at the revised pier and abutment locations of the new proposed five span structure, where conditions were not previously investigated as part of the original work.

This report contains the results of the supplementary investigation, plus the relevant results of the original investigation, compiled for the five span structure arrangement of the Highway 11, northbound lane crossing of Trout Creek. It is Trow's understanding that the 5 span structure will be located with the central span crossing Trout Creek. The structure will include an approximately 11 metre high south abutment, 22 metre high centre span, and 5 metre high north abutment. The span lengths will be approximately 40, 54, 54, 54, 40 metres in length, creating a bridge with a length of about 242 metres.

### **1.2 Site Description and Geological Setting**

The site is located in Lot 30, Concession 2, Township of South Himsforth, District of Parry Sound, along the banks of Trout Creek, about 2 km west of the Town of Trout Creek, and 750 metres south of Highway 552, as shown on Figure A1, in Appendix A.

Generally, the terrain at the site is sloping towards the creek and is well drained. However several drainage gullies run parallel and perpendicular to the creek, with steep embankments on either side of the gullies, creating a highly variable terrain in the vicinity of the proposed structure. The relief

can vary at slopes steeper than 1H:1V within the site. There are mature trees with heavy underbrush across the site.

According to OGS Maps 2544 and 2556, as well as Ontario Geological Survey Map P.3160 (Quaternary Geology, South River Area), the site is located in what is known as the Central Gneiss Belt, with mainly felsic igneous rocks of the Mesoproterozoic Group.

The overburden soils have been mapped as glaciolacustrine deposits consisting of laminated, rhythmically bedded (also referred to as varved) to massive silts and clays. The southernmost portion of the site edges onto an area mapped as a bedrock drift complex with relatively thin, primarily cohesionless soil cover (till).

## **1.3 Investigative Procedures**

### **1.3.1 General**

Part 1 of this report describes the investigative procedures used for the geotechnical assessment of the northbound lanes crossing of Trout Creek, within the Trout Creek By-Pass, King's Highway 11. Properties of the overburden soils were obtained by *in situ* and laboratory testing and the procedures employed during the investigation are described below.

### **1.3.2 Field Investigation**

The initial field investigation, or explorations, for the originally proposed three span arrangement was carried out between June 9 and July 2, 1998. Originally these explorations investigated only the pier and abutment locations, as well as the immediate approach embankments. With the discovery of clay at this site additional explorations were advanced, while on site, to outline the vertical and areal extent of the clay within the approach embankments. A further investigation of the north approach was completed September 23 and 24, 1998. The supplementary investigation of the revised, five span, arrangement occurred between November 10 and 24, 1998. The locations of the boreholes, cones and probes, completed as part of these investigations are shown on Figures A2a and A2b, in Appendix A, as well as on Drawings, No. 1 and No. 2, located in the pockets at the end of this report. These locations, as well as the surface elevations, were established from the terrain model for the project, and/or a survey crew from Marshall Macklin Monaghan, and are referenced to geodetic datum.

The investigation of the piers and abutments of the five span arrangement includes 13 boreholes (BH-1NF to BH-10NF, inclusive, BH-8DP, BH-3DF, BH-18DF), two dynamic cone penetration tests (C-3DF, C-5DF), and four auger probes (AP-1NF to AP-4NF inclusive). All of these explorations were advanced to auger refusal or to refusal of the penetration cone ( $N > 100$ ). Six of the boreholes were advanced into the bedrock to obtain core, as described below.

The investigation of the approaches included six additional boreholes (BH-23DP, BH-24DP, BH-10DP, BH-11DP, BH-12DP, BH-18EP). Boreholes BH-23DP and BH-24DP were advanced within the northern approach and were advanced to beneath the upper silt and clay layer. Boreholes BH-10DP, BH-11DP, BH-12DP and BH-18EP were advanced within the southern approach, until auger refusal.

Other explorations completed in the vicinity of the NBL structure, as part of the original three span investigation, or as part of the subsequent investigation to determine the horizontal extent of the clay layer included 16 boreholes (BH-1DF, BH-2DF, BH-4DF, BH-6DP, BH-7DP, BH-9DP, BH-11 EP, BH-13DP, BH-14DP, BH-15DP, BH-16DP, BH-17DP, BH-19DF, BH-20 DP, BH-21DP, BH-22DP), five dynamic cone penetration tests (C-1DF, C-2DF, C-4DF, C-5DF, C-6DF) and two auger probes (AP-1DF, AP-2DF), plus those explorations completed as part of the SBL investigations (see foundation report for the SBL structure).

The boreholes, cones and probes were advanced through the overburden soils using a Bombardier mounted CME-55 drill and a Dietric 50 drill, equipped with solid and hollow stem augers, and supplied by a soils drilling contractor, Master Soil Investigations Limited. At some of the borehole locations, a bulldozer was used to prepare the site for drill access.

Soil samples were obtained by using a 51 mm OD split-spoon sampler in conjunction with standard penetration tests at approximately 0.75 m and 1.5 m intervals. The standard penetration ( $N$ ) values, together with the blows from the dynamic cone penetration tests, were recorded and used to provide an assessment of the compactness or consistency of the overburden soils. Sampling and testing procedures were in general accordance with ASTM D1586.

Several undisturbed, nominal 50 mm diameter, 'Shelby' tube samples were also obtained in the cohesive deposits. Field vane testing was completed in the boreholes and the auger probes throughout the cohesive soils to measure the *in situ* undrained shear strength of the cohesive soils. The field vane used had dimensions of 152 mm long by 70 mm diameter, not including the pointed 45° point. Torque measurement was by using two calibrated scales on a calibrated lever arm threaded to the drill rods. The testing was in general accordance with ASTM D2573.

At each bridge foundation element, conventional rock coring techniques were used to advance one of the explorations approximately 3 metres into the underlying bedrock. A BQ size core barrel and casing was used and core samples of the bedrock were retrieved for rock quality determination and classification. These six explorations advanced into the underlying bedrock included BH-2NF, BH-3NF, BH-5NF, BH3-DF, BH-7NF and BH-10NF.

Details of the soil and bedrock conditions encountered in the boreholes are included on the logs in the attached Appendix B.

### **1.3.3 Laboratory**

The laboratory testing program for selected soil samples consisted of the following:

- natural moisture content
- unit weight
- grain size distribution
- laboratory shear tests (lab vane, penetrometer)
- Atterberg limits
- 1-d consolidation test

The laboratory test results are summarized on the attached borehole logs in Appendix B and are also presented in Appendix C. Many of the results are also discussed in the following descriptive sections.

## **1.4 Subsurface Conditions**

The borehole locations are shown on the site plans, Figures A2a and A2b in Appendix A, as well as on Drawings, No. 1 and No. 2. Centerline soil profiles are also shown on the Figures and Drawing No. 1 and the soil cross sections at the locations of the foundation elements are shown on Drawing No. 2.

The borehole, auger probe and dynamic cone penetration (CPT) logs are provided in Appendix B. In general, the following main soil layers were encountered, with increasing depth:

- topsoil
- sand
- silty sand
- silty clay

- silty sand and gravel
- bedrock

A summary of the soil strata encountered in the boreholes, and inferred from the probes and dynamic cone penetration tests, is presented below.

#### **1.4.1 Topsoil**

The majority of boreholes encountered a surficial layer of approximately 150 mm to 200 mm of topsoil. At some of the borehole locations, a bulldozer was used to prepare the site for drill access, thus the topsoil in these areas was removed in the process and the logs do not indicate the presence of topsoil.

#### **1.4.2 Sand**

A layer of loose to compact (N-values from 5 to 25), fine to medium grained brown sand was encountered beneath the topsoil at many of the boreholes advanced at the foundation elements. The thickness ranged from less than 200 mm to over 2 m, and was usually underlain by silty clay, as described below. For the boreholes drilled at the foundation elements, the sand was not encountered in Boreholes, BH-5NF, BH-6NF, BH-8NF, BH-3DF, BH-18DF, BH-5DF, BH-10DF or BH-23DF.

#### **1.4.3 Silty Sand**

Brown to grey, very loose to compact silty sand with local gravel was encountered beneath the topsoil in Boreholes, BH-3DF and BH-18DF at the location of Pier WP4, adjacent to the north bank of Trout Creek. The standard penetration indices ranged from about 1 to 17. The soil contained organics such as roots and pieces of wood and may be a recent stream alluvium. This soil was underlain by silty sand and gravel at a depth of about 4 m.

#### **1.4.4 Silty Clay**

Beneath the upper sand or topsoil, a stratum of silty clay was encountered as the principal soil in all boreholes at the locations of the foundation elements except BH-2NF (WP1 - south abutment) and BH-3DF and BH-18DF (WP4 - pier). In BH-2NF, silty sand and gravel, as described in the next section was encountered. In BH-3DF and BH-18DF, silty sand, as described in the previous section was encountered.

The silty clay is relatively thin (3 m to 4 m) at the location of the south abutment and increases in thickness to between about 12 m to 14 m on the north side of Trout Creek, in the areas of Pier WP5 and the north abutment and approach. Generally, the silty clay is thinly laminated with silt (varved). The individual layers, or laminations, varied in thickness from a few millimetres to a few centimetres, but in general were less than about one centimetre. The proportions of clay to silt and clayey silt varied also, but in general the clay portion dominated.

The natural moisture content of the clay varies from about 20% to over 50% (depending on the silt content) and consistency. Atterberg limit determinations provided the following approximate ranges: Plastic Limit, 18 to 23; Liquid Limit, 28 to 50; Plasticity Index, 8 to 30. These data indicate that, in general, the clay can be described as a low plasticity silty clay (CL) to medium plasticity silty clay (CI). Typically, the silty clay is drier near the top and bottom, but there does not appear to be a depth relationship relative to the Atterberg Limits. The laboratory test data are shown on the borehole logs, on Figure A3 in Appendix A, and in Appendix C.

Standard penetration test (SPT) values ranging from about 1 to over 20 were obtained in the silty clay. The higher values were generally obtained within the upper metre, or so. *In situ* field vanes and laboratory shear vane testing, as well as the SPT values, indicate that the silty clay has a stiff to very stiff crust with undrained shear strengths exceeding 100 kPa. The silty clay consistency, however, reduces with depth to soft to firm to stiff, with undrained shear strengths of about 20 kPa to 70 kPa, at depths of about 3 m to 5 m. Sensitivity, the measure of peak shear strength to remoulded shear strength, ranged from about 2 to 16, with an average of about 7, indicating the clay is moderately sensitive. The undrained shear strength profile which includes shear strength data from all boreholes is shown on Figure A4. The strength profile shown on Figure A3 is based on the strength data from boreholes at the locations of the abutments and the design profile assumed is based on the actual vane test results and the SPT values.

Based on all the above, and with reference to Figure A3, it is evident that the clay is heavily overconsolidated in the upper 3 m to 4 m, becoming moderately to lightly overconsolidated with increasing depth. The preconsolidation pressure near the top of the stratum is estimated at about 400 kPa, on average. The overconsolidation ratio (OCR) is estimated as ranging from over 30 near the top to about 3 at a depth of about 3 m. Thereafter with increasing depth, the OCR decreases gradually to about 1.6 at a depth of 14 m.

A one-dimensional consolidation test was performed on a sample of the silty clay extruded from a thin walled Shelby tube, obtained from BH-21EP. The results are presented graphically and numerically

in Appendix C. The data are also summarized below in Table 1-1, along with the value ranges used in our subsequent analyses.

<b>Table 1-1. Consolidation Parameters for Silty Clay</b>		
	<b>BH-21EP, 3 m</b>	<b>Values for Analyses</b>
Compression ratio, $C_c' (= C_c/(1+e_0))$	0.08	0.08 - 0.20
Recompression ratio, $C_r' (= C_r/(1+e_0))$	0.006	0.008 - 0.02
Coefficient of consolidation (recompression), $C_{vr}$	40	25 - 60 (avg 40)
Coefficient of consolidation (virgin), $C_v$	7	5 - 12 (avg 8)
Coefficient of secondary compression, $C_{\alpha\epsilon}^*$	0.003 - 0.004	0.003 - 0.005
Notes: Coefficients of consolidation in units of $m^2/year$ * $C_{\alpha\epsilon} = \Delta H/H_0$ (= secondary compression ratio, in some literature)		

#### 1.4.5 Silty Sand and Gravel

A basal deposit of loose to dense, brown silty sand and gravel was encountered above the bedrock (or refusal) surface in most of the boreholes. Standard penetration indices (N-values) ranged from about 6 to 58. Its thickness varied from less than about 0.5 m to over 4 m.

#### 1.4.6 Bedrock

The presence of bedrock was established by retrieving "BQ" size cores in one sampled borehole at each of the six foundation element locations, for depths of between about 3.1 m and 3.4 m. Detailed descriptions of the rock are presented in Table 1-1 in Appendix B, following the borehole logs. Generally, the bedrock can be described as a pink and light grey, biotite-hornblende gneiss. The rock is strong and unweathered for the most part.

Rock core recovery was 100% for all runs and the Rock Quality Designation (RQD) values for individual core runs ranged from about 10% to 96%. The average RQD for the rock core recovered was about 70%, based on the 20 core runs. In accordance with the MTO classification system, the rock quality can be described as very poor to excellent, with an average of fair. It is noted that the RQD values are likely conservative; it is expected that higher values would be obtained using NQ core.

Table 1-2, below, lists the bedrock depths and elevations as well as those of refusal, at the locations of each of the six foundation elements. It can be seen that the bedrock and refusal depths and elevations are quite variable, even within short distances at the individual element locations. Refusal (to augering or dynamic cone penetration testing (CPT)) is inferred to be due to probable bedrock, but it is noted that refusal may be due to cobbles, boulders, or very dense soil. The bedrock depths and elevations have been positively established only at the locations where the bedrock has been cored.

<b>Table 1-2. Bedrock and Refusal Depths and Elevations at Boreholes</b>					
<b>Location</b>	<b>Borehole</b>	<b>Orig. Gr. Elev. (m)</b>	<b>Bedrock or Refusal Elev (m)</b>	<b>Overburden Thickness (m)</b>	<b>Comment RQD (by run)</b>
<b>WP1 South Abutment</b>	BH-1NF	313.8	307.46	6.34	auger refusal
	BH-2NF	314.3	310.43	3.87	B/R cored 87%, 96%
	AP-1NF	313.82	308.58	5.24	auger refusal
	AP-2NF	313.91	309.76	4.15	auger refusal
	AP-3NF	313.9	309.72	4.18	auger refusal
	AP-4NF	314.21	310.6	3.61	auger refusal
<b>WP2 Pier</b>	BH-3NF	312.54	302.42	10.12	B/R cored 78%, 90%
	BH-4NF	313.51	304.52	8.99	auger refusal
	BH-8DP	313.26	303.51	9.75	auger refusal
<b>WP3 Pier</b>	BH-5NF	307.04	300.12	6.92	B/R cored 10%, 20%, 75%
	BH-6NF	304.88	296.04	8.84	auger refusal
	C-5DF	305.9	299.42	6.48	CPT refusal
<b>WP4 Pier</b>	BH-3DF	301.11	294.34	6.77	B/R cored 98%, 100%
	BH-18DF	300.76	295.09	5.67	auger refusal
	C-3DF	300.64	294.95	5.69	CPT refusal



Table 1-2. Bedrock and Refusal Depths and Elevations at Boreholes					
Location	Borehole	Orig. Gr. Elev. (m)	Bedrock or Refusal Elev (m)	Overburden Thickness (m)	Comment RQD (by run)
WP5 Pier	BH-7NF	310.51	299.14	11.37	B/R cored 65%, 65%, 78%
	BH-8NF	312.21	298.4	13.81	auger refusal
WP6 North Abutment	BH-9NF	312.8	298.11	14.69	auger refusal
	BH-10NF	312.93	298.18	14.75	B/R cored 82%, 93%

## 1.5 Groundwater Conditions

Information regarding the groundwater levels at the site was obtained by measuring the water levels in the open boreholes after completion of drilling and by tactile examination of the recovered samples. The measured or inferred groundwater levels are shown on the borehole logs and the stratigraphic sections. In general, the groundwater table at the times of the field work was between about 1 m and 4 m in depth. It appears to follow the topography and this suggests that local subsurface drainage would be towards Trout Creek.

## Part 2 Engineering Discussion and Recommendations

### 2.1 Introduction

The following subsection addresses the geotechnical design considerations pertaining to the proposed five span bridge for the Northbound Lanes crossing of Trout Creek, as well as the approaches.

An original investigation was completed at this site in June and July of 1998 for an originally proposed 105 m long ( $\pm$ ) three span structure and revealed significant clay deposits on either side of Trout Creek which caused concern for the stability of the approach embankments, which were about 20 m high. Various design alternatives were considered including removal of the clay to the underlying bedrock, extensive berming of the embankments, the use of lightweight fill within the embankments, retaining walls, and a lengthening of the structure to limit the impact of the clay deposits. A subsequent cost benefit analysis of the design alternatives proposed by Trow, performed by Marshall Macklin Monaghan, indicated the preferred alternative was to lengthen the structure.

It was considered that the lengthened structure would effectively span the clay deposit, so that the approach embankments could either be located on a thinner or absent clay layer, or alternatively the lengthened structure would extend to the point where the height of the approach embankments could be reduced, and thus reduce or eliminate the complications encountered by placing high approach embankments on thick clay deposits.

Upon MTO acceptance of the lengthened structure alternative, a supplementary investigation was completed during September and November, 1998 as described in Part 1 of this report. The supplementary investigation examined the soil conditions at the revised pier and abutment locations of the new proposed five span structure, where conditions were not previously investigated as part of the original work.

The five span bridge is proposed to carry northbound Highway 11 traffic over Trout Creek and its valley. It is Trow's understanding that the bridge will be located with the central span crossing Trout Creek. The structure will include an approximately 11 metre high south abutment, 22 metre high centre span, and 5 metre high north abutment. The span lengths will be approximately 40, 54, 54, 54, 40 metres in length, creating a bridge with a length of about 242 metres.

It is understood that the project is scheduled for completion within a two year time frame. Accordingly, some of the discussions and design recommendations presented herein are based on this time frame.

## **2.2 Foundations**

In general, because of the presence of loose to compact sand and silt and relatively weak and compressible clay at the locations of the foundation elements, it is not considered feasible to support the proposed bridge foundations on spread footings constructed on the native mineral soils. For all foundation elements, driven steel H-piles are considered to be the preferred alternative. Alternate types of foundations may, however, be considered for the support of the bridge piers and abutments. The alternate types that are considered applicable to the site and proposed layout include large diameter reinforced concrete caissons and spread footings on rock or structural fill. Not all of these foundation types are applicable to all six foundation elements.

The following sections present the foundation design recommendations for the six foundation elements of the proposed bridge.

### **2.2.1 Steel H-Piles (all locations)**

All abutments and piers are recommended to be supported on steel H-piles driven to the bedrock surface, using the ULS capacities for HP310x110 and HP310x132 sections, as given in Table 2-1, below.

Current design practice requires that consideration be given to downdrag loads that may be generated as a result of soil consolidation due to fills, groundwater table fluctuations and the soil stresses induced by pile installation. We consider that, even after primary consolidation of the clay soils due to the fill placement is complete, the potential exists for downdrag loads due to events such as significant groundwater lowering and the long term process of secondary compression. At this site, secondary compression is the likely mechanism. Secondary compression of the clay foundation soils is discussed further in a subsequent section of this report. Accordingly, we have considered the effects of downdrag loads on the pile capacities given in the following sections, where applicable.

<b>Table 2-1. H-Pile Design Pile Capacities (kN)</b>						
	<b>HP 310x110</b>			<b>HP 310x132</b>		
Factored Structural Capacity (OHBDC)	3285			3890		
Factored Axial Resistance (MTO*)	2000			2300 (est)		
Pile Location ----->	South Abutment W1	Piers W2, W3, W4, W5	North Abutment W6	South Abutment W1	Piers W2, W3, W4, W5	North Abutment W6
Factored Downdrag Load	1000	-	1000	1150	-	1150
<b>Factored Axial Capacity at ULS**</b>	<b>2000</b>	<b>2000</b>	<b>2000</b>	<b>2300</b>	<b>2300</b>	<b>2300</b>
Notes: MTO* = Structural Office Policy Memo 98-01, April 15, 1998 ** Factored axial capacity at ULS is the lesser of: (a) factored structural capacity minus factored downdrag load, and (b) factored axial resistance SLS capacity not applicable to piles driven to bedrock						

The axial capacity at SLS is not applicable to steel piles driven to bedrock because very high loads, in excess of the ULS capacity, are required to produce unacceptable deformations. The pile structural capacity will govern.

The capacities given are axial capacities, for both vertical and inclined piles. No reduction is required for inclined piles. Notwithstanding the above, the horizontal and vertical loads acting on all piles must be resolved axially, such that the vector sum of these loads does not exceed the ULS capacity provided above.

For design purposes, the modulus of horizontal subgrade reaction for steel H-piles can be taken as 8,000 kN/m<sup>3</sup> for the cohesive soils (silty clays) and 40,000 kN/m<sup>3</sup> for the cohesionless soils. It is expected, however, that inclined piles will be required to accommodate the lateral loads. These can be designed using the same axial capacities given in Table 2-1.

A minimum soil embedment depth of 3 m below the pile cap is recommended. Pile caps should be provided with at least 2 m of soil cover for frost protection. Local grade raises may be required in order to provide this cover.

If the underside of the pile caps cannot be provided with a minimum of 2 m earth cover, insulation will be required. Insulation should consist of rigid board extruded polystyrene, meeting

CAN/CGSB-51.20-M87 (Type 4), such as *DOW SM™*. The insulation is recommended to be placed beneath the pile caps, prior to placement of concrete. Since the insulation will not carry any significant loads, high strength/low compressibility insulation (such as *DOW HI40™*, etc.) is not required. Products other than those made by *DOW CORNING* may be used, provided they meet the above noted specification.

The insulation thickness and lateral extension beyond the edges of the pile caps will depend on the depth of placement (i.e., underside of pile cap), in accordance with Table 2-2, below. A minimum soil cover of 300 mm is recommended over the top of the insulation.

Table 2-2. Pile Cap Insulation Dimensions		
Depth (mm)	Thickness (mm)	Lateral Extension (mm)
500	90	1500
1000	50	1000
1500	25	500

As discussed in following subsections of this report, substantial settlements of the approach fills will occur as will some associated lateral strains. Accordingly, piles should not be installed until after the majority of the fill settlement has occurred, based on monitoring during construction. This is particularly important for inclined piles, that would be subjected to bending stresses.

All piles should be driven to bedrock. Pile tip elevations can be estimated from Table 1-2 which provides the bedrock or refusal elevations encountered at the boreholes drilled at the various foundation elements. The boreholes indicate that the bedrock elevations are quite erratic and the potential for irregular steeply sloping bedrock at the foundation locations is considered to be high at most locations. As such, problems may arise during pile seating. At some locations, the piles may have a tendency to skip over the bedrock surface resulting in alignment problems and deeper penetration. In the event that this problem occurs, somewhat longer piles may be required and, in some cases, piles may have to be added or replaced.

To minimize seating difficulties, rock injector points should be used to facilitate proper seating. All piles must be fitted with reinforcing plates welded to the flanges as per OPSD 3301 to minimize pile damage. It is recommended that, during pile driving and upon initial contact with the bedrock, the

pile driving energy should be reduced and subsequently increased incrementally until the piles have been sufficiently seated.

## 2.2.2 Concrete Caissons (WP1 - South Abutment)

As an alternative foundation system, concrete caissons installed on or into the bedrock can be considered for all locations. However, they will likely only be practical for the foundation at the south abutment (WP1), where the bedrock surface is generally within about 3.5 m to 6.5 m below original grade. The load capacity will be derived by end bearing, in accordance with the values given in Table 2-3. As for steel H-piles, the effects of downdrag loads must be considered.

Table 2-3. Concrete Caisson Design Capacities	
Factored Downdrag Load (WP1 abutment)*	2800 kN/m pile diameter
Factored Axial Capacity at ULS	8000 kPa
Notes: SLS capacity not applicable to caissons on bedrock * Factored Downdrag Load to be applied to the factored dead loads	

In order to provide an adequate socket, the caisson should be installed at least one pile diameter into the bedrock, or be heavily dowelled. While these units can provide high capacities, because of the irregular and potentially steeply sloping bedrock surface expected at this site, caisson installation may prove difficult, thus expensive. This is complicated by the fact that in most cases the bedrock is overlain by silty sand and gravel that may create dewatering and stability problems during work at the base of the caissons.

## 2.2.3 Spread Footings (WP1 - South Abutment)

An alternate foundation for the south abutment is a conventional abutment on a spread footing. This alternative is not considered practical for the other locations because of the low capacities that will result from SLS considerations when constructed on soil or the need to excavate greater than 6 m for construction on bedrock. At the south abutment, a spread footing abutment can be constructed either on the bedrock or on structural fill placed over the bedrock or over the native soil.

### 2.2.3.1 Spread Footing on Bedrock

The elevations of a footing on bedrock can be estimated from Table 1-2. The factored bearing resistance at ULS for footings on unweathered bedrock is **8,000 kPa**. The bearing resistance at SLS does not apply because of the much higher pressures required to produce unacceptable deformations.

In order to evaluate the sliding resistance of spread footings on bedrock, the unfactored coefficients of friction for mass concrete on clean bedrock can be taken as 0.7. If the factored resistance against sliding failure is inadequate based on friction only, steel dowels will be required for footings on bedrock.

The ULS capacity of spread footings must be reduced for the effects of inclined loads. The reduction factors given in Table 2-4, below, can be used for footings on bedrock. Interpolation is possible. These factors must be applied to the ULS bearing resistance given previously.

<b>Table 2-4. ULS Reduction Factors for Inclined Loads on Spread Footings</b>	
<b>Ratio of Horizontal to Vertical Load</b>	<b>Footings on Rock</b>
0.1	0.86
0.2	0.76
0.3	0.66
0.4	0.58
The ULS reduction factors for inclined loads have been taken from Figure 6-8.4.2 of the OHBDC	

### 2.2.3.2 Spread Footing on Structural Fill

Spread footings can be designed for construction on structural fill. Structural fill can be constructed after removal of the overburden soils, where shallow, or it can consist of the granular (or rockfill) approach fill placed on the stripped native soils. For the abutment support, it is recommended that

the structural fill consist of OPSS Granular A, placed in small lifts and adequately compacted (100% standard Proctor). Alternatively, a relatively fine well graded rockfill, with a maximum size of 300 mm can be used. This finer graded rockfill should be placed in lifts limited to about 500 mm and adequately compacted with heavy vibratory rollers (minimum 6 passes, 10 tonnes).

At the south abutment, a spread footing abutment foundation, if considered, should be designed for construction in the approach fill, with a depth of about 2 m below the slope face (to provide adequate frost cover). This would place the base of the footing at a distance of about 6 m above original ground, at an elevation of about 320 m.

If the native soils are left in place, the structural fill supporting the foundation should have a thickness at least equal to the width of the footing. In addition, the structural fill should be constructed to occupy a zone, down and out from the footing edges at a slope of no steeper than 1H:2V, in order to accommodate the footing stresses.

For a foundation constructed on structural fill, as discussed above, the ULS bearing resistance given in Table 2-5, below, can be used.

<b>Table 2-5. Spread Footing ULS and SLS Bearing Resistance</b>	
Factored Bearing Resistance at ULS on Unweathered Bedrock <sup>1</sup>	8000 kPa
Factored Bearing Resistance at ULS on Structural Fill <sup>2</sup>	1000 kPa
Bearing Resistance at SLS - for 25 mm settlement - Fill on native soil <sup>3</sup>	See Figure A5
Bearing Resistance at SLS - for 25 mm settlement - Fill on bedrock <sup>4</sup>	> 1000 kPa
Notes: 1. SLS resistance not applicable to footings on bedrock 2. Thickness of fill greater than footing width. 3. 25 mm settlement is combined immediate granular settlement and primary consolidation settlement of clay soil. Structural fill placed on native soils after removal of organics/topsoil, and bottom of footing about 6 m above original ground. 4. Structural fill placed on bedrock after removal of all native soil, and bottom of footing about 6 m above original ground.	

The bearing resistance at SLS, however, will depend on whether the fill is placed on the bedrock after the removal of the native soil, or if it is placed over the native soil after stripping. If the native soil



is removed and replaced with granular structural fill, the SLS bearing resistance for 25 mm immediate elastic settlement is greater than at ULS. Accordingly, the ULS resistance governs the design.

If the native soil is left in place, the resulting settlements will be due to a combination immediate elastic settlement of the fill as well as consolidation settlement of the underlying cohesive soil. Figure A5 shows the footing bearing resistance at SLS for various footing sizes, based on 25 mm combined settlement.

Because of the consolidation settlement that will occur due to the loading by the embankment itself, footing construction should be delayed until the majority of settlement is complete, as discussed in a subsequent section of this report.

It is important to note that the stratigraphic cross section at the south abutment (WP1) shows silty clay as not having been encountered in BH-2NF or AP-4NF (the east end of the abutment). Accordingly, differential consolidation settlement of a large spread footing constructed over this zone will likely result in rotation or tilting. The magnitudes will likely be in the same order as the total deflections on which the SLS bearing resistance curves shown on Figure A5 are based.

Secondary compression of the silty clay soils will also occur, primarily due to the embankment fill loading, as discussed later in Section 2.5.4. Up to about 20 mm of associated settlement may occur within 10 years following construction, and this magnitude may be differential as discussed in the previous paragraph. Additional minor related settlements will occur beyond the 10 year period.

For the determination of the sliding resistance of spread footings, the unfactored coefficient of friction for mass concrete on granular structural fill can be taken as 0.6. If the factored resistance against sliding failure is inadequate based on friction only, a soil key can be considered for footings on structural fill, making use of the passive soil resistance. Passive earth pressure coefficients are provided in Section 2.3.

The ULS resistance of spread footings must be reduced for the effects of inclined loads. For footings on granular structural fill, Figure 6-8.4.2 of the OHBDC may be used for the applicable footing depth to effective width ratio. These factors must be applied to the ULS resistance given previously in Table 2-5.

## 2.3 Backfill

Backfill for abutments or retaining walls should consist of free draining granular materials such as Granular A, Granular B, or rock fill. Computation of earth pressures shall be in accordance with Section 6.7.4 of the Ontario Highway Bridge Design Code. Unfactored properties for backfill materials are provided in the following Table 2-6.

<b>Table 2-6. Fill Types and Unfactored Geotechnical Properties</b>					
<b>Material</b>	<b>Friction Angle, <math>\phi'</math></b>	<b><math>\gamma</math> (kN/m<sup>3</sup>)</b>	<b><math>K_A</math></b>	<b><math>K_P</math></b>	<b><math>K_0</math></b>
Granular A	35 degrees	22	0.27	3.7	0.43
Granular B	30 degrees	21	0.33	3	0.5
Rock Fill	42 degrees	20	0.2	5	0.33
Note: Values given for $K_A$ and $K_P$ are for horizontal backfill, and will vary for other configurations. $K_A$ is the earth pressure coefficient corresponding to the active state. $K_P$ is the earth pressure coefficient corresponding to the passive state. $K_0$ is the earth pressure coefficient at rest.					

The mobilization of full active or passive resistance requires a measurable and perhaps significant wall movement or rotation, in general accordance with that indicated in Figure C6-7.1 of the OHBDC. Therefore, unless the structural element can tolerate these deflections, the at rest earth pressure should be used in design. Alternatively, partially mobilized active and passive earth pressure coefficients may be used, in accordance with the deflections given in the Figure.

The effects of compaction surcharge should be taken into account in the calculation of active and at rest earth pressures. In accordance with Section 6-7.4.3 of the OHBDC, the lateral pressure due to compaction should be taken as at least 16 kPa at the surface, and its magnitude should be assumed to diminish linearly with depth to zero at the depth where the active (or at rest) pressure is equal to 16 kPa. This pressure distribution should be added to the calculated active (or at rest) pressure. Notwithstanding, lighter compaction equipment and smaller lifts should be used adjacent to walls to prevent overstressing.

If rock fill is used as backfill behind abutments, the particle size should be limited to no greater than 300 mm and the backfill must be placed carefully in a manner that does not cause damage to the abutments or other structural components of the bridge.

Weep holes should be provided in the abutments and a filter wrapped, 150 mm diameter perforated PVC drainage pipe should be installed behind the abutments, at or slightly above the base.

## **2.4 Excavations and Dewatering**

All work associated with design and construction relative to excavations shall be in accordance with Part III of Ont. Reg. 213/91 of the Occupational Health and Safety Act. Where the width of the base of the excavation is less than twice its depth, conformance with this regulation is required.

The existing organic zone (topsoil) with a depth ranging to about 200 mm will have to be removed from beneath the approach fill footprints. For pile caps at the pier locations, excavations of at least 2 m depth will likely be required, in order to provide the recommended frost cover. Since the ground surface is uneven at the pier locations, deeper excavations will be required to place the pile cap at a common elevation, unless the finished site grades are raised with fill or the caps are provided with insulation. Based on the borehole information, excavations would generally proceed through the loose to compact upper sand and into the stiff to firm silty clay at most locations.

At the locations of Piers WP2, WP3 and WP5, excavations should be relatively straight forward, since they will likely terminate above the prevailing groundwater level. The upper sand and silty clay is considered a Type 3 soil and excavations should be cut back to at least 1H:1V. If minor groundwater seepage occurs and loosens/softens the soil, flatter slopes will be required. Dewatering of the excavations should be possible by pumping from sumps within the excavations.

At the location of Pier WP4, on the north side of Trout Creek, the excavation will likely be carried out entirely within the loose to compact silty sand and would terminate close to or below the prevailing water level of Trout Creek. These soils would then be classified as Type 4 soils and excavations should be sloped back at 3H:1V. Pumping from sumps within the excavation should suffice here as well, in order to maintain a safe and workable area, although more aggressive effort will likely be required. In order to stabilize the base of the excavation if it becomes loosened due to groundwater infiltration, a 300 mm layer of crushed clear stone may be required to improve working conditions.

If the native soils are to be removed and replaced with structural fill at the south abutment, or to place foundations on the bedrock, excavations as deep as about 6.5 m will be required, based on the results of the investigation. The soils to be excavated will consist of sand, silty clay and silty sand and gravel. The water table would be encountered well above the 6.5 m depth. Accordingly, excavations in this area should be sloped back at 3H:1V, or flatter. Aggressive pumping from sumps will be required. Alternatively, sheeted and braced excavations could be considered, but this may prove problematic because of the variable bedrock surface.

Excavations carried out within granular structural fill in the approaches can likely be completed using a 1H:1V cut since it will be above the water table.

It is recommended that a non-standard special provision (NSSP) for dewatering be provided in the contract documents.

## **2.5 Bridge Approach Fills**

The construction of the bridge approaches will require embankment fills of up to about 11 m height at the south abutment and about 5 m height at the north abutment over areas with varying thicknesses of predominantly firm, compressible silty clay soils. This creates two principal design and construction considerations: embankment stability and consolidation settlement. These two issues are discussed in the following sections.

In all of the following discussions, it is assumed that all organic material (topsoil) is removed from beneath the embankments and the embankments are constructed on the native mineral soils. Fill heights should be measured from the top of the native mineral soil.

### **2.5.1 Embankment Stability**

Highway embankments can be constructed using structural fill of various acceptable soil materials. Typically, however, in this part of the province they are constructed using rockfill as the principal component. A typical MTO section will consist of a minimum (steepest) slope of 1.25H to 1V to a maximum height of 6 m. For embankment sections greater than 6 m height, a 2 m wide horizontal bench is required before the next 6 m, 1.25H to 1V lift is constructed. A 14 m crest width has also been assumed, based on the drawings provided.

Slope stability analyses have been performed using SLOPE/W (ver. 3.x), based on Bishop's Method using total stress parameters. This analysis would apply to rapid construction (short term stability) and factors of safety can be expected to increase with time. The undrained shear strength profile shown on Figure A3 was used to provide the shear strength parameters for the clay soils. Table 2-7, below, lists the parameters used.

<b>Table 2-7. Geotechnical Parameters for Slope Stability Analyses</b>			
	$\gamma_{\text{total}}$ (kN/m <sup>3</sup> )	$c_u$ (kPa)	$\phi'$
Rockfill	20	0	42°
Sand	20	0	32°
Silty clay	19.5	variable (see Fig. A3)	0
Sand and Gravel	21.5	0	35°
Notes: Embankment crest width 14 m.			

Many of the SLOPE/W graphical printouts, for the various analyses performed and discussed below, are included in Appendix D.

### 2.5.1.1 South Approach Stability

The results of the total stress analyses performed on the embankment cross-sections, for heights of 10 m to 13 m, are shown in Table 2-8, below. The short term factors of safety for the embankments assessed are marginally below the recommended value of 1.3. It is our opinion, however, that the factors of safety are likely greater than calculated, since some excess pore water pressure dissipation will occur during the assumed one month construction period. At this location, the silty clay layer is relatively thin (3 m to 4 m) and the results of the consolidation analyses, discussed later in Section 2.5.5.2, support this.

Notwithstanding, we have performed calculations for embankments with benches wider than the standard 2 m width at 6 m height. These required widths to provide a calculated factor of safety of at least 1.3 are shown in Table 2-8, also. Accordingly, it is considered prudent to provide the indicated berm widths to provide a calculated factor of safety of 1.3.

Since the recommended short term factors of safety are considered adequate, with only minor modifications to the geometry, effective stress analyses are not required and have not been performed.

**Table 2-8. Safety Factors for Total Stress Stability Analyses - South Approach**

Embankment Height	Factor of Safety*	Bench Width**
10 m	1.28	+ 0.5 m (2.5 m)
11 m	1.23	+ 2 m (4 m)
12 m	1.18	+ 3 m (5 m)
13 m	1.21	+ 4 m (6 m)

Notes: \* Factor of safety for embankment with standard 2 m wide bench at 6 m height  
\*\* Bench width required to provide safety factor >1.3. First number is width to add to standard 2 m wide lower bench.

An analysis was also performed on the longitudinal design section (front slope) as shown on Figure A2b and Drawing No. 2. The calculated factor of safety for this slope is 1.58, which is considered adequate.

#### **2.5.1.2 North Approach Stability**

The results of the total stress analyses performed on the embankment cross-sections, for heights of 4 m to 7 m, resulted in safety factors ranging from about 3.6 to 2.2, which are considered more than adequate.

An analysis was also performed on the longitudinal design section (front slope) as shown on Figure A2b and Drawing No. 2. The calculated factor of safety for this slope is 3.0, which is also considered adequate.

## 2.5.2 Consolidation Settlement of Embankments

### 2.5.2.1 Magnitudes of Consolidation Settlement

Consolidation settlement calculations have been performed using the effective stress profiles shown on Figure A3 and compression ratios ( $C_c' = C_c / [1 + e_0]$ ) ranging from 0.08 to 0.20. The values used were established from the consolidation test data, previous experience at the north and south interchanges of this project, as well as from geotechnical literature. Recompression indices ( $C_r' = C_r / [1 + e_0]$ ) ranging from about 0.008 to 0.02 were used. The stress increases in the foundation soils due to the embankment loadings have been calculated based on the Boussinesq elastic solutions. Recompression and virgin consolidation has been calculated based on the assumed existing overburden pressure and preconsolidation stress profiles shown on Figure A3.

#### South Approach

The results of the calculations for embankment centerline and crest settlement using rockfill to construct the embankments are provided in Table 2-9, below and are shown graphically on Figure A6. The thickness of the compressible silty clay soil ranges up to about 4 m. Because of the unevenness of the original ground surface at the approaches, the height of fill may vary slightly. Accordingly, the analyses have been performed taking this into account by using a varying fill height.

Table 2-9. Estimated Embankment Consolidation Settlement - South Approach		
Embankment Ht	Centerline Settlement (mm)	Crest Settlement (mm)
10 m	170	155
11 m	185	175
12 m	200	190
13 m	215	205
Notes: Embankment crest width 16 m, average side slopes = 1.4H:1V Values rounded to nearest 5 mm		

In addition, the loadings imposed by the fill will exceed the indicated preconsolidation pressure, such that the silty clay soils will be normally consolidated following the consolidation process. This will

result in greater settlements due to future additional loadings, such as footings as discussed in a previous section, or due to grade changes, for example. For these reasons, it may be prudent to overbuild the embankment by 1 or 2 m, allow consolidation to take place and subsequently remove the excess fill height (*i.e.* preloading). This will result in a slight overconsolidation of the foundation silty clay soils.

It is noted that, the compressible silty clay was not encountered everywhere beneath the embankment, as was discussed in Section 2.2.3 of this report. Accordingly, there may be greater differential settlement between crest and centerline, as well as along the length of the approach. The differential settlement may approach the total settlements indicated in the Table above.

Notwithstanding the potential for relatively large differential settlement, the time to facilitate this settlement should be relatively rapid (within about nine months), as discussed in Section 2.5.2.2, below. Therefore, minor regrading prior to providing the final granulars and pavement should be possible within the two year construction period that has been inferred.

### **North Approach**

The results of the calculations for embankment centerline and crest settlement using rockfill to construct the embankments are provided in Table 2-10, below and are shown graphically in the top panel of Figure A7. The thickness of the compressible silty clay soil ranges up to about 12 m. Because of the unevenness of the original ground surface at the approaches, the height of fill may vary slightly. Accordingly, the analyses have been performed taking this into account by using a varying fill height.

<b>Table 2-10. Estimated Embankment Consolidation Settlement - North Approach</b>		
<b>Embankment Ht</b>	<b>Centerline Settlement (mm)</b>	<b>Crest Settlement (mm)</b>
4 m	30	25
5 m	35	30
6 m	40	35
7 m	70	45
Notes: Embankment crest width 16 m, average side slopes = 1.4H:1V Values rounded to nearest 5 mm		



The loadings imposed by the fill will approach, and may exceed, the indicated preconsolidation pressure, such that the silty clay soils will be normally consolidated following the consolidation process due to the fill. Examination of the top panel of Figure A7 indicates that the settlement is expected to increase significantly as the embankment height approaches 6 m, where the settlement curve steepens. This is due to the preconsolidation pressure of the foundation soil being exceeded.

This will result in greater settlements due to any future additional loadings, such as grade changes, for example. Accordingly, it may be prudent to overbuild the embankment by 1 or 2 m, allow consolidation to take place and subsequently remove the excess fill height (*i.e.* preloading). The preload should be left in place for about a year, unless monitoring indicates that it can be removed sooner. This will result in a slight overconsolidation of the foundation silty clay soils.

If consideration is to be given to the use of light weight fill, for preliminary design purposes, the total consolidation settlements can be reduced **approximately** by proportion of rockfill replaced by lightweight polystyrene (eg. 30% replacement of fill height with polystyrene will result in about 70% of the indicated settlement).

### 2.5.2.2 Time Rate of Consolidation Settlement

The time rate of consolidation settlement has been calculated for the various cases of embankment height for vertical drainage only. It is also assumed for the purposes of calculation, all embankments are constructed to full height in about a one month construction period. Coefficients of consolidation (virgin),  $C_v$ , of 8 m<sup>2</sup>/year, and (recompression),  $C_{vr}$ , of 40 m<sup>2</sup>/year, have been used in the analyses, based on the results of the consolidation tests and the geotechnical literature.

The bottom panels of Figures A6 and A7 show the calculated consolidation rate for the various fill heights at the south and north approaches, respectively.

At the south approach, virtually all of the primary consolidation settlement should be complete within about 9 months. At the north approach, because of the thicker silty clay deposit, the time rate of consolidation will be slower, and should be complete anywhere between about 15 months to over 24 months, depending on the fill height.

### 2.5.3 Secondary Compression of Clays

Calculations have been performed to estimate the magnitudes of secondary compression of the foundation clays, following the completion of primary consolidation. For purposes of analysis and

discussion, the primary consolidation is assumed to be essentially complete within one to two years from the start of construction of the embankments. The calculations are based on use of a coefficient of secondary compression,  $C_{\alpha}$ , of 0.004, based on the results of the consolidation tests, previous experience, and the geotechnical literature.

At the south approach, calculations indicate that 15 mm to 20 mm may occur in the first 10 years, while at the north approach, the secondary settlement may be about 30 mm to 50 mm in the first 10 years.

### **2.5.5 Rockfill and Rockfill Settlement**

Rockfill used in the embankments and approaches should consist of hard, durable, angular quarried rock. Rockfill should be tested for acid drainage potential and checked against the appropriate regulatory requirements for acceptability.

For the bridge abutments that are to be supported by driven steel H-piles, as discussed in a previous section, these will have to be installed after all the embankment fill has been placed. The driving of steel piles through rockfill will be difficult, if not impossible, unless care is taken to ensure that the rockfill meets a tight size range, generally smaller than 75 mm. For the majority of the embankments and approach fills, however, larger rockfill can be used.

The use of large rockfill, loosely placed or end dumped and compacted by conventional construction traffic only may result in long term settlements of up to about 2% of the fill height. Accordingly, a 10 m high rockfill may settle up to 200 mm following completion of construction. This is primarily due to the rockfill "adjusting" under highway service conditions.

In order to minimize the long term settlements associated with the rockfill itself, consideration can be given to specifying the use of a finer graded rockfill, with a maximum size of 400 mm. This finer graded rockfill should be placed in lifts limited to about 800 mm and compacted with heavy vibratory rollers. While the production of the finer graded rockfill and its placement, as described above, carries with it a cost premium, the long term performance of the finished highway pavement will be improved.

It is recommended that an NSSP for rockfill material and placement requirements be included in the contract documents.

## **2.6 Instrumentation and Construction Monitoring**

Construction of embankments should be monitored during construction using a properly designed and effective system of instrumentation, consisting primarily of settlement points. Lateral slope movements should also be monitored both during and following construction of the embankments. This will provide indications of the rate of settlement, such that construction timing of the foundations can be modified, if required.

## **2.7 Closing Comments**

The recommendations made in this report are in accordance with our present understanding of the project and are provided solely for the use of Marshall Macklin Monaghan and its design team for the design of the bridge foundations and approach fills for the northbound lanes, five span bridge to be constructed over Trout Creek, as part of the Highway 11 Trout Creek By-Pass Project. We request that we be retained to review the design and our recommendations as the design proceeds, to ensure that the final design is in general agreement with our recommendations and that our recommendations have been interpreted as intended.

A subsurface investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those reported at the test locations, we require that we be notified immediately in order to allow reassessment of our recommendations. It may then be necessary to carry out additional field work and analyses.

Contractors bidding on or undertaking the works should, relative to the subsurface conditions, decide on their own investigations, if deemed necessary, as well as their own interpretations of the factual results provided herein, so they may draw their own conclusions as to how the subsurface conditions may affect them.

The information presented in this report is based on a limited investigation designed to provide information to support an overall assessment of the current geotechnical conditions at the site of the proposed northbound lanes bridge over Trout Creek. The conclusions presented in this report reflect site conditions existing at the time of the investigation. It is noted that the soil boundaries indicated on the logs are inferred from discontinuous sampling and observations during drilling. These boundaries are intended to reflect transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change.

This report has been prepared by Mr. D.N. Georgiou, P.Eng. and was reviewed by Messrs. S.E. Gonsalves, P.Eng., and E. Gonneau, P.Eng. The field investigation was performed by Messrs. I. Dumpis, C.E.T., R. Moore, C.E.T., and S. McAuliffe, and was supervised by Mr. E.A. Gonneau, P.Eng.

We trust this report is satisfactory for your purposes. Should you have any questions, please do not hesitate to contact us.

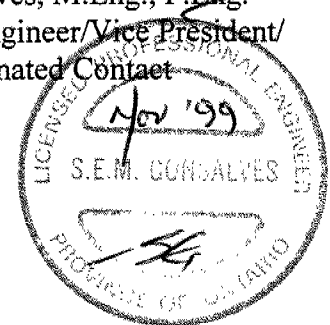

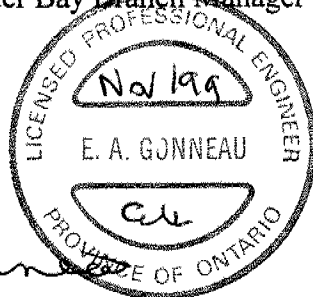
All the foregoing and attachments respectfully submitted,  
**Trow Consulting Engineers Ltd.**



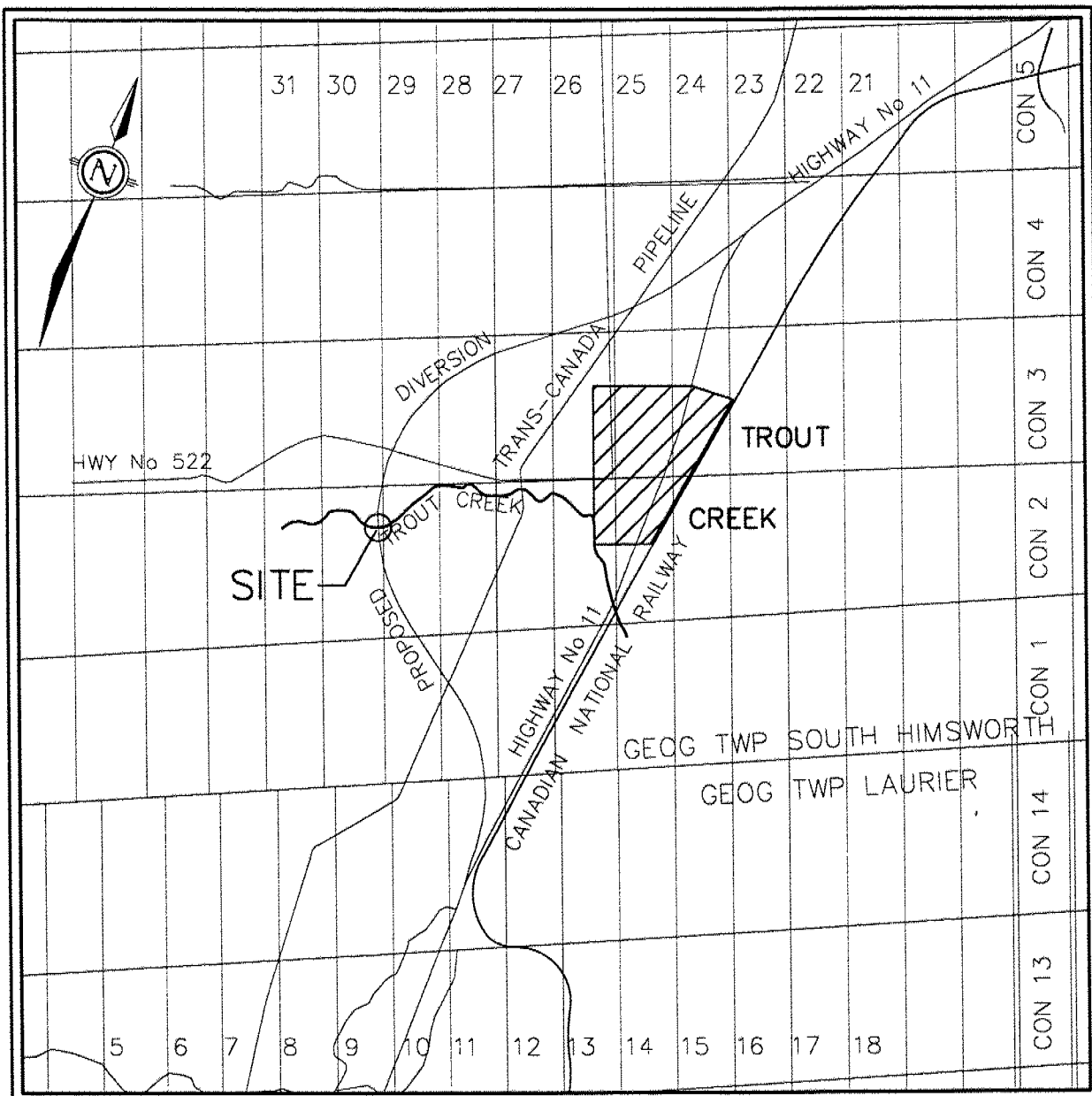

Demetri N. Georgiou, M.A.Sc., P.Eng.  
Principal Engineer/Thunder Bay Branch Manager



S.E. Gonsalves, M.Eng., P.Eng.  
Principal Engineer/Vice President/  
MTO Designated Contact


  
Eric A. Gonneau, P.Eng.  
Project Manager


A



500 0 500 1000  
IN METRES



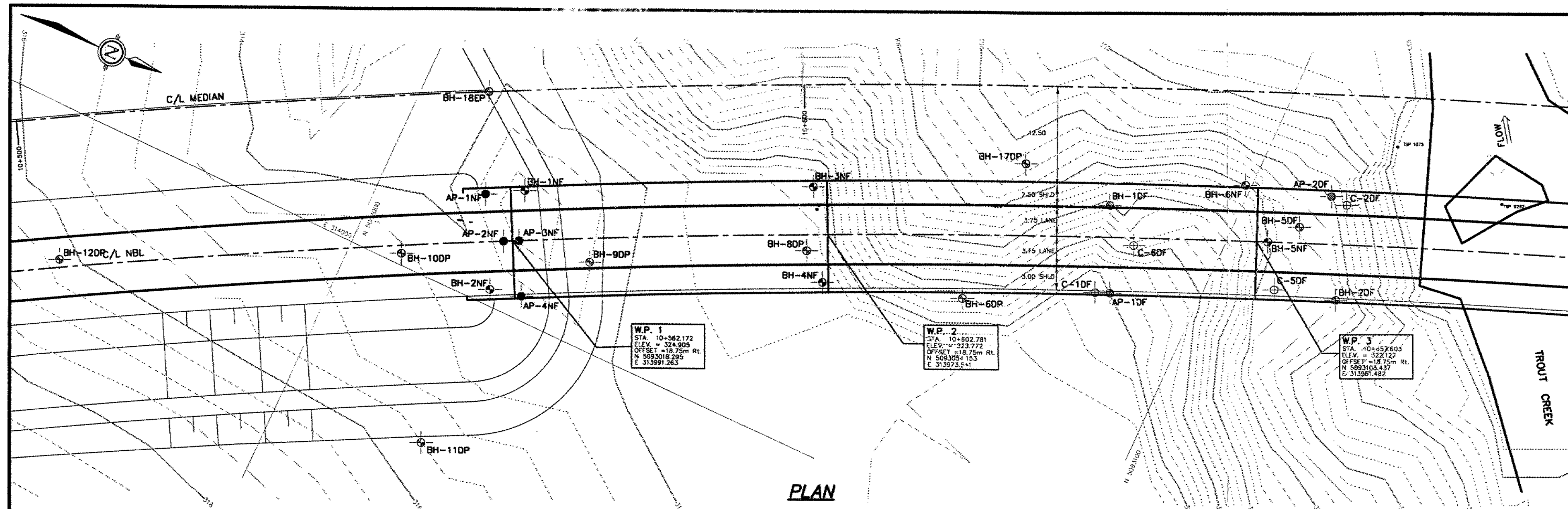
Trow Consulting Engineers Ltd.  
Thunder Bay, Ontario

FIGURE  
A1

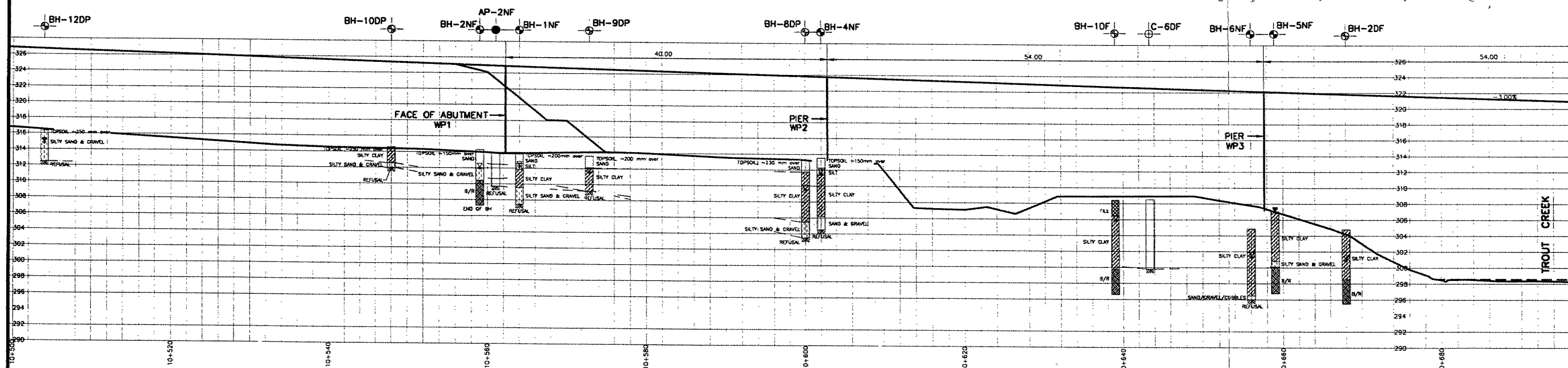
### KEY PLAN

Trout Creek By Pass  
Trout Creek Bridge-Northbound Lanes

PROJECT NO.:	F-98179-B/G
SCALE:	1:50000
DRAWN BY:	DT
CHECKED BY:	DG
DATE:	MARCH 12, 1999



**PLAN**



**PROFILE**

**LEGEND:**

- BOREHOLE (NOV/98)
- AUGER PROBE (NOV/98)
- ⊕ PREVIOUS BOREHOLE (JUN/98)
- ⊗ PREVIOUS AUGER PROBE (JUN/98)
- ⊕ DYNAMIC CONE PENETRATION TEST (JUN/98)
- ▽ ESTIMATED GROUND WATER LEVEL
- ▽ MEASURED GROUND WATER LEVEL

5 0 5 10  
IN METRES

**NOTE:**

The stratigraphic boundaries and soil types have been established at Test Hole locations. Between Test Holes they are assumed and may be subject to error.

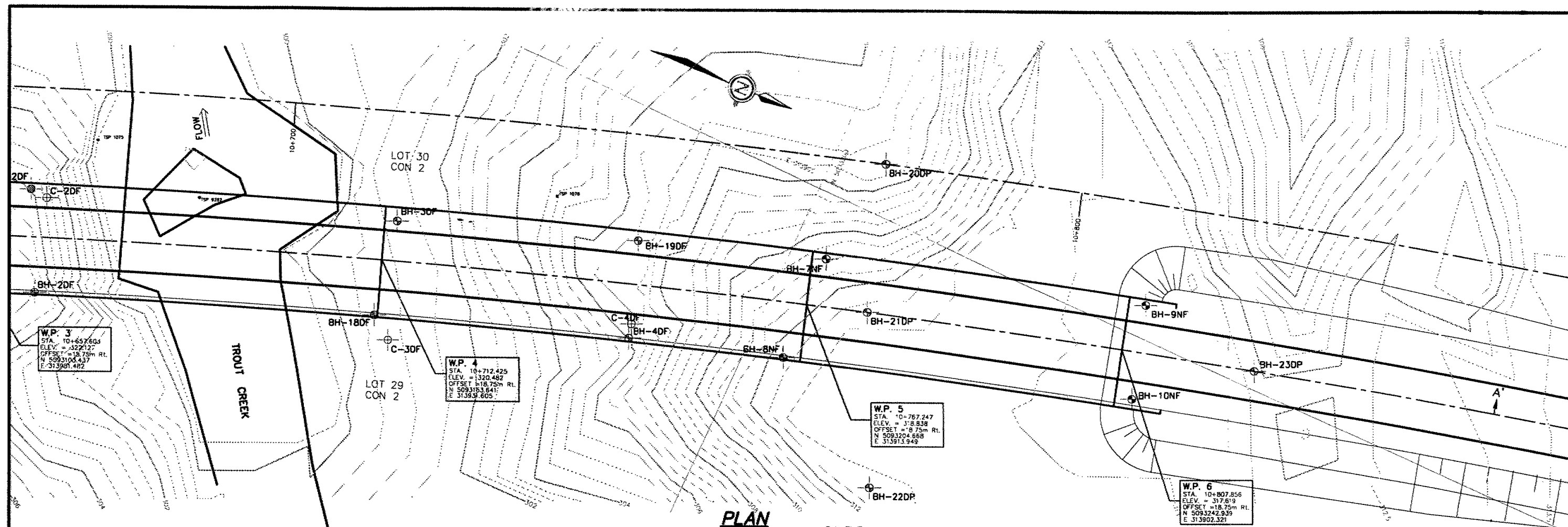


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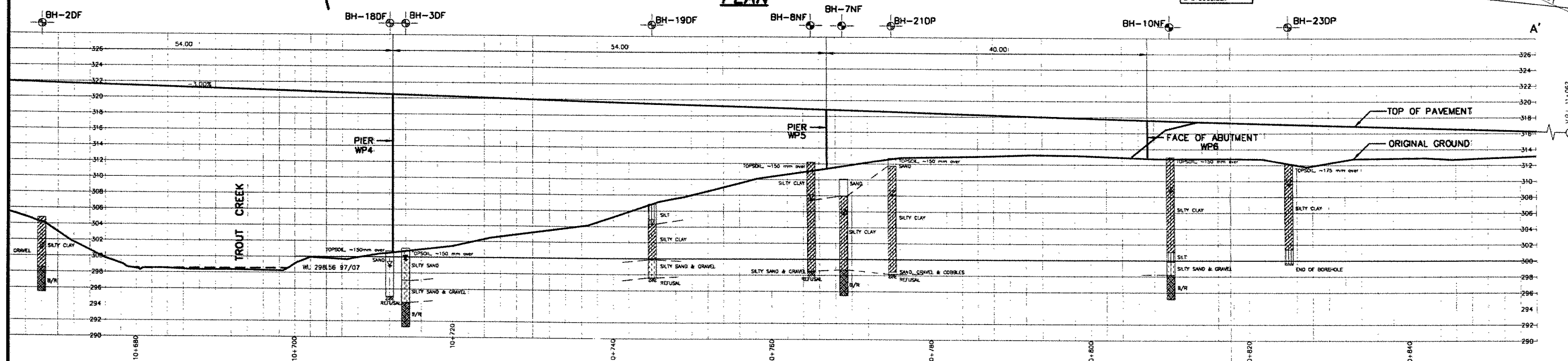
FIGURE  
A2a

**BOREHOLE LOCATION PLAN  
& PROFILE (SOUTH)**  
Trout Creek By Pass  
Trout Creek Bridge-Northbound Lanes

PROJECT NO.: F-98179-B/G  
SCALE: 1:500  
DRAWN BY: DT  
CHECKED BY: DG  
DATE: NOV. 25, 1999



**PLAN**



**PROFILE**

**LEGEND:**

- BOREHOLE (NOV/98)
- AUGER PROBE (NOV/98)
- ⊕ PREVIOUS BOREHOLE (JUN/98)
- ⊕ PREVIOUS AUGER PROBE (JUN/98)
- ⊕ DYNAMIC CONE PENETRATION TEST (JUN/98)
- ⊕ ESTIMATED GROUND WATER LEVEL
- ⊕ MEASURED GROUND WATER LEVEL

**NOTE:**

The stratigraphic boundaries and soil types have been established at Test Hole locations. Between Test Holes they are assumed and may be subject to error.



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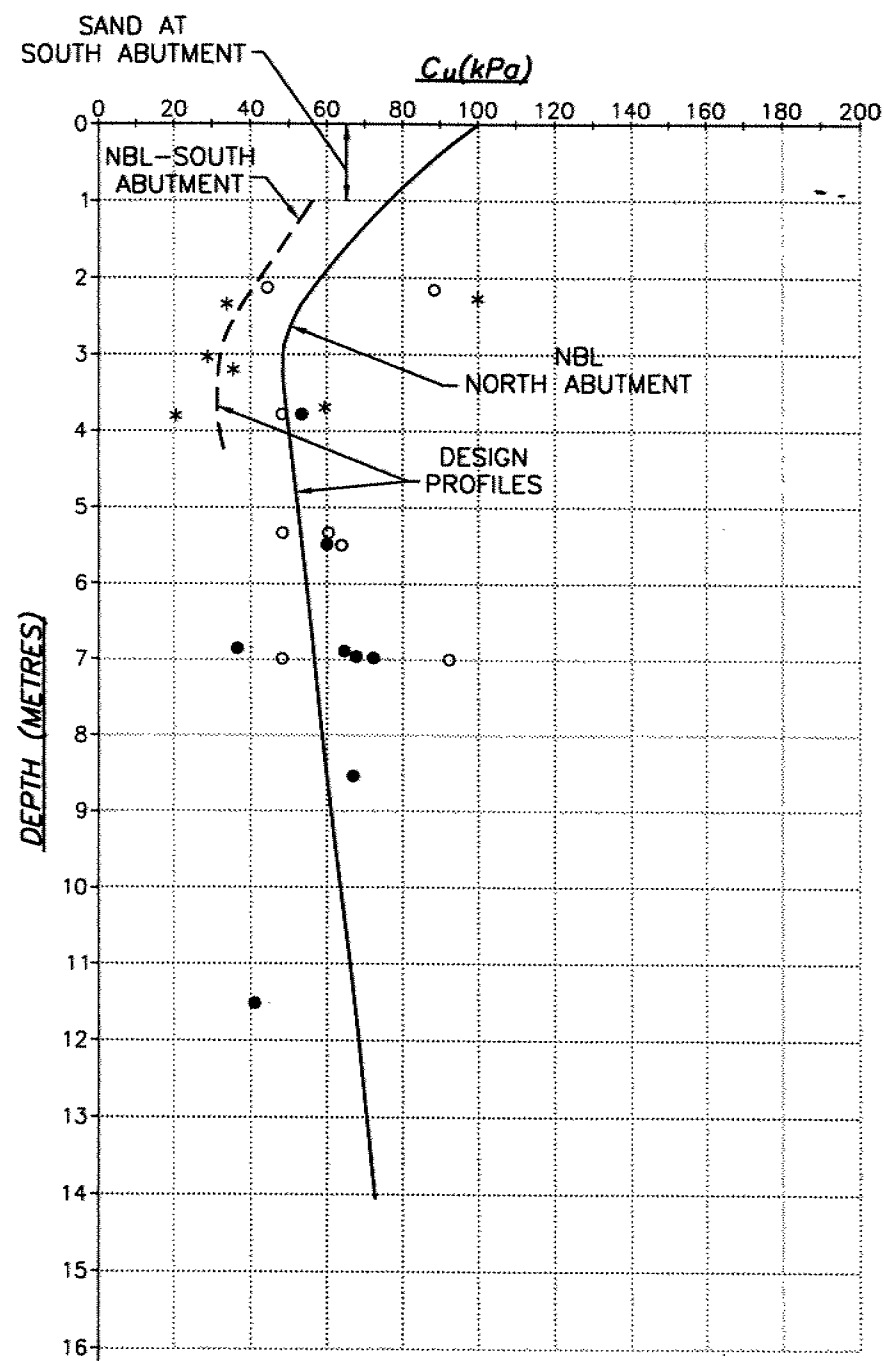
**FIGURE A2b**

**BOREHOLE LOCATION PLAN & PROFILE (NORTH)**  
Trout Creek By Pass  
Trout Creek Bridge-Northbound Lanes

PROJECT NO.:	F-98179-B/G
SCALE:	1:500
DRAWN BY:	DT
CHECKED BY:	DG
DATE:	NOV. 25, 1999

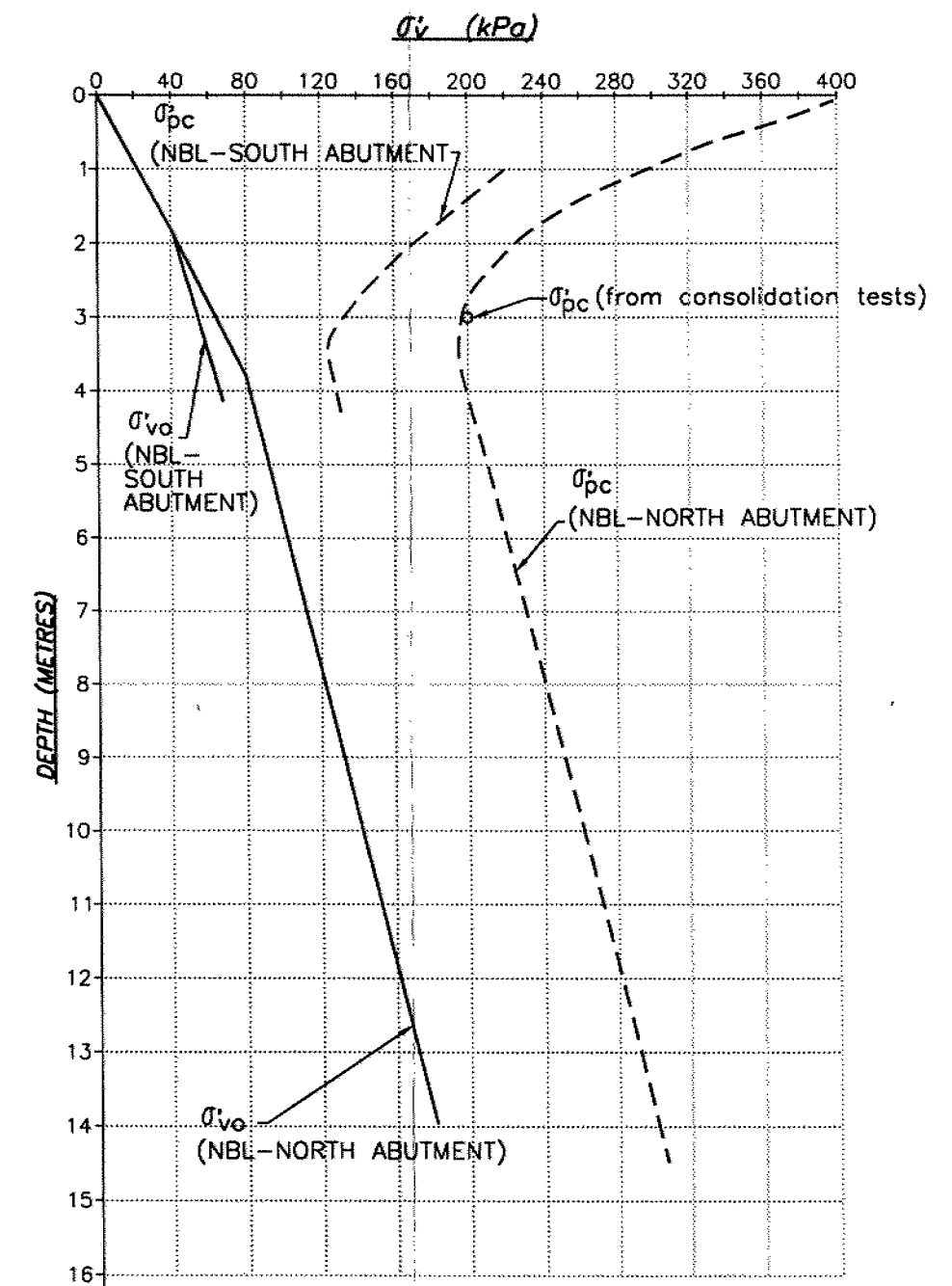
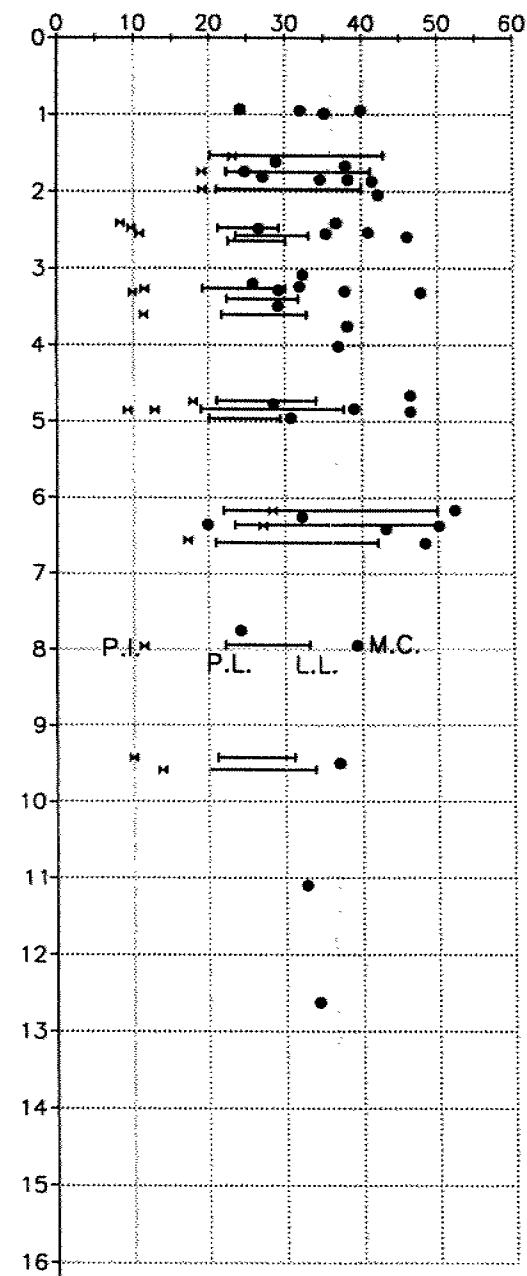






\*NBL - SOUTH ABUTMENT  
 ●NBL - NORTH ABUTMENT  
 ○SBL - NORTH ABUTMENT

### MOISTURE CONTENT & ATTERBERG LIMITS

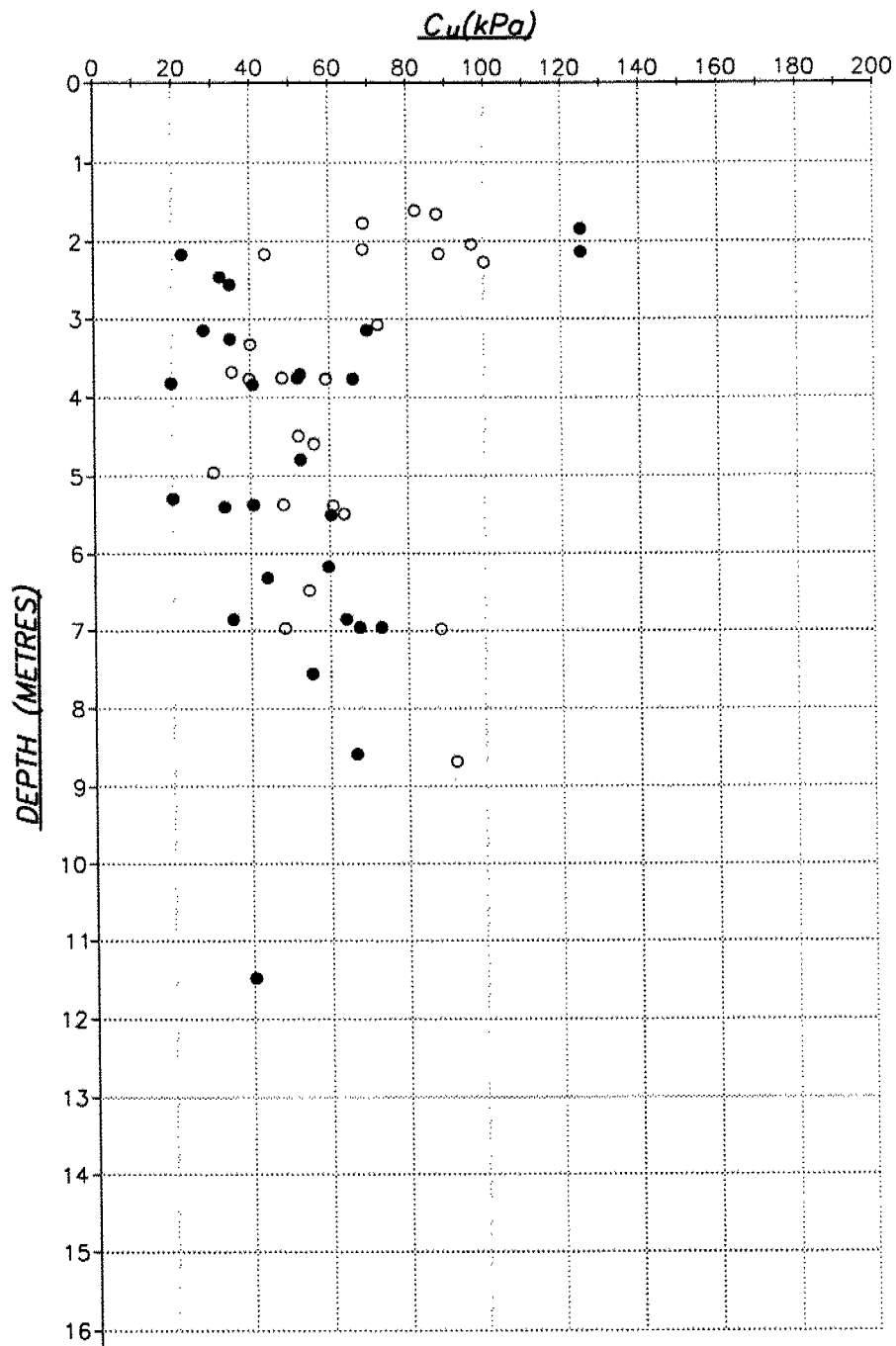


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FIGURE  
 A3

UNDRAINED SHEAR STRENGTH,  
 ATTERBERG LIMITS &  
 EFFECTIVE STRESS PROFILES  
 Trout Creek By Pass  
 Trout Creek Bridge-Northbound Lanes

PROJECT NO.: F-98179-B/G  
 SCALE: AS SHOWN  
 DRAWN BY: DT  
 CHECKED BY: DG  
 DATE: MARCH 12, 1999



LEGEND:

- SHEAR STRENGTHS (NBL)
- SHEAR STRENGTHS (SBL)



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Thunder Bay, Ontario

FIGURE  
A4

UNDRAINED SHEAR STRENGTH  
ALL BOREHOLES

Trout Creek By Pass  
Trout Creek Bridge—Northbound Lanes

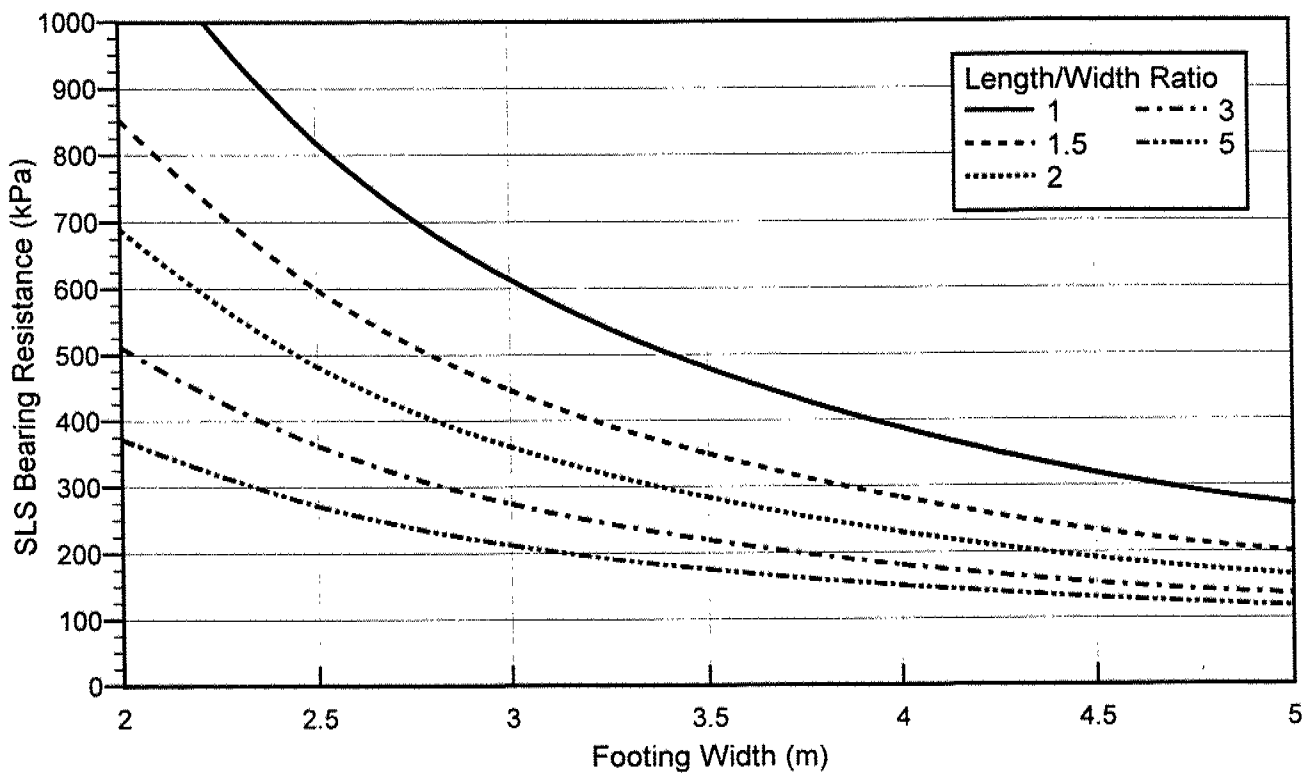
PROJECT NO.: F-98179-B/G

SCALE: AS SHOWN

DRAWN BY: DT

CHECKED BY: DG

DATE: MARCH 12, 1999



Combined Immediate and  
Consolidation Settlement = 25 mm  
Bottom of footing at 6 m above O/G



Trow Consulting Engineers Ltd.  
Thunder Bay, Ontario

Marshall Macklin Monaghan

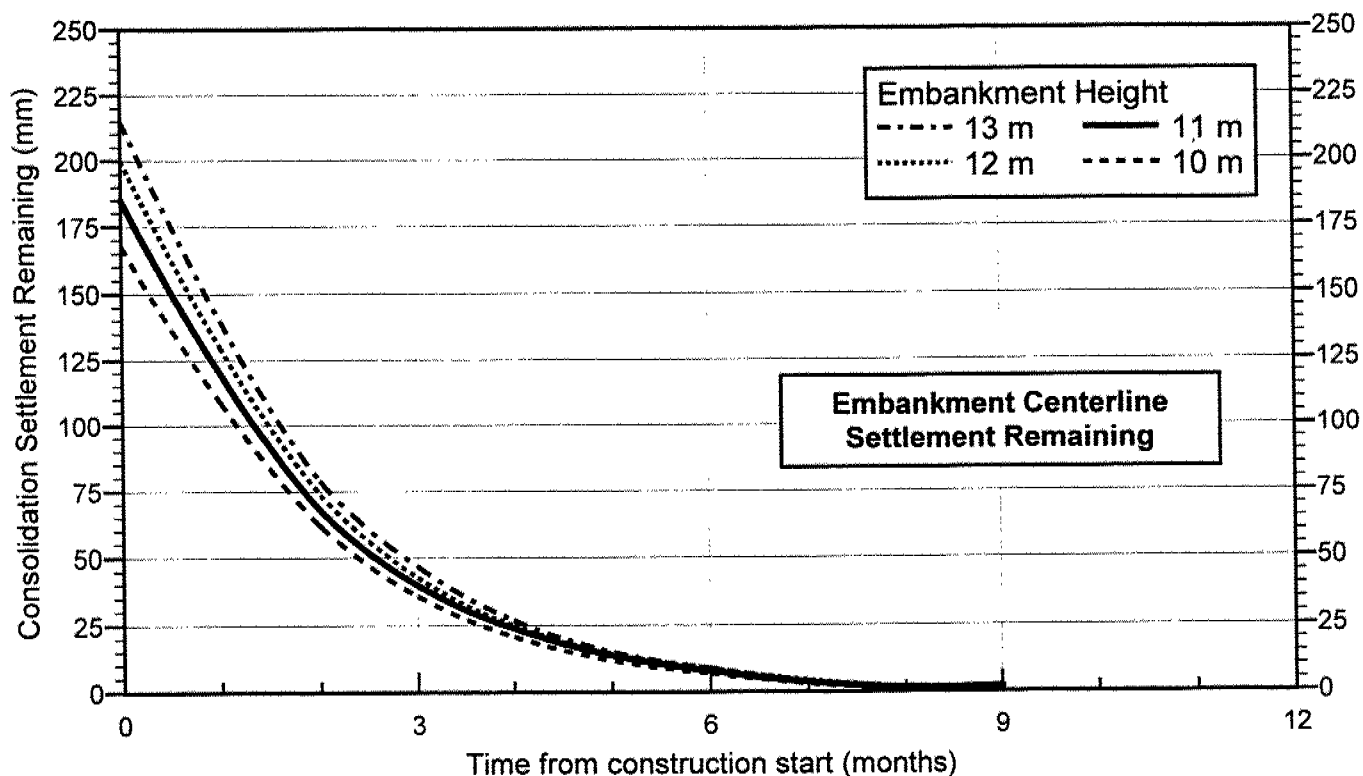
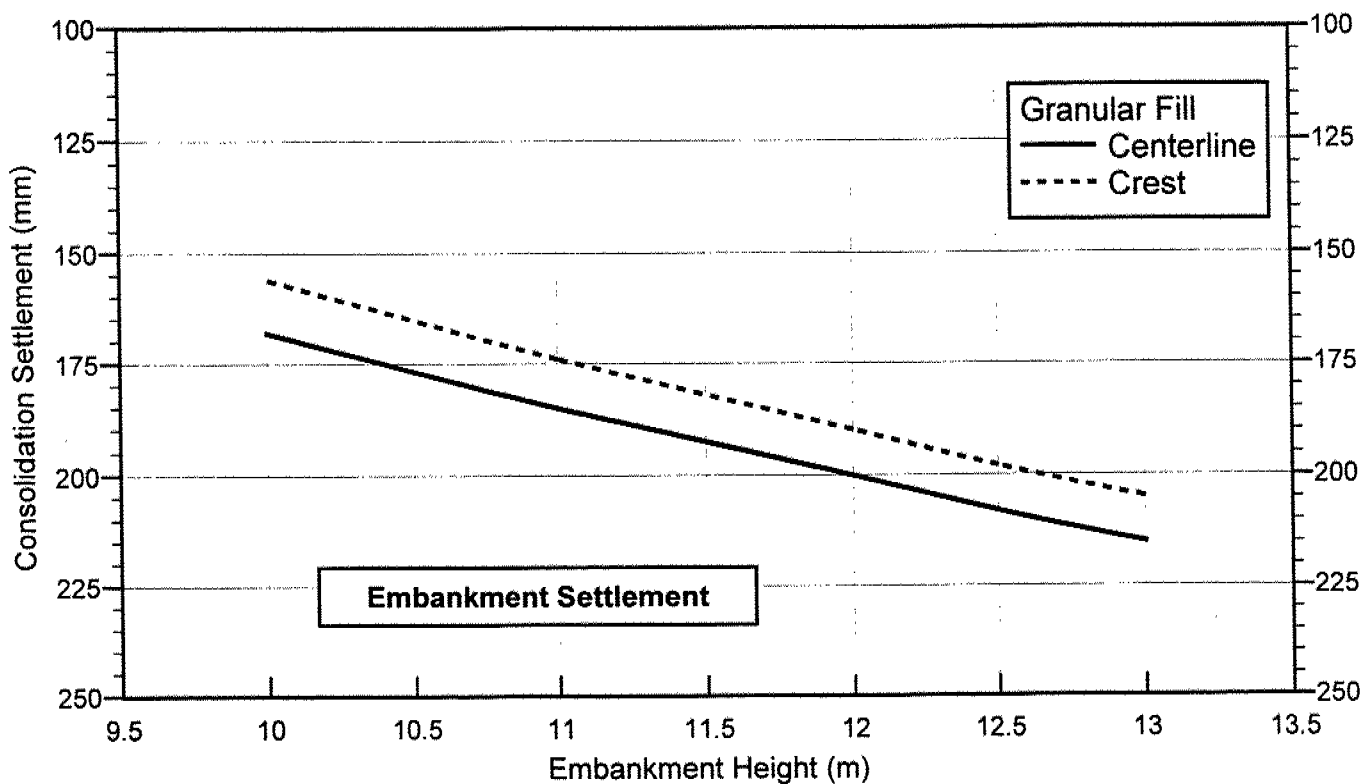
Footing SLS Bearing Resistance  
Granular Fill on Native Soil - SOUTH APPROACH

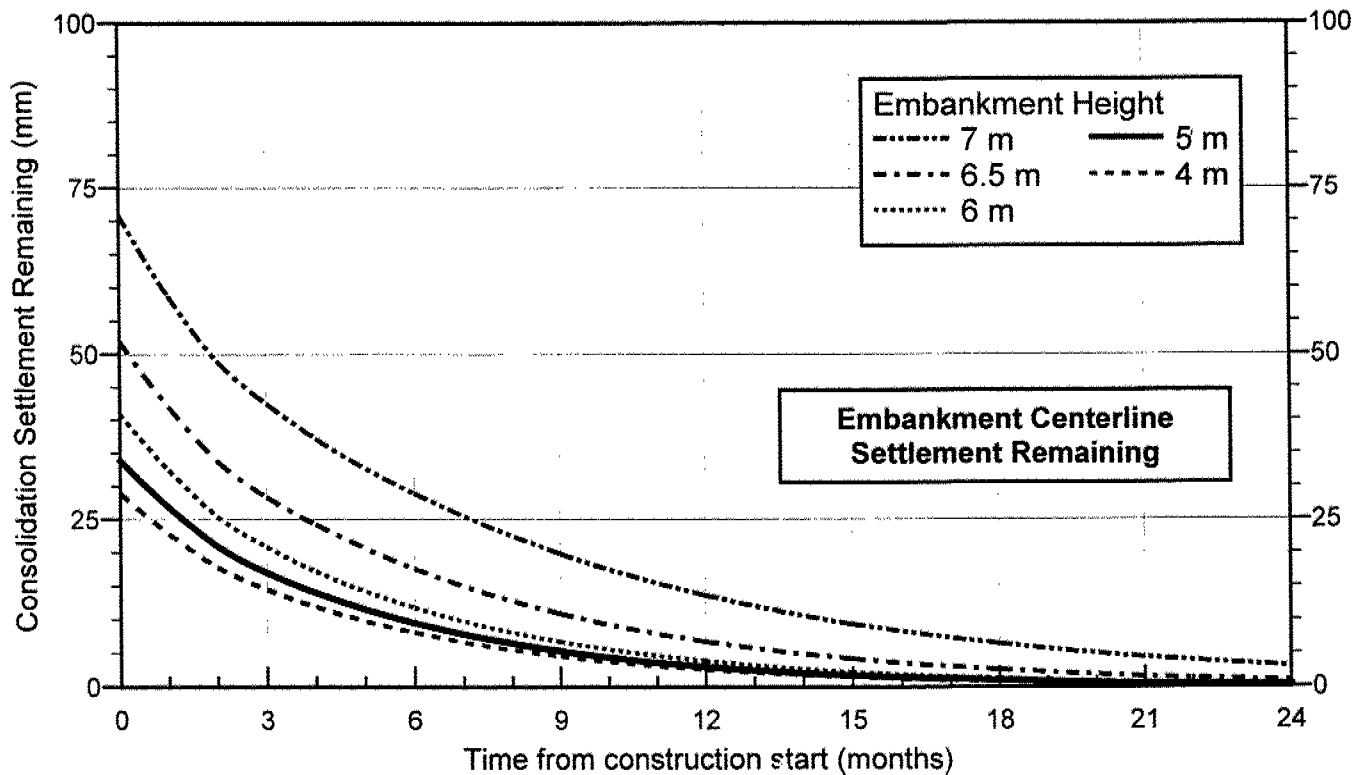
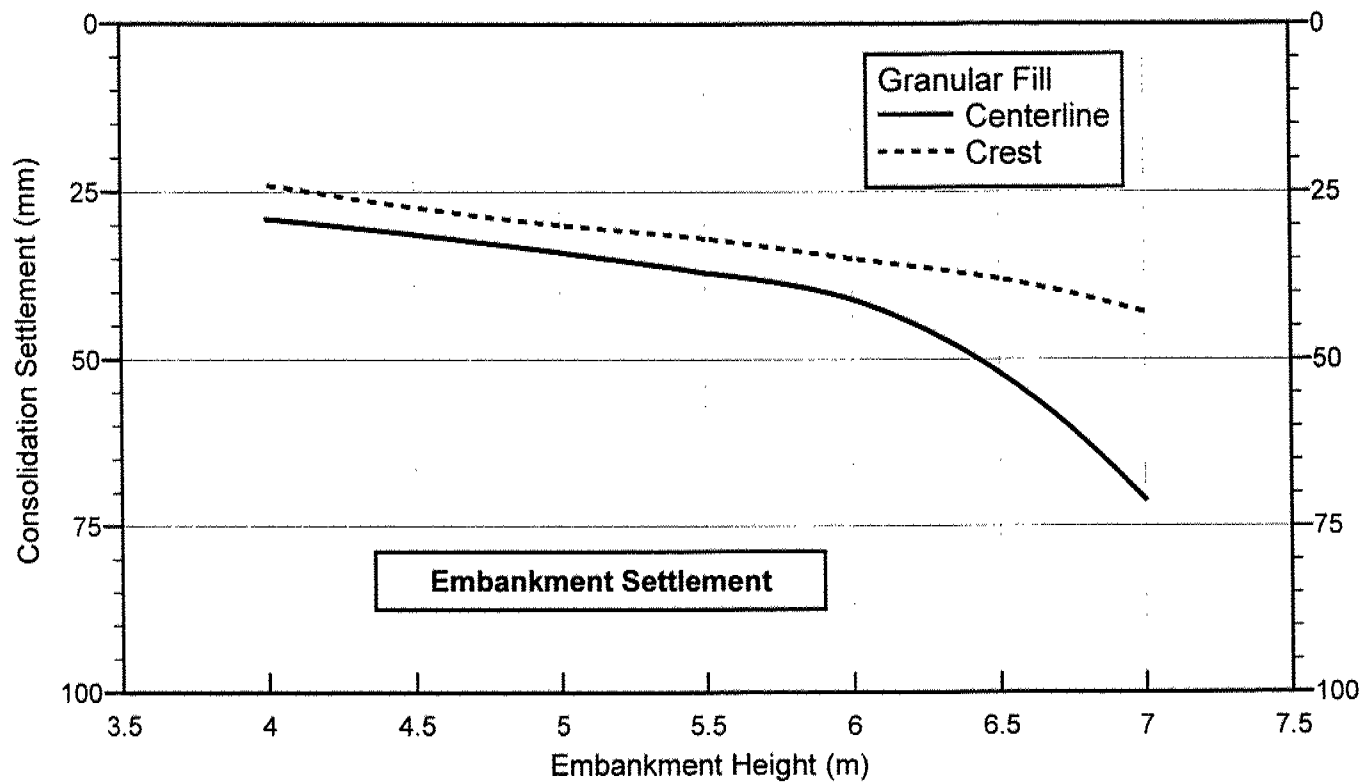
Trout Creek Bridge - NORTHBOUND LANES

F98179-B/G

Nov 23/99

Figure A5





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Thunder Bay, Ontario

### Estimated Consolidation Settlement NORTH APPROACH

F98179-B/G

Mar 3/99

Marshall Macklin Monaghan

Trout Creek Bridge - NORTHBOUND LANES

Figure A7

B

# RECORD OF BOREHOLE BH-1NF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station +10+564, offset 7.6 m left of centreline of Northbound Lane  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55  
 DATUM Geodetic DATE November 17, 1998  
 ORIGINATED BY I.D.  
 COMPILED BY M.D.  
 CHECKED BY E.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20	40	60	80					
313.80	GROUND SURFACE															
0.00	TOPSOIL, 150 mm over SAND, fine to medium, some SILT, brown. (loose)		1	SS	9		313									
312.60	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (stiff becoming firm)		2	SS	8		312									
1.20			3	SS	2		311									
			4	SS	2		310									
309.53	SILTY SAND & GRAVEL, till like structure, some cobbles, grey, wet. (dense)		5	SS	27		309									
4.27																
307.46	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER		6	SS	60		308									
6.34																
	Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 017.2 N, 313 985.1 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model.															



# RECORD OF BOREHOLE BH-2NF 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station ~10+559, offset ~6 m right of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / B.Q. core COMPILED BY M.D.  
 DATUM Geodetic DATE November 17, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT				UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20    40    60    80				wp    —    w    —    wl					
								SHEAR STRENGTH: Cu, KPa ● UNCONFINED QUICK TRIAXIAL    × FIELD VANE LAB SHEAR				WATER CONTENT (%) 10    20    30    40					
314.30	GROUND SURFACE																
0.00	TOPSOIL, ~150 mm over SAND, fine to medium, trace of SILT, brown, occasional organics. (compact)		1	SS	25	≡	314								0% 97% 3%		
			2	SS	15		313										
311.95			3	SS	17		312										
2.39	SILTY SAND & GRAVEL, till-like structure, some cobbles, grey, wet. (compact)		4	SS	11		311										
310.43							310										
3.87	BIOTITE HORNBLENDE GNEISS		5	BQ			309									Rec. 100% RQD 87%	
			6	BQ		308									Rec. 100% RQD 96%		
307.29	END OF BOREHOLE																
7.01	Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 092 981.7 N, 313 998.0 E. 3) Borehole elevation obtained Marshall Macklin Monaghan terrain model. 4) Water level was ~2.3 & hole was open to ~2.4 m depth on completion.																





# RECORD OF BOREHOLE BH-3NF 1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station ~10+601, offset ~6 m left of centreline of Northbound Lane

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Hollow stem augers / B.Q. core

COMPILED BY M.D.

DATUM Geodetic

DATE November 16, 1998

CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT wp	NATURAL MOISTURE CONTENT w	LIQUID LIMIT wl	UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOW(S) 0.3m	SHEAR STRENGTH: Cu, KPa						
								UNCONFINED QUICK TRIAXIAL						FIELD VANE LAB SHEAR
							20 40 60 80	20 40 60 80	WATER CONTENT (%) 10 20 30 40					
312.54	GROUND SURFACE													
0.00	SAND, fine to medium, occasional organics, brown, dry.													
311.94														
0.60	CLAYEY SILT, with bands of SILT, brown to grey, wet seams below ~2.5 m depth. (compact to loose)		1	SS	14									
			2	SS	12									
			3	SS	4									
309.49			4	SS	3									
3.05	SILTY CLAY, moist to wet, thinly laminated with SILT, low to medium plasticity. (stiff becoming firm)		5	SS	2									
			6	SS	3									
			7	SS	18									
304.62	SAND & GRAVEL, till-like structure, some cobbles, trace of SILT, grey, wet. (dense)		8	SS	30								0% 7% 93%	
7.92			9	BQ										
302.42	BIOTITE HORNBLende GNEISS		10	BQ									Rec. 100% RQD 78%	
10.12														
299.25	END OF BOREHOLE												Rec. 100% RQD 90%	
13.29	Notes: 1) This borehole forms part of the Trout Creek Bridge, Northbound Lane, Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 050.0 N, 313 968.9 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model. 4) Borehole caved dry at ~8.4 m depth on completion. 5) Area levelled with a bulldozer prior to advancing borehole. 6) This borehole is located at the top edge of a steep slope (dropping of northward).													



# RECORD OF BOREHOLE BH-4NF 1 OF 1

## METRIC

W.P. 774-93-00      LOCATION Station ~10+602, offset ~6 m right of centreline of Northbound Lane      ORIGINATED BY I.D.  
 DIST 54      HWY 11      BOREHOLE TYPE Hollow stem augers / CME-55      COMPILED BY M.D.  
 DATUM Geodetic      DATE November 16, 1998      CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60			80	wp	w	wl																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
313.51	GROUND SURFACE																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							



# RECORD OF BOREHOLE BH-5NF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+659, on centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 10, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION			
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20						40	60	80
								SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL      FIELD VANE LAB SHEAR								
307.04	GROUND SURFACE															
0.00	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		1	SS	5											
			2	SS	3											
			3	SS	2											
			4	SS	4											
			5	SS	6											
301.81	SILTY SAND & GRAVEL, brown, wet. (compact)		6	SS	20											
5.23																
300.12	BIOTITE HORNBLende GNEISS,		7	BQ												
6.92			8	BQ												
			9	BQ												
296.77	END OF BOREHOLE															
10.27	Notes: 1) This borehole forms part of the Trout Creek Bridge, Northbound Lane, Foundation Investigation. 2) Borehole located at U.T.M. coordinates 7 093 104.7 N, 313 950.9 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model. 4) Water level was at surface & hole was open to ~5.7 m depth on completion. 5) This area was levelled with a bulldozer prior to advancing borehole. 6) This borehole is located at the top edge of a cliff (dropping off northward).															



# RECORD OF BOREHOLE BH-6NF 1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station 10+656, offset 7 m left of centreline of Northbound Lane

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Hollow stem augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE November 13, 1998

CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION  GR   SA   (SI & CL)			
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20   40   60   80				wp      w   wl							
							SHEAR STRENGTH: Cu, KPa ● UNCONFINED QUICK TRIAXIAL   × FIELD VANE LAB SHEAR				WATER CONTENT (%) 10   20   30   40							
304.88	GROUND SURFACE																	
0.00	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		1	SS	4		304											
			2	SS	5		303											
			3	SS	6		302											
			4	SS	8		301											
			5	SS	6		300											
			6	SS	3		299											
			7	SS	7		297											
296.50																		
8.38			SAND, GRAVEL & COBBLES															
296.04																		
8.84	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER																	
Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 099.2 N, 313 945.6 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Borehole caved wet at 7.2 m depth on completion. 5) This area was levelled with a bulldozer prior to advancing borehole. 6) This borehole is located at the top edge of a cliff (dropping off northward).																		



# RECORD OF BOREHOLE BH-7NF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+769, offset ~6 m left of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / BQ core COMPILED BY M.D.  
 DATUM Geodetic DATE November 20, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION  GR SA (SI & CL)		
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80				wp — w — wl							
								SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)							
								UNCONFINED QUICK TRIAXIAL		FIELD VANE LAB SHEAR									
310.51	GROUND SURFACE																		
0.00	SAND, medium, brown, moist. (compact)		1	SS	20														
			2	SS	15														
308.38	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (stiff to firm)		3	SS	17														
2.13			4	SS	17												0% 0% 100%		
			5	SS	3												0% 0% 100%		
			6	SS	8														
			7	SS	6														
			8	SS	9														
			9	SS	9														
299.14	BIOTITE HORNBLENDE GNEISS		10	BQ													Rec. 100% RQD 78%		
11.37			11	BQ													Rec. 100% RQD 65%		
295.94	END OF BOREHOLE		12	BQ													Rec. 100% RQD 65%		
14.57	<b>Notes:</b> 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 204.5 N, 313 907.7 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model. 4) Borehole caved dry at ~3.8 m depth on completion. 5) This area was levelled with a bulldozer prior to advancing borehole.																		

Notes:  
 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation.  
 2) Borehole located at U.T.M. coordinates 5 093 204.5 N, 313 907.7 E.  
 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model.  
 4) Borehole caved dry at ~3.8 m depth on completion.  
 5) This area was levelled with a bulldozer prior to advancing borehole.



# RECORD OF BOREHOLE BH-8NF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station +10+765, offset 7 m right of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 20, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION			
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20						40	60	80
312.21	GROUND SURFACE															
0.00	TOPSOIL, ~150 mm over SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (stiff to firm)		1	SS	22											
			2	SS	26											
			3	SS	15											
			4	SS	10											
			5	SS	5											
			6	SS	3											
			7	SS	3											
			8	SS	6											
			9	SS	7											
			10	SS	11											
298.51			11	SS	60											
13.78	SILTY SAND & GRAVEL, till-like structure, possible cobbles, wet. (dense)															
298.40	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER															
13.81	Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 204.7 N, 313 921.3 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Borehole caved wet at ~11.6 m depth on completion.															



# RECORD OF BOREHOLE BH-9NF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+810, offset ~6 m left of centreline of Northbound Lane ORIGINATED BY J.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 24, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	SHEAR STRENGTH: Cu, KPa		WATER CONTENT (%)			
								UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR	wp			w
312.80	GROUND SURFACE						20 40 60 80	20 40 60 80	10 20 30 40		GR SA (SI & CL)		
0.00	TOPSOIL, ~150 mm over SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (stiff to firm)		1	SS	15								
			2	SS	15							0% 0% 100%	
			3	SS	8								
			4	SS	6								
			5	SS	1								
			6	SS	2							0% 2% 98%	
			7	SS	3								
			8	SS	6								
			9	SS	6								
301.37													
11.43	SILT, occasional thin CLAY seams, grey, wet. (loose)		10	SS	8								
299.39													
13.41	SILTY SAND & GRAVEL, till-like structure, grey, wet. (compact to dense)		11	SS	13								
298.11													
14.69	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER												
Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 243.3 N, 313 896.0 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model. 4) Borehole caved wet at ~12.7 m depth on completion.													



# RECORD OF BOREHOLE BH-10NF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+810, offset ~6 m right of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 24, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)				CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)								
								UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR	wp	w	wl	10	20	30	40				
312.93	GROUND SURFACE																			
0.00	TOPSOIL, ~150 mm over SAND, fine, ~150 mm over SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (stiff to firm)		1	SS	17		312													
			2	SS	12		311													
			3	SS	9		310													
			4	SS	4		309													
			5	TW			308													
			6	SS	2		307													
			7	TW			306													
			8	SS	8		305													
			9	SS	12		304													
			10	SS	21		303													
			11	SS	6		302													
301.04	SILT, with occasional thin CLAY seams, grey, wet. (loose)		12	BQ			301													
299.82	SILTY SAND & GRAVEL, till-like structure, grey, moist. (compact to dense)		13	BQ			300													
298.18	BIOTITE HORNBLENDE GNEISS						299													
	Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 246.6 N, 313 907.5 E.						298													
							297													
							296													
295.13	END OF BOREHOLE																			
17.80	Notes: (cont'd) 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model. 4) Borehole caved wet at ~11.4 m depth on completion.																			





# RECORD OF BOREHOLE BH-2DF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+668, offset ~7 m right of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 11, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20	40	60	80	wp	wl	10	20	30	40		
304.80 0.00	GROUND SURFACE																	
	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		1	SS	7													
			2	SS	8													
			3	SS	5													
			4	SS	3													
			5	SS	4													
298.55 6.25	BIOTITE HORNBLende GNEISS, good to excellent rock quality, unweathered.		6	SS	8													0% 15% 85%
			7	BQ														Rec. 99% RQD 78%
			8	BQ														Rec. 100% RQD 96%
295.50 9.30	END OF BOREHOLE																	
Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 115.5 N, 313 954.0 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Borehole caved wet at ~6.0 m depth on completion.																		



# RECORD OF BOREHOLE BH-3DF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+714, offset ~5 m left of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / Dynamic cone COMPILED BY M.D.  
 DATUM Geodetic DATE June 26, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60					
301.11	GROUND SURFACE														
0.00	TOPSOIL, ~150 mm over SILTY SAND, some wood inclusions, brown to grey, wet below ~1.5 m depth, CREEK ALLUVIUM. (loose to very loose)		1	SS	7										
			2	SS	9										
			3	SS	3										
			4	SS	0										
297.11	SILTY SAND & GRAVEL, till-like structure, occasional cobbles, grey, wet. (dense)		5	SS	40										
4.00			6	SS	10										
294.34	BIOTITE HORNBLende GNEISS, excellent rock quality, unweathered.		7	BQ											
6.77			8	BQ											
291.27	END OF BOREHOLE														
9.84	Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 153.4 N, 313 926.4 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan survey crew. 4) Water level was at ~1.4 m & hole was open to ~3.9 m depth on completion.														



# RECORD OF BOREHOLE BH-4DF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+745, offset 7 m right of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / Dynamic cone COMPILED BY M.D.  
 DATUM Geodetic DATE June 29, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION					
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60			80	wp	w	wl	
306.06	GROUND SURFACE																
0.00	SAND																
305.46																	
0.60	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		1	SS	9												
			2	SS	4												
			3	SS	5												
			4	SS	6												
			5	SS	6												
			6	SS	6												
298.86	SAND, GRAVEL & COBBLES																
7.20																	
298.50	BIOTITE HORNBLENDE GNEISS, fair to good rock quality, slightly weathered to weathered.		7	BQ													Rec. 100% RQD 71%
7.56			8	BQ													Rec. 100% RQD 78%
295.36	END OF BOREHOLE																
10.70	Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 186.1 N, 313 927.5 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Water level was at ~5.2 m & hole was open to ~5.4 m depth on completion. 5) Dynamic cone penetration test driven at station ~10+745, offset ~7 m right of centreline as referenced to the Northbound Lane.																



# RECORD OF BOREHOLE BH-5DF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+663, offset 2 m left of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 11, 1998 CHECKED BY I.G.

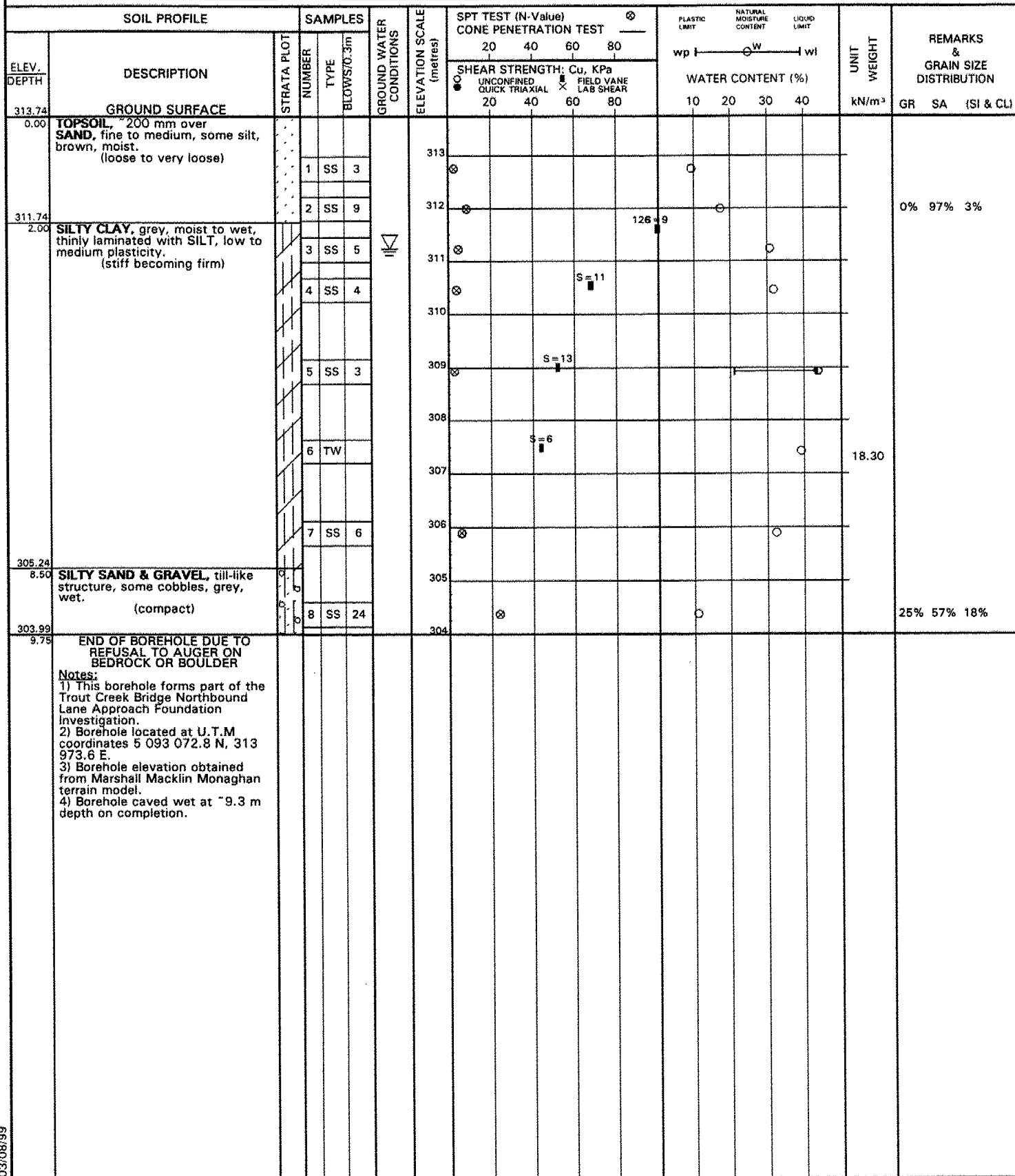
SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT wp	NATURAL MOISTURE CONTENT w	LIQUID LIMIT wl	UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20	40	60	80					
304.31 0.00	GROUND SURFACE														
	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		1	SS	5										
			2	SS	6										
			3	SS	5										
			4	SS	6										
			5	SS	7										0% 0% 100%
298.27 6.04	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER														
Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 107.6 N, 313 947.6 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Borehole was dry & open to full depth on completion.															



# RECORD OF BOREHOLE BH-6DP 1 OF 1

METRIC

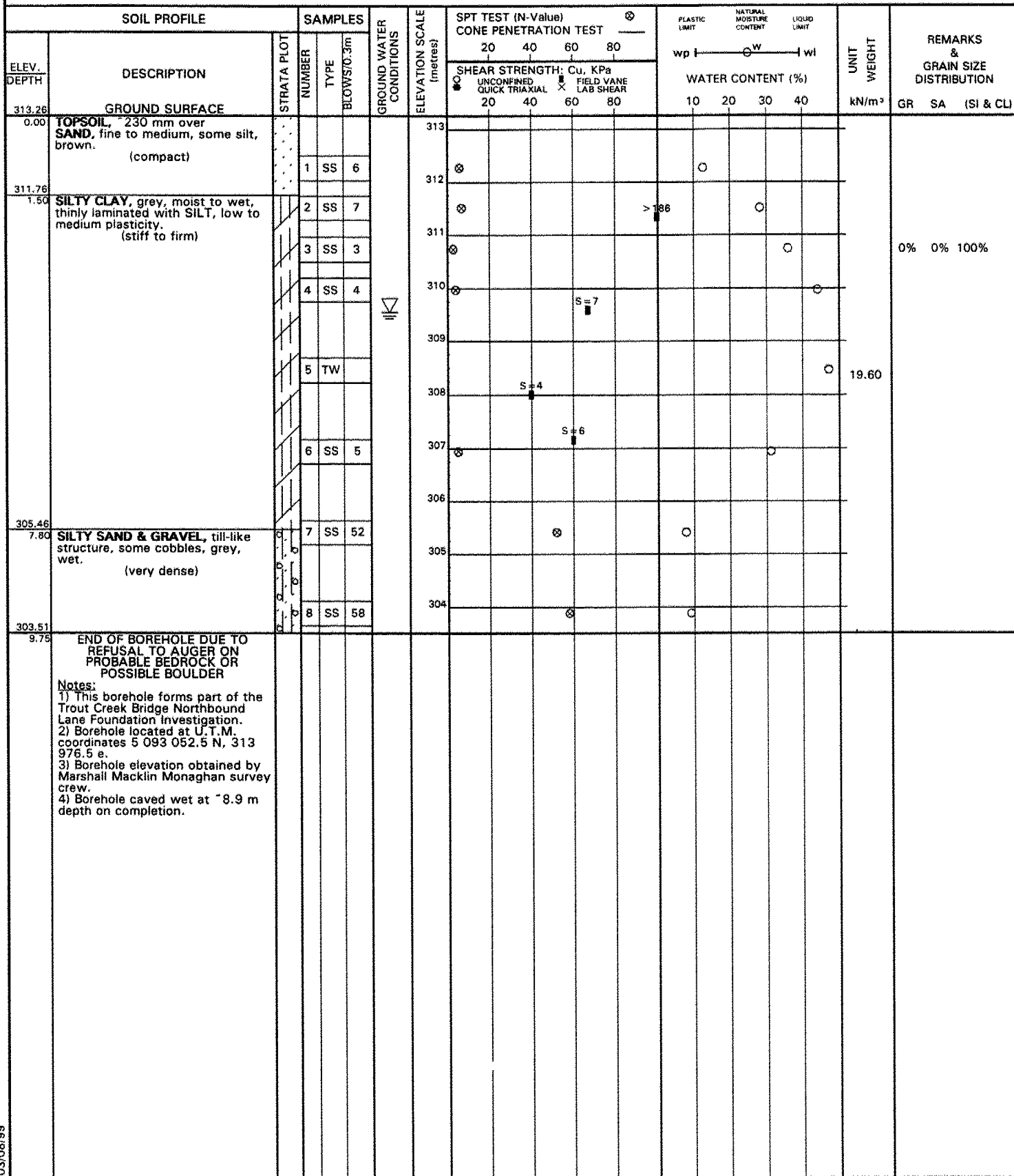
W.P. 774-93-00 LOCATION Station ~10+620, offset ~8 m right of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 16, 1998 CHECKED BY I.G.



# RECORD OF BOREHOLE BH-8DP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+600, offset ~2 m right of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 16, 1998 CHECKED BY J.G.



# RECORD OF BOREHOLE BH-9DP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+572, offset ~3 m right of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 16, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)		CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20	40	60	80	wp	w		
313.59	GROUND SURFACE														
0.00	TOPSOIL, ~200 mm over SAND, fine to medium, some silt, brown. (compact)		1	SS	16										
312.09															
1.50	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		2	SS	10										
			3	SS	1										
			4	TW											
309.63															
3.96	SAND, GRAVEL & COBBLES														
309.17															
4.42	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER														
Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Approach Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 028.3, 313 989.5 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Water level was at ~4.1 m & hole was open to ~4.2 m depth on completion.															



# RECORD OF BOREHOLE BH-10DP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+548, offset 1 m right of centreline of Northbound Lane  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55  
 DATUM Geodetic DATE June 17, 1998  
 ORIGINATED BY I.D.  
 COMPILED BY M.D.  
 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA	NUMBER	TYPE		20	40	60	80	wp	w	wl	WATER CONTENT (%)		
314.56	GROUND SURFACE														
0.00	TOPSOIL, 230 mm over SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (stiff to firm)		1	SS	17										
312.56			2	SS	21										
2.00	SILTY SAND & GRAVEL, some cobbles. (compact to dense)		3	SS	33										
311.97															
2.59	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER														
	Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Approach Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 006.4 N, 313 998.6 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Borehole was dry & open to depth on completion. 5) Drill moved 2.0 m south of BH-10DF & met auger refusal at 2.6 m depth.														





# RECORD OF BOREHOLE BH-12DP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+504, offset ~1 m left of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 17, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20      40      60      80				wp      —w      wl				
								SHEAR STRENGTH: Cu, KPa ● UNCONFINED      × FIELD VANE QUICK TRIAXIAL      LAB SHEAR				WATER CONTENT (%) 10      20      30      40				
316.32	GROUND SURFACE														GR    SA    (SI & CL)	
0.00	TOPSOIL, ~250 mm over SILTY SAND & GRAVEL, till-like structure, some cobbles, brown to grey, wet below ~1.2 m depth. (compact to dense)		1	SS	12										29% 53% 18%	
			2	SS	35											
			3	SS	25											
			4	SS	21											
312.45 3.87	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER															
<p>Notes:</p> <p>1) This borehole forms part of the Trout Creek Bridge Northbound Lane Approach Foundation Investigation.</p> <p>2) Borehole located at U.T.M. coordinates 5 092 967.5 N, 314 017.8 E.</p> <p>3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew.</p> <p>4) Water level was at ~1.5 m &amp; hole was open to ~3.7 m depth on completion.</p> <p>5) Drill moved ~2.0 m north of BH-12DF &amp; met auger refusal at ~3.8 m depth.</p>																



# RECORD OF BOREHOLE BH-17DP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+628, offset 9 m right of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 19, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  KN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20      40      60      80				wp      —w      wl				
								SHEAR STRENGTH: Cu, KPa ● UNCONFINED      ■ FIELD VANE ○ QUICK TRIAXIAL      X LAB SHEAR				WATER CONTENT (%) 10      20      30      40				
304.11	GROUND SURFACE															
0.00	TOPSOIL, ~150 mm over SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm to soft)		1	SS	4	≡	304									
			2	SS	2		303									
			3	SS	4		302									
			4	SS	50		301									
			5	SS	28		300									
301.11	SILTY SAND & GRAVEL, till-like structure, grey, some cobbles. (compact to dense)						299									
298.93	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER															
5.18	Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Approach Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 07.0 N, 313 954.9 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Borehole caved wet at ~2.6 m depth on completion.															



# RECORD OF BOREHOLE BH-18DF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+712, offset ~7 m right of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 26, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR		
300.76	GROUND SURFACE										
0.00	TOPSOIL, ~150 mm over SAND, traces of organics & roots, brown to grey, wet below ~2.0 m depth. (loose)		1	SS	17						
			2	SS	5						
			3	SS	6						
			4	SS	10						
			5	AS							
295.09	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER										
5.67	Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 155.7 N, 313 938.3 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Water level was at ~2.1 m & hole was open to ~2.4 m depth on completion. 5) Dynamic cone penetration test driven at station ~10+712, offset ~6 m right of centreline as referenced to the Northbound Lane.										



## 1 OF 1

METRIC

CHECKED BY I.G.

# RECORD OF BOREHOLE BH-21DP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+775, on centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE July 2, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)				
								UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR	wp	w	wl				
							20	40	60	80	10	20	30	40		
312.76	GROUND SURFACE															
0.00	TOPSOIL, ~150 mm over SAND, medium, brown, moist. (loose)															
311.76	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (stiff to firm)		1	SS	8											
1.00			2	SS	5											
			3	SS	13											
			4	SS	9											
			5	SS	5											
			6	SS	1											
			7	TW												
			8	SS	7											
			9	SS	7											
			10	SS	10											
			11	SS	12											
298.46	SAND, GRAVEL & COBBLES															
14.30	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER															
298.19	Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Approach Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 211.9 N, 313 911.6 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model. 4) Borehole caved dry at ~11.4 m depth on completion.															
14.57																



# RECORD OF BOREHOLE BH-23DP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station +10+825, on centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE September 23, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20 40 60 80				wp — w — wl				
								SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL X FIELD VANE LAB SHEAR 20 40 60 80				WATER CONTENT (%) 10 20 30 40				
312.14 0.00	GROUND SURFACE															
	TOPSOIL, ~175 mm over SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		1	SS	11											
			2	SS	7										0% 0% 100%	
			3	SS	6											
			4	SS	4											
			5	TW										19.50		
			6	SS	4										0% 0% 100%	
			7	SS	8											
			8	SS	7											
301.39 10.75	SILT, grey, wet. (loose to compact)		9	SS	11											
299.64 12.50	SAND, brown. (dense)		10	SS	16											
299.49 12.65	END OF BOREHOLE															
Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Approach Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 259.2 N, 313 897.8 E. 3) Borehole elevation obtained from Dearden and Stanton Ltd., O.L.S. terrain model. 4) Borehole caved wet at ~5.8 m depth on completion.																



# RECORD OF BOREHOLE BH-18EP 1 OF 1

## METRIC

W.P. 774-93-00

LOCATION Station ~10+560, on centreline of Median

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Hollow stem augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE June 19, 1998

CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20	40	60	80	wp	w	wl		
313.75	GROUND SURFACE															
0.00	TOPSOIL, ~125 mm over CLAYEY SILT, seams of SILT, brown to grey, wet below ~2.0 m depth. (stiff)															
			1	SS	6											
			2	SS	4											
	Cobbles at base.															
309.48	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER															
4.27	Notes: 1) This borehole forms part of Trout Creek Bridge Southbound Lane Approach Foundation investigation. 2) Borehole located at U.T.M. coordinates 5 093 007.8 N, 313 975.6 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Borehole caved dry at ~3.3 m depth on completion.															



# RECORD OF BOREHOLE AP-1NF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+559, offset 6 m left of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 17, 1998 CHECKED BY E.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION		
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	20	40	60	80	wp	w			wl	10
313.82 0.00	GROUND SURFACE																
312.60 1.22	Probable SAND																
310.42 3.35	Probable SILT and SILTY CLAY																
308.58 5.24	Probable SAND, GRAVEL & COBBLES TILL																
END OF PROBE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER Notes: 1) This auger probe forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Probe located at U.T.M. coordinates 5 093 012.8 N, 313 987.4 E. 3) Probe elevation obtained from Marshall Macklin Monaghan terrain model.																	





# RECORD OF BOREHOLE AP-2NF 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station ~10+561, on centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 17, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				WATER CONTENT (%)				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION		
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60	80	wp	w	wl			10	20
313.91 0.00	GROUND SURFACE																	
312.54 1.37	Probable SAND					313												
310.71 3.20	Probable SILT and SILTY CLAY					312												
309.76 4.15	Probable SAND, GRAVEL & COBBLE TILL					311												
	END OF PROBE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER.					310												
Notes: 1) This auger probe forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 017.3 N, 313 991.8 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model.																		



# RECORD OF BOREHOLE AP-3NF 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station ~10+563, on centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 17, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20	40	60	80					
313.90 0.00	GROUND SURFACE														
312.53 1.37	Probable SAND					313									
	Probable SILT and SILTY CLAY					312									
310.85 3.05	Probable SAND, GRAVEL & COBBLES					311									
309.72 4.18	END OF PROBE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER.					310									
<b>Notes:</b> 1) This auger probe forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Probe located at U.T.M. coordinates 5 093 019.0 N, 313 990.9 E. 3) Probe elevation obtained from Marshall Macklin Monaghan terrain model.															



# RECORD OF BOREHOLE AP-4NF 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station ~10+563, offset ~7 m right of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 17, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT				UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20    40    60    80				wp    —    w    —    wl					
								SHEAR STRENGTH: Cu, KPa ● UNCONFINED    ■ FIELD VANE ○ QUICK TRIAXIAL    x LAB SHEAR				WATER CONTENT (%) 10    20    30    40					
314.21 0.00	GROUND SURFACE  TOPSOIL, ~150 mm over						314										
	Probable SAND																
312.99 1.22	-----  Probable SILT and SILTY CLAY						313										
311.47 2.74	-----  Probable SAND, GRAVEL & COBBLES						312										
310.60 3.61	END OF PROBE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER.  Notes: 1) This auger probe forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Probe located at U.T.M. coordinates 5 093 022.2 N, 313 997.1 E. 3) Probe elevation obtained from Marshall Macklin Monaghan terrain model.						311										



# RECORD OF BOREHOLE AP-1DF 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station ~10+639, offset ~7 m right of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 19, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLLOT	NUMBER	TYPE			20	40	60	80					
311.05 0.00	GROUND SURFACE					311									
	Probable SILTY CLAY					310									
						309									
						308									
						307									
						306									
						305									
						304									
						303									
						302									
						301									
						300									
299.62 11.43	END OF AUGER PROBE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER														
	Notes: 1) This auger probe forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Auger probe located at U.T.M. coordinates 5 093 089.4 N, 313 965.1 E. 3) Probe elevation obtained from Marshall Macklin Monaghan terrain model.														



# RECORD OF BOREHOLE AP-2DF 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station 10+667, offset 16 m left of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 19, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20   40   60   80				wp   ——— w   ——— wl					
								SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)					
							UNCONFINED   FIELD VANE										
							QUICK TRIAXIAL   LAB SHEAR										
							20   40   60   80				10   20   30   40				kN/m <sup>3</sup> GR   SA   (SI & CL)		
303.83	GROUND SURFACE																
0.00																	
	Probable SILTY CLAY						303										
							302										
							301										
							300										
							299										
297.86							298										
5.97	END OF AUGER PROBE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER																
	Notes: 1) This auger probe forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Auger probe located at U.T.M coordinates 5 093 109.7N, 313 942.4 E. 3) Probe elevation obtained by Marshall Macklin Monaghan survey crew.																



# RECORD OF BOREHOLE C-1DF

1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station 10+637, offset 7 m right of centreline of Northbound Lane

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Dynamic cone /

COMPILED BY M.D.

DATUM Geodetic

DATE June 9, 1998

CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLAT	NUMBER			TYPE	BLOWS/0.3m	UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR			wp
312.16	GROUND SURFACE											
0.00	Dynamic cone penetration test only.											
312												
311												
310												
309												
308												
307												
306												
305												
304												
303												
302												
301												
300.33												
11.83	END OF CONE TEST DUE TO 'BOUNCING' REFUSAL ON BEDROCK OR BOULDER											
	Notes: 1) This cone test forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Cone test located at U.T.M. coordinates 5 093 087.6 E, 313 965.9 E. 3) Cone test elevation obtained by Marshall Macklin Monaghan survey crew.											



# RECORD OF BOREHOLE C-2DF 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station +10+669, offset 5 m left of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Dynamic cone / COMPILED BY M.D.  
 DATUM Geodetic DATE June 9, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT				UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR   SA   (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	20	40	60	80	wp	w		
303.77 0.00	GROUND SURFACE					SHEAR STRENGTH: Cu, KPa UNCONFINED   QUICK TRIAXIAL   FIELD VANE   LAB SHEAR 20   40   60   80				WATER CONTENT (%) 10   20   30   40					
299.20 4.57	END OF CONE TEST DUE TO BOUNCING REFUSAL ON BEDROCK OR BOULDER  Notes: 1) This cone test forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Cone test located at U.T.M. coordinates 5 093 111.9 N, 313 924.5 E. 3) Cone elevation obtained by Marshall Macklin Monaghan survey crew.														



# RECORD OF BOREHOLE C-3DF

1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station +10+714, offset +10 m right of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Dynamic cone / COMPILED BY M.D.  
 DATUM Geodetic DATE June 26, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60	80	wp	w	wl			10
300.64	GROUND SURFACE																
0.00	Dynamic cone penetration test only.																
294.95	END OF CONE TEST DUE TO "BOUNCING" REFUSAL ON BEDROCK OR BOULDER																
5.69	Notes: 1) This cone test forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Cone test located at U.T.M. coordinates 5 093 158.6 N, 313 940.4 E. 3) Cone elevation obtained by Marshall Macklin Monaghan survey crew.																





# RECORD OF BOREHOLE C-4DF

1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station 10+745, offset 5 m left of centreline of Northbound Lane

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Dynamic cone /

COMPILED BY M.D.

DATUM Geodetic

DATE June 30, 1998

CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER TYPE			BLOWS/0.3m	UNCONFINED QUICK TRIAXIAL	SHEAR STRENGTH: Cu, KPa	WATER CONTENT (%)		
306.87	GROUND SURFACE										
0.00	Dynamic cone penetration test only.										
297.60	END OF CONE TEST DUE TO 'BOUNCING' REFUSAL ON BEDROCK OR BOULDER										
9.27	Notes: 1) This cone test forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Cone test located at U.T.M. coordinates 5 093 185.5 N, 313 925.6 E. 3) Cone elevation obtained by Marshall Macklin Monaghan survey crew.										



# RECORD OF BOREHOLE C-5DF

1 OF 1

**METRIC**

W.P. <u>774-93-00</u>	LOCATION <u>Station ~10+660, offset ~6 m right of centreline of Northbound Lane</u>	ORIGINATED BY <u>I.D.</u>
DIST <u>54</u> HWY <u>11</u>	BOREHOLE TYPE <u>Dynamic cone /</u>	COMPILED BY <u>M.D.</u>
DATUM <u>Geodetic</u>	DATE <u>June 9, 1998</u>	CHECKED BY <u>I.G.</u>

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION								
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)											
305.90	GROUND SURFACE					UNCONFINED QUICK TRIAXIAL	20	40	60	80	FIELD VANE LAB SHEAR	20	40	60	80	10	20	30	40	kN/m³	GR	SA	(SI & CL)
0.00	Dynamic cone penetration test only.																						
299.42 6.48	<b>END OF CONE TEST DUE TO BOUNCING' REFUSAL ON BEDROCK OR BOULDER</b> <b>Notes:</b> 1) This cone test forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Cone test located at U.T.M. coordinates 5 093 107.9 N, 313 956.1 E. 3) Cone elevation obtained by Marshall Macklin Monaghan survey crew.																						



# RECORD OF BOREHOLE C-6DF

1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+642, offset 1 m right of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Dynamic cone / COMPILED BY M.D.  
 DATUM Geodetic DATE June 9, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60	80	wp	w		
308.50	GROUND SURFACE														
0.00	Dynamic cone penetration test only.														
299.81	END OF CONE TEST DUE TO 'BOUNCING' REFUSAL ON BEDROCK OR BOULDER														
8.69	Notes: 1) This cone test forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Cone test located at U.T.M. coordinates 5 093 089.7 N, 313 958.4 E. 3) Cone elevation obtained by Marshall Macklin Monaghan survey crew.														



SO7524G/N/F						
TABLE 1 ROCK CORE DESCRIPTION						
BH#	Core Recovery			Core Description		
	RC #	Depth (m)	% CR*	% RQD**	Depth (m)	Description
SOUTH ABUTMENT - NORTH BOUND LANE						
2-NF	1	3.87 to 4.68	100	80	3.87 to 7.01	<b>Biotite Hornblende Gneiss</b> (Garnetiferous), light grey to grey pink, medium to coarse grained, unweathered sulfate inclusions in joints, moderately close spacing of fractures dipping 45° to 90° from vertical, planar to smooth
	2	4.68 to 7.01	100	85		
3-NF	1	10.12 to 11.60	100	75	10.12 to 13.29	<b>Biotite Hornblende Gneiss</b> , grey to grey pink, medium to coarse grained, unweathered with sulfide inclusions, moderately spaced fractures and joints dipping 45° to 90° from vertical, planar and rough
	2	11.60 to 13.29	100	93		
5-NF	1	6.92 to 8.33	100	53	6.92 to 10.27	<b>Biotite Hornblende Gneiss</b> (Garnetiferous), grey pink to pinkish red, medium to coarse grained, weathered, fractures very closely spaced, dipping 0° to 90° from vertical, planar, smooth to slightly undulated
	2	8.45 to 10.27	100	81.4		

SO7524G/N/F						
TABLE 1 ROCK CORE DESCRIPTION						
BH#	Core Recovery			Core Description		
	RC #	Depth (m)	% CR*	% RQD**	Depth (m)	Description
7-NF	1	11.37 to 12.86	100	67	11.37 to 14.57	<b>Biotite Hornblende Gneiss</b> , light grey to grey-pink, medium to coarse grained, unweathered with sulfide inclusions, moderately spaced fractures and joints dipping 45° to 90° from vertical, rough and slightly undulating
	2	12.86 to 14.57	100	63		
10-NF	1	14.76 to 16.29	100	90	14.76 to 17.80	<b>Biotite Hornblende Gneiss (Garnetiferous)</b> , light grey to grey-pink, unweathered, some sulfide inclusions, moderately close spacing of fractures and joints dipping from 40° to 90° from vertical, planar and smooth
	2	16.29 to 17.80	100	100		
*CR	= Core Recovery					
**RQD	= Rock Quality Designation					

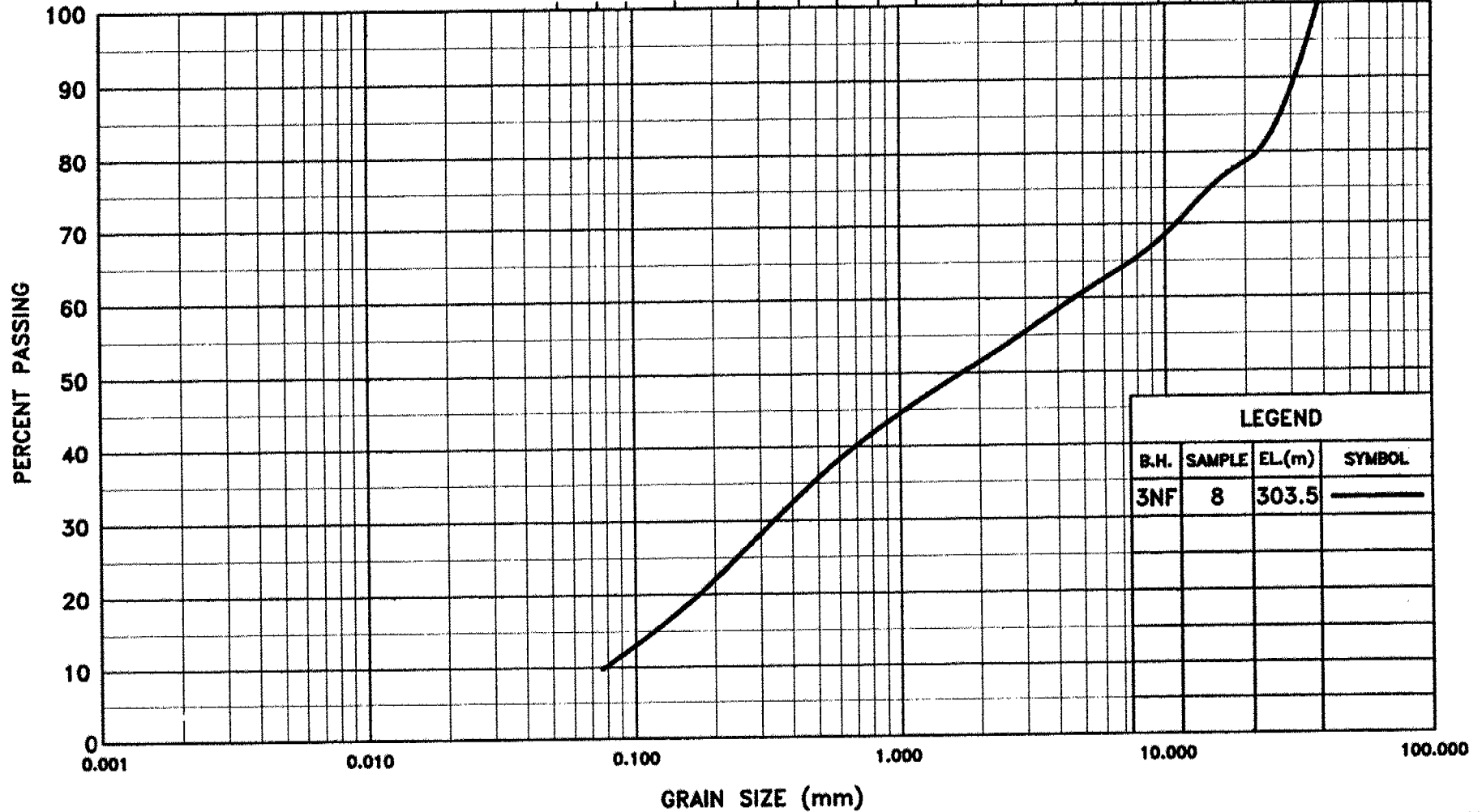
C

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)

53µm 75µm 106µm 150µm 250µm 300µm 425µm 600µm 850µm 1.18mm 2.0mm 2.36mm 4.75mm 9.5mm 13.2mm 18.0mm 28.5mm 37.5mm 53.0mm 63.0mm 75.0mm



Ministry of  
Transportation

METRIC

BH-3NF, SS-8

GRAIN SIZE DISTRIBUTION

SAND & GRAVEL

FIGURE C-1

W.P. 774-93-00

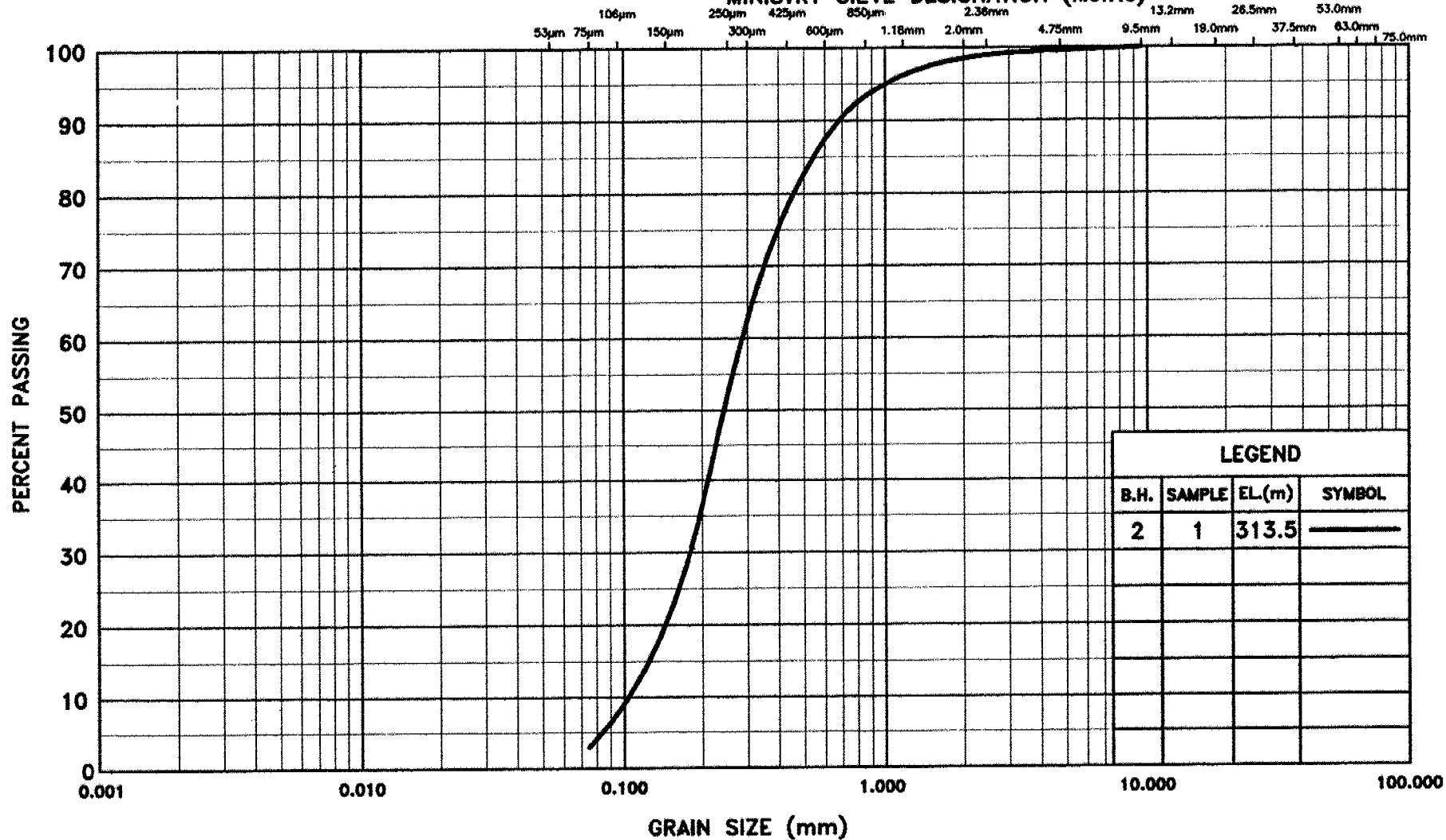


PROJ. No. S07524GN

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

### GRAIN SIZE DISTRIBUTION

BH-2NF, SS-1 SAND

FIGURE C-2

W.P. 774-93-00



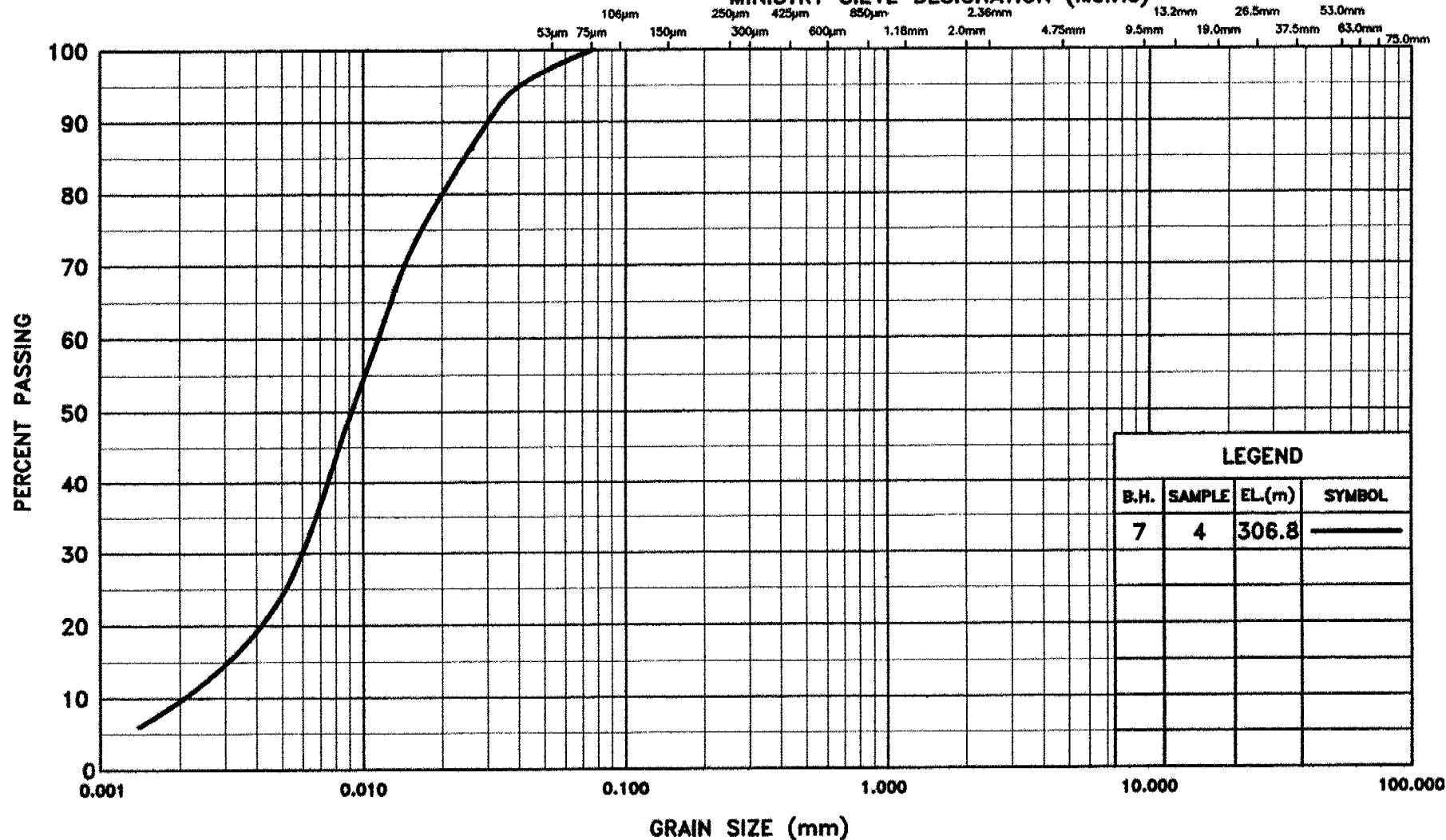
PROJ. No. S07524GN



# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

### GRAIN SIZE DISTRIBUTION

BH-7NF, SS-4 SILT

FIGURE C-3

W.P. 774-93-00

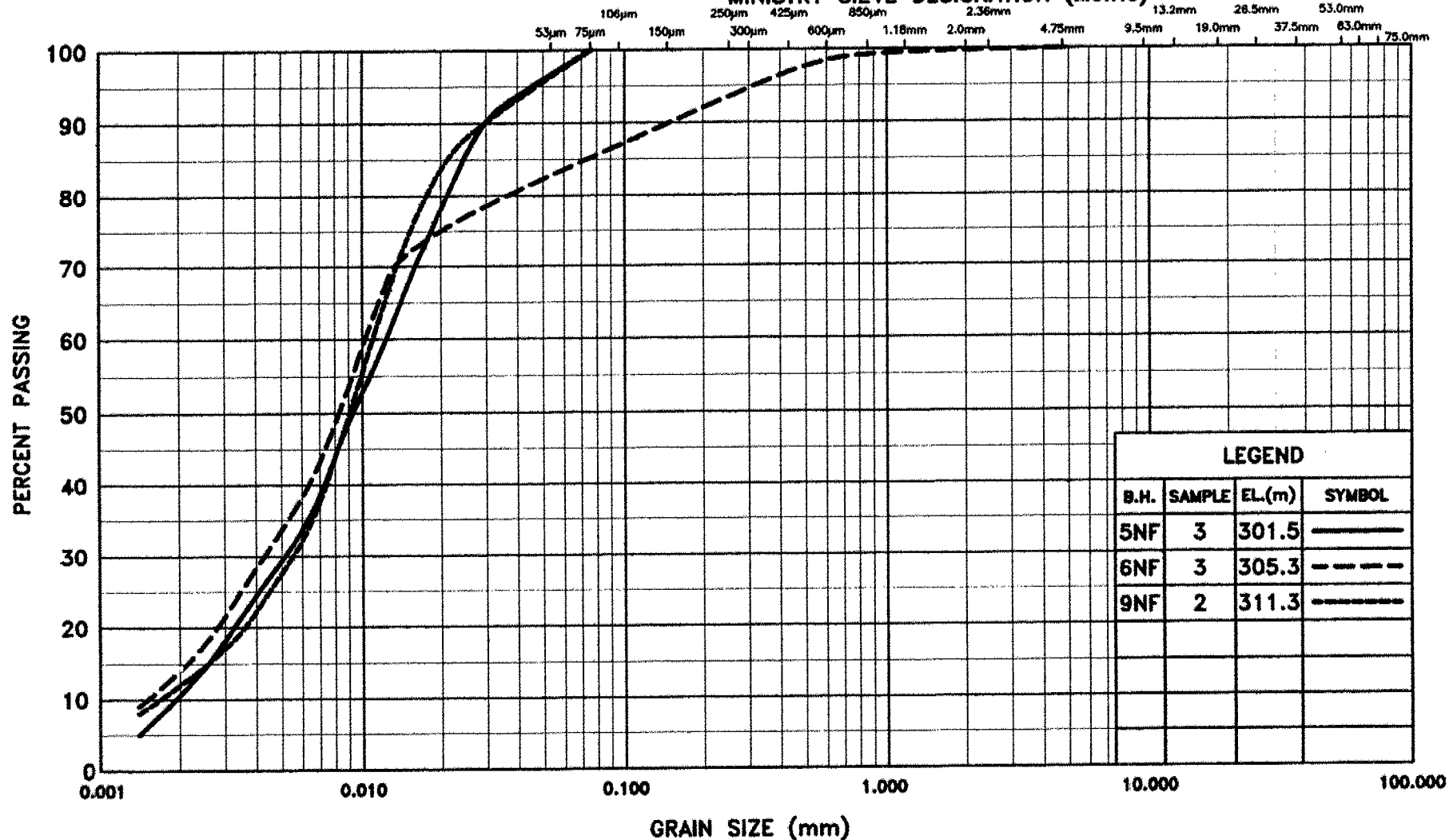


PROJ. No. S07524GN

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



LEGEND			
B.H.	SAMPLE	EL.(m)	SYMBOL
5NF	3	301.5	————
6NF	3	305.3	-----
9NF	2	311.3	-.-.-.-.-

Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION  
CLAYEY SILT

FIGURE C-4

W.P. 774-93-00



PROJ. No. S07524GN

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT

SAND

GRAVEL

FINE

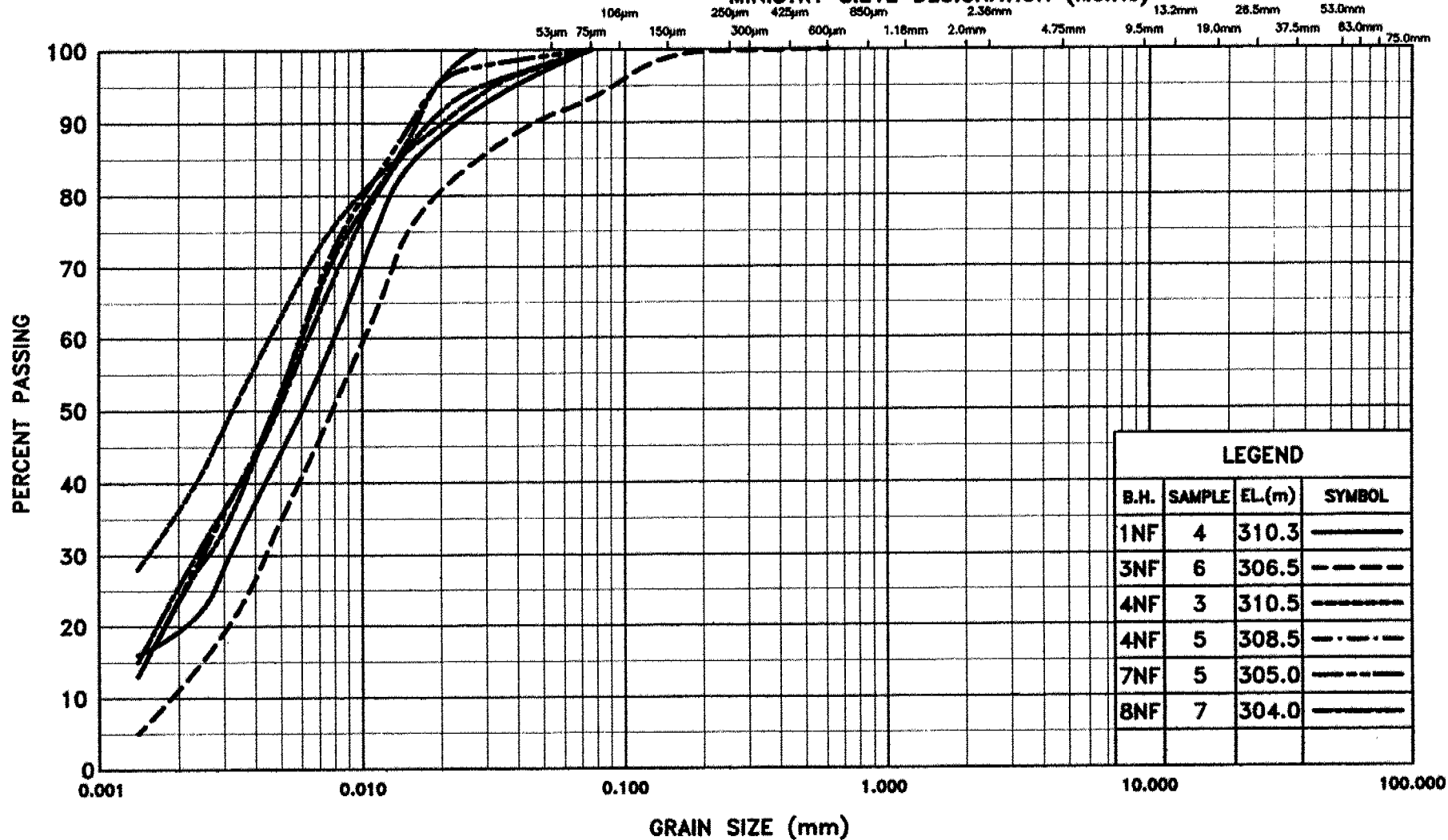
MEDIUM

COARSE

FINE

COARSE

MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION

SILTY CLAY

FIGURE C-5

W.P. 774-93-00



PROJ. No. S07524GN

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT

SAND

GRAVEL

FINE

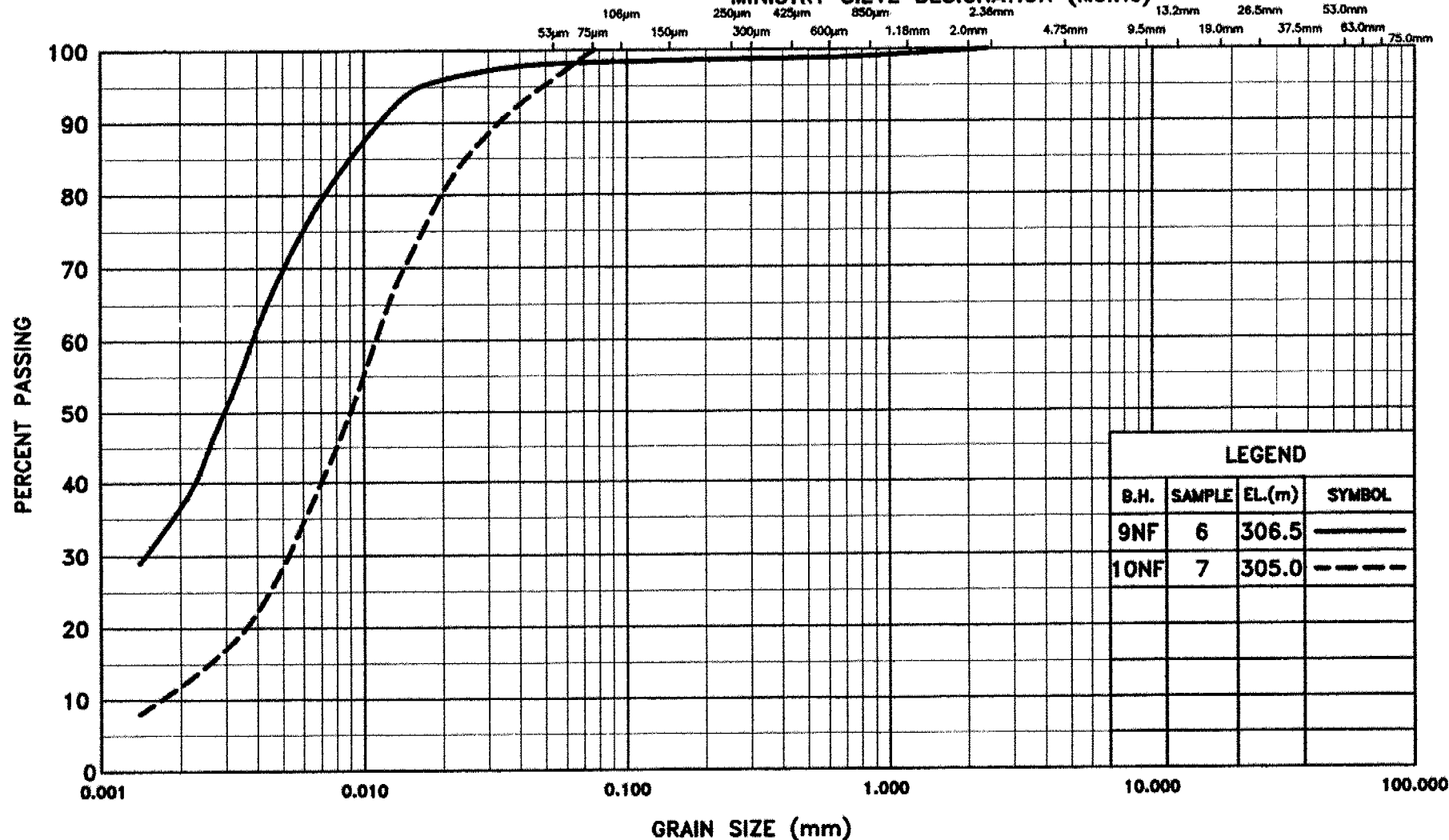
MEDIUM

COARSE

FINE

COARSE

MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION

SILTY CLAY

FIGURE C-6

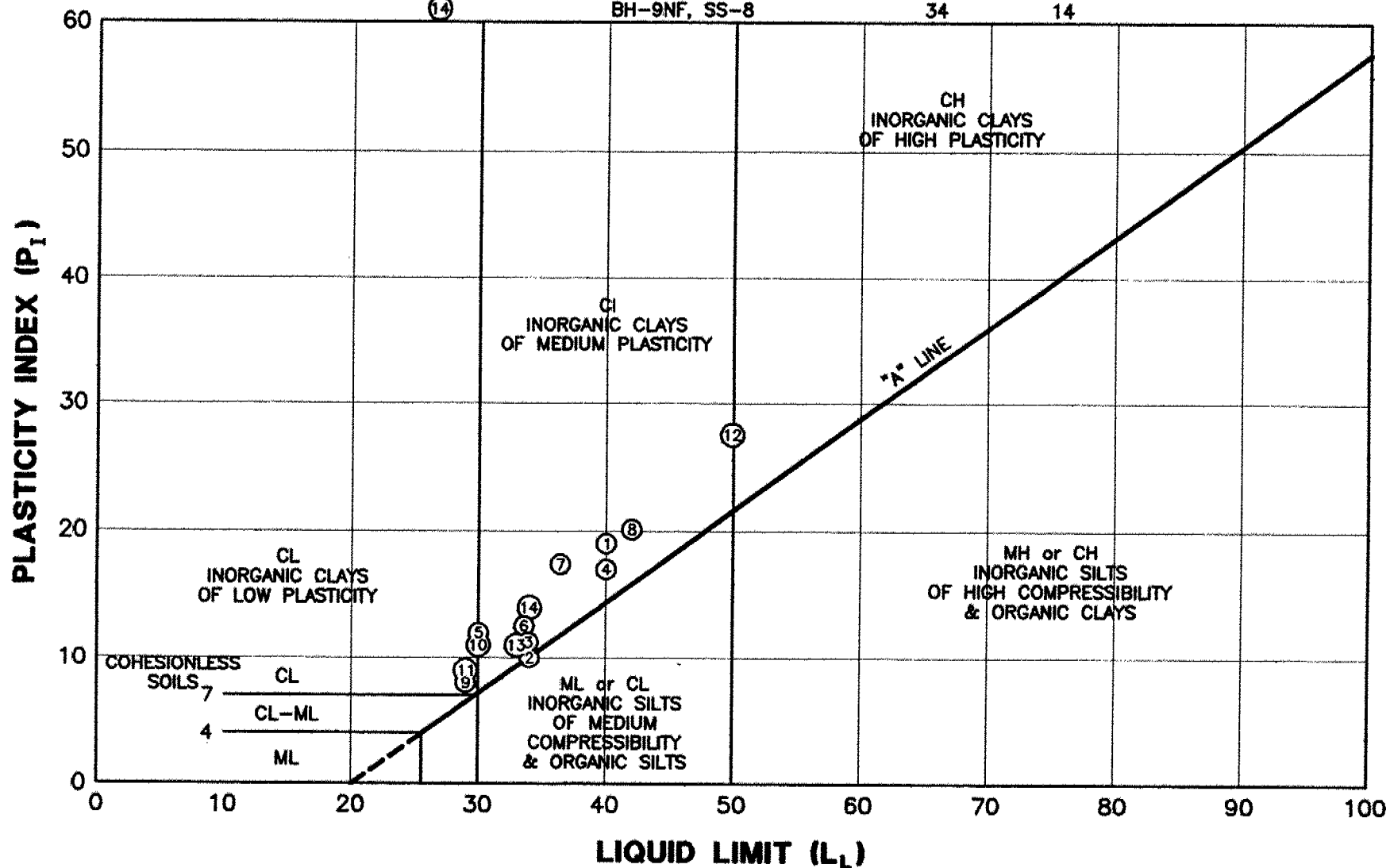
W.P. 774-93-00



PROJ. No. S07524GN

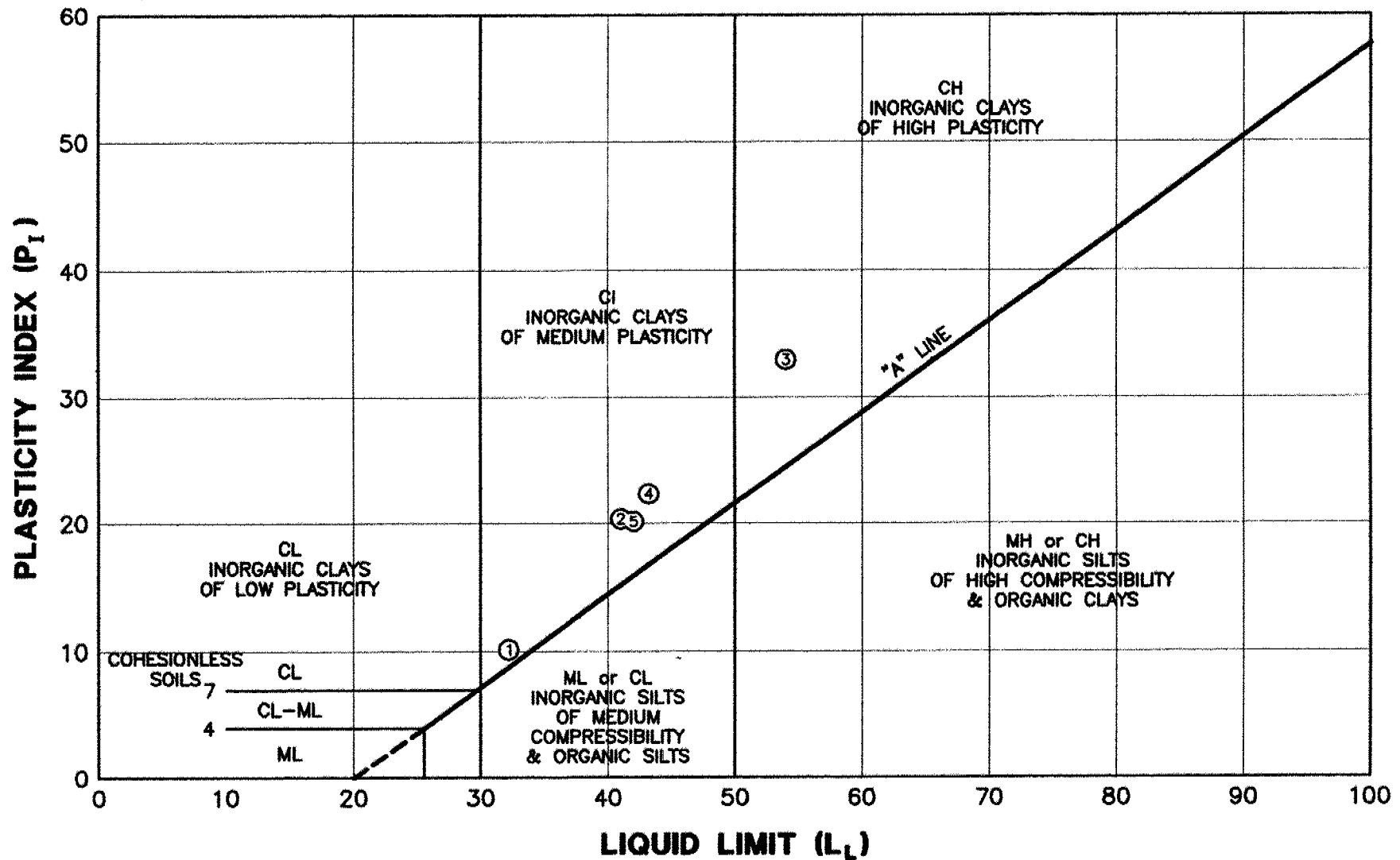
# **ATTERBERG LIMITS - PLASTICITY CHART**

SYMBOL	DESCRIPTION	LL	PI
①	BH-1NF, SS-2	40	19
②	BH-1NF, SS-3	34	10
③	BH-1NF, SS-4	34	11
④	BH-3NF, SS-3	40	17
⑤	BH-5NF, SS-3	30	12
⑥	BH-6NF, SS-5	34	12
⑦	BH-7NF, SS-5	36	17
⑧	BH-8NF, SS-7	42	20
⑨	BH-9NF, SS-3	29	8
⑩	BH-9NF, SS-4	30	11
⑪	BH-9NF, SS-6	29	9
⑫	BH-9NF, SS-6	50	28
⑬	BH-9NF, SS-7	33	11
⑭	BH-9NF, SS-8	34	14



# **ATTERBERG LIMITS - PLASTICITY CHART**

SYMBOL	DESCRIPTION	LL	PI
①	BH-1DF, SS-3	32	10
②	BH-21DP, SS-6	41	21
③	BH-4DF, SS-2	54	33
④	BH-6DP, SS-5	43	22
⑤	BH-8NF, SS-8	42	20



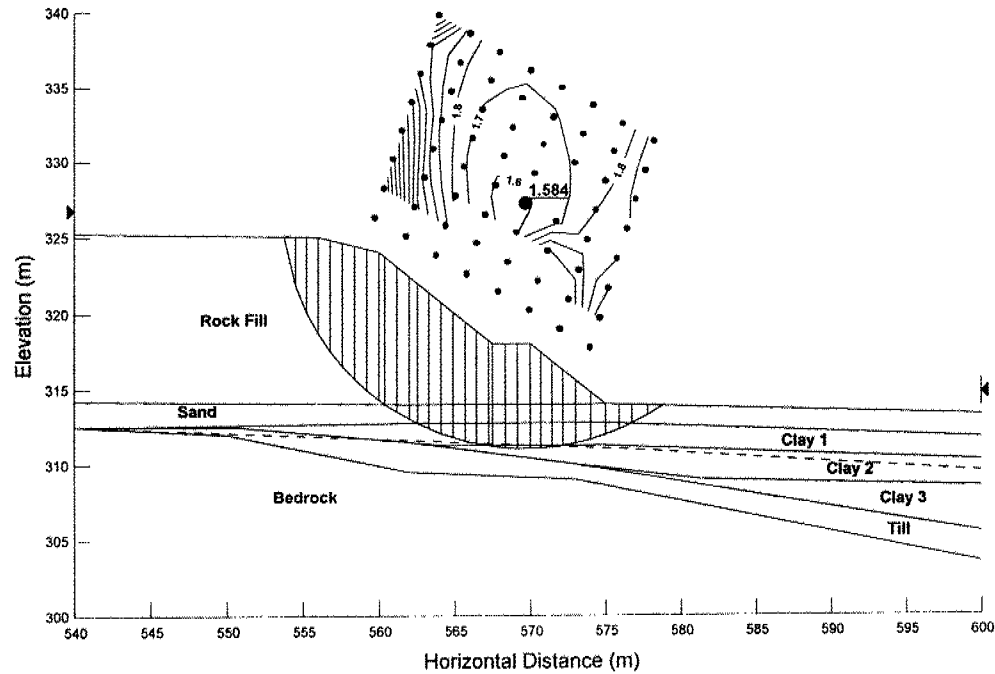
S07524GN

FIG. No. C-8



D

Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-B/G)  
 Northbound Lane, South Abutment  
 NBL\_SALU.SLP



Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Sand  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 32

Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 60  
 Rate of Increase -16.67  
 Cv - Minimum 35  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model Undrained ( $\Phi=0$ )  
 Unit Weight 19  
 Cohesion 35

Soil 5  
 Clay 3  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 35  
 Rate of Increase 2.86  
 Cv - Maximum 70  
 Ch/Cv Ratio 1

Till  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Bedrock  
 Soil Model Bedrock



Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-B/G)  
 North Abutment  
 4 metre embankment height, 1.25:1 side slopes  
 NS\_N4H.SLP

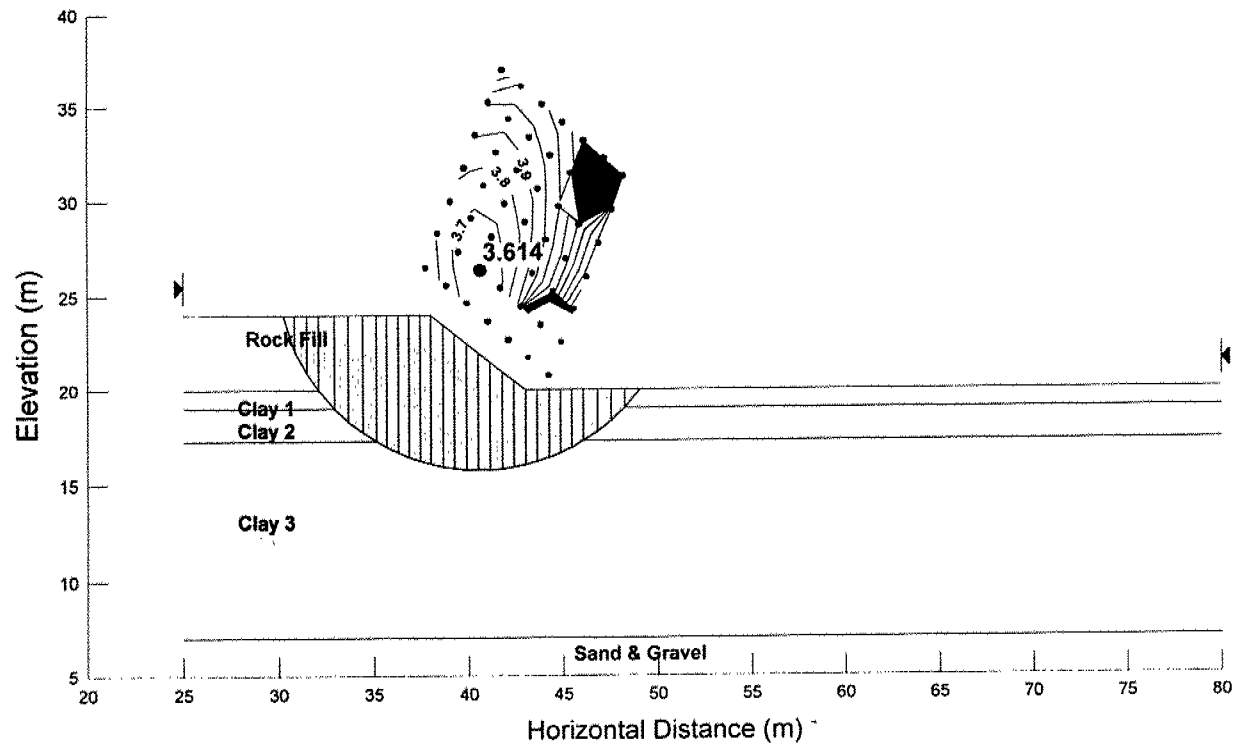
Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 100  
 Rate of Increase -30  
 Cv - Minimum 70  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 70  
 Rate of Increase -28.57  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Clay 3  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 50  
 Rate of Increase 1.95  
 Cv - Maximum 70  
 Ch/Cv Ratio 1

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39



Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-B/G)  
 North Abutment  
 5 metre embankment height, 1.25:1 side slopes  
 NS\_N5H.SLP

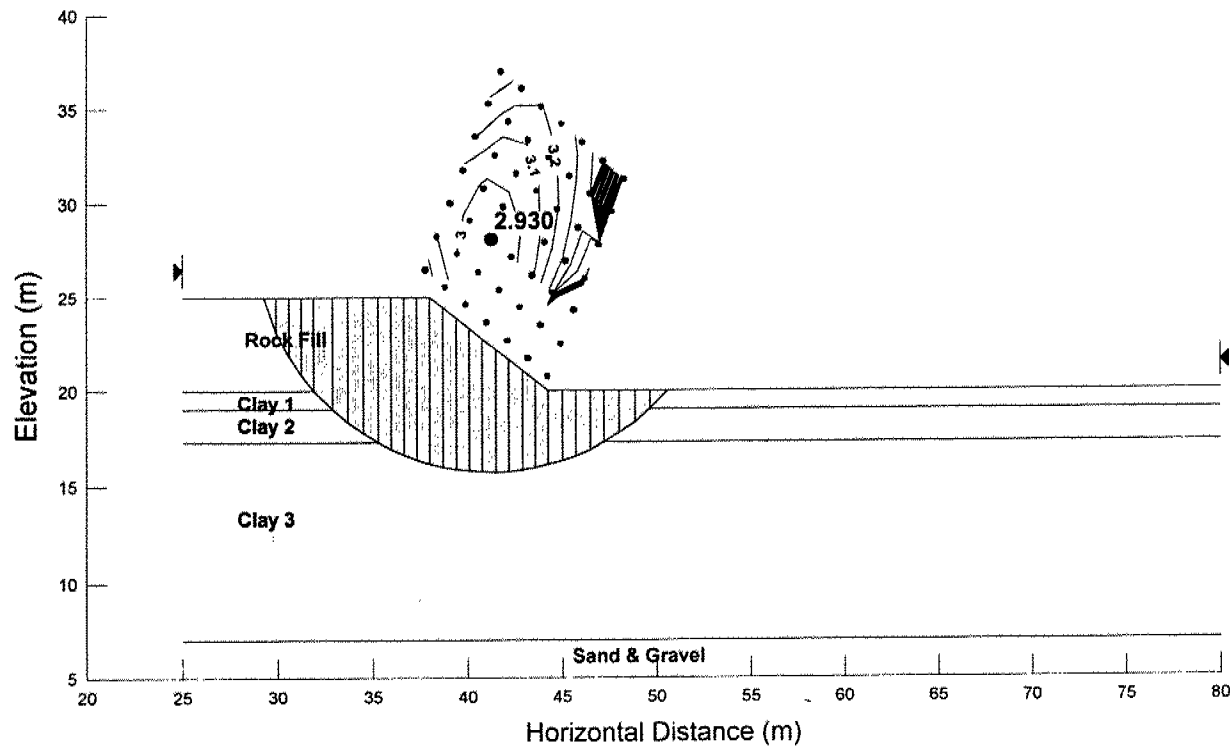
Rock Fill.  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 100  
 Rate of Increase -30  
 Cv - Minimum 70  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 70  
 Rate of Increase -28.57  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Clay 3  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 50  
 Rate of Increase 1.95  
 Cv - Maximum 70  
 Ch/Cv Ratio 1

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39



Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-B/G)  
 North Abutment  
 6 metre embankment height, 1.25:1 side slopes  
 NS\_N6H.SLP

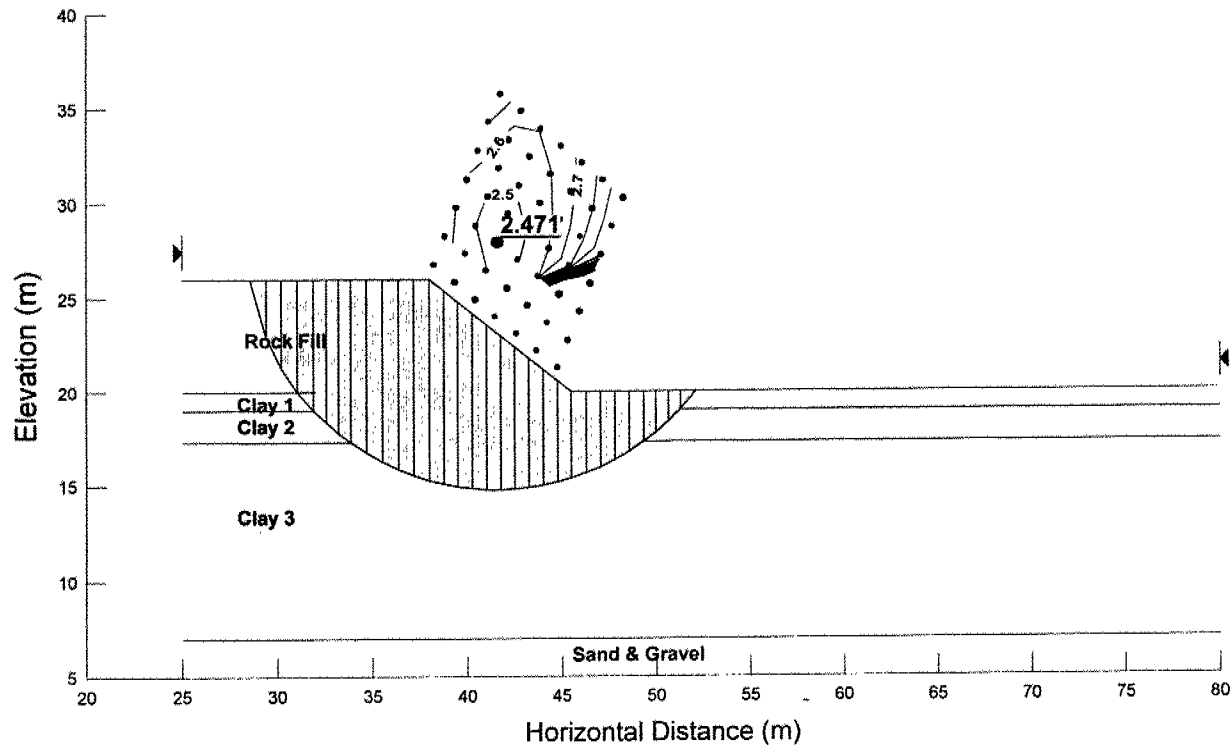
Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 100  
 Rate of Increase -30  
 Cv - Minimum 70  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 70  
 Rate of Increase -28.57  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Clay 3  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 50  
 Rate of Increase 1.95  
 Cv - Maximum 70  
 Ch/Cv Ratio 1

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39



Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-B/G)  
 North Abutment  
 7 metre embankment height, 1.25:1 side slopes  
 4 metre high, 2 metre wide bench  
 NS\_N7H.SLP

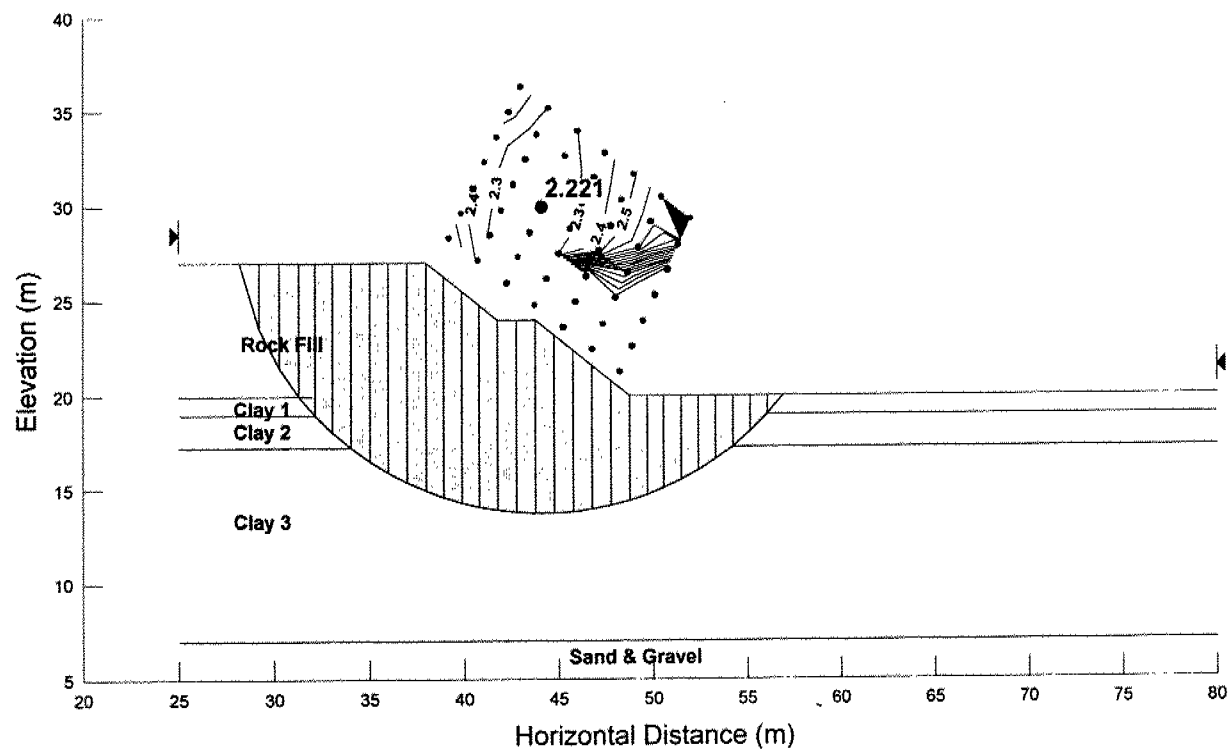
Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 100  
 Rate of Increase -30  
 Cv - Minimum 70  
 Ch/Cv Ratio 1

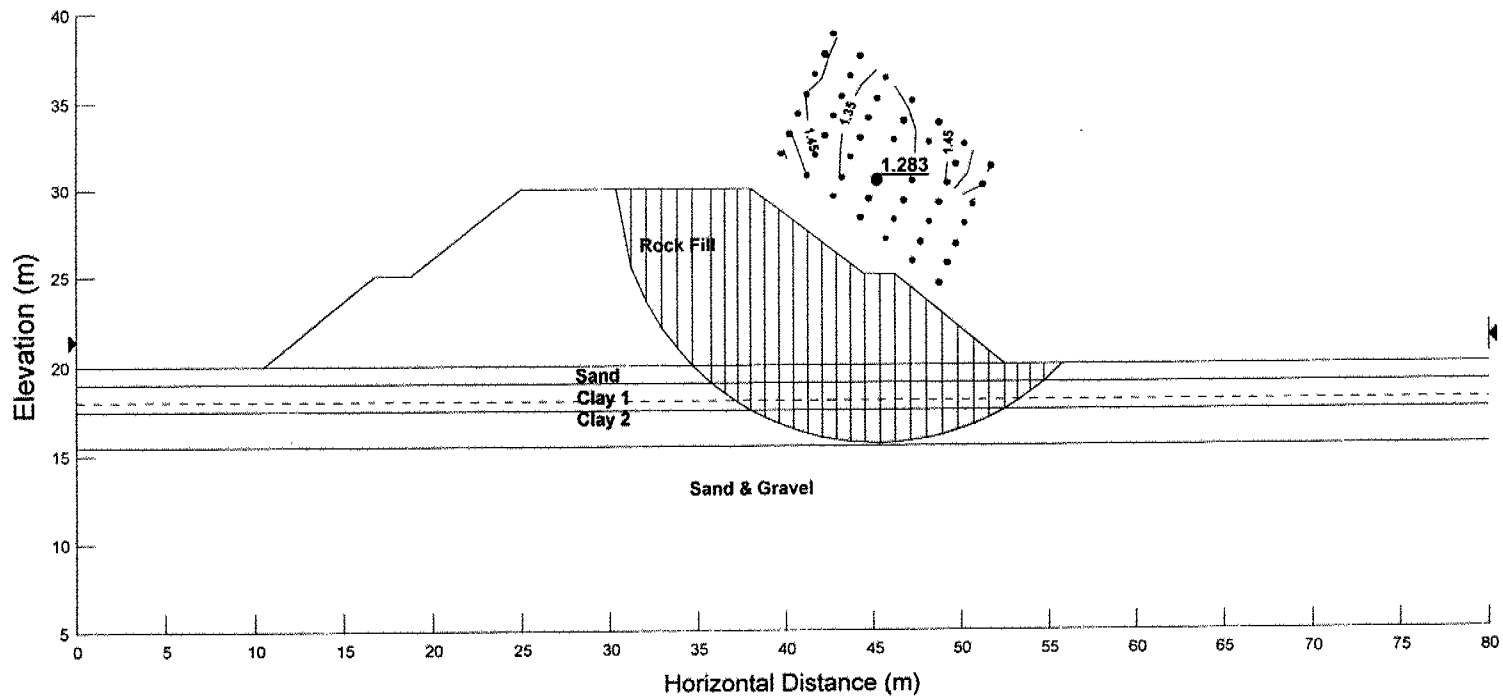
Clay 2  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 70  
 Rate of Increase -28.57  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Clay 3  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 50  
 Rate of Increase 1.95  
 Cv - Maximum 70  
 Ch/Cv Ratio 1

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39



Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-B/G)  
 Northbound Lane, South Abutment  
 10 metre embankment height, 1.25:1 side slopes  
 5 metre high, 2 metre long bench  
 N\_S10H.SLP



Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

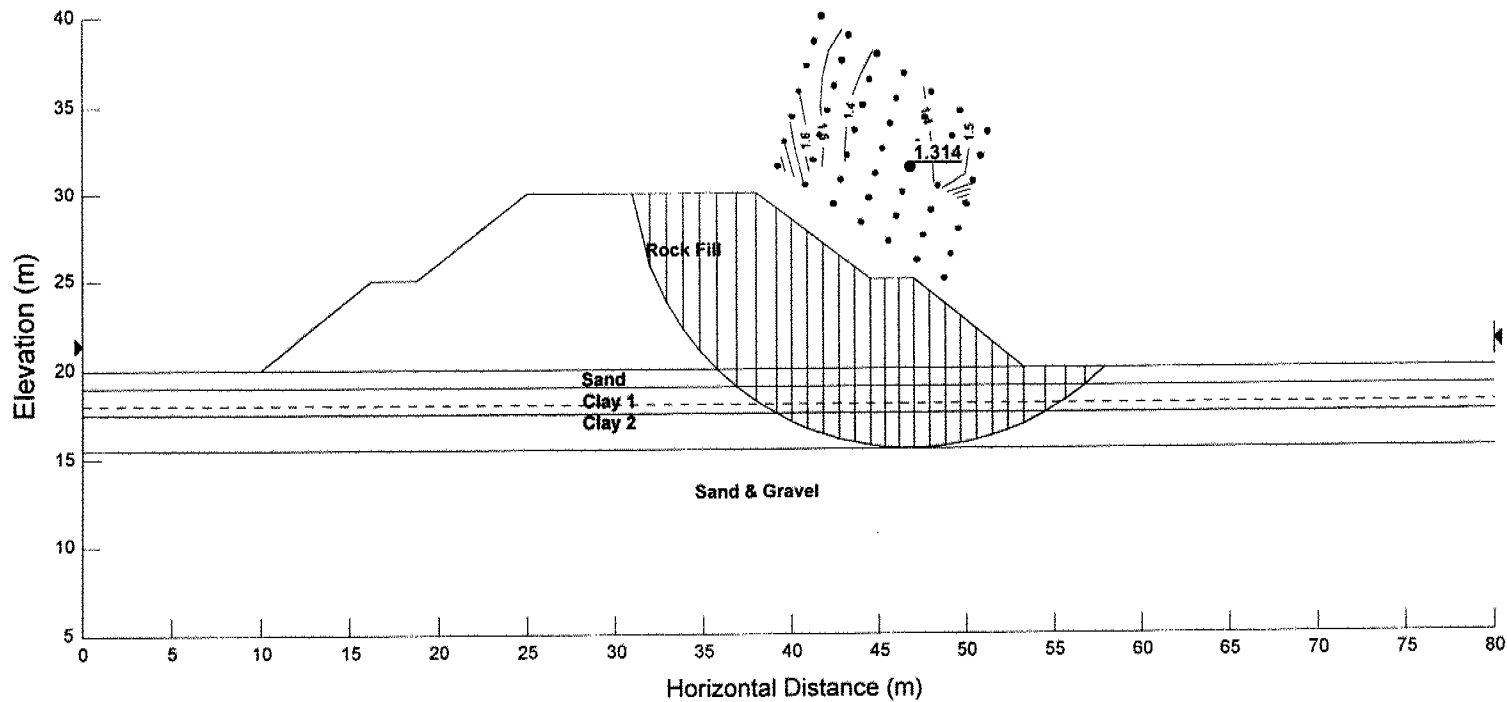
Sand  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 32

Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 60  
 Rate of Increase -16.67  
 Cv - Minimum 35  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 19  
 Cohesion 35  
 Phi 0

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39

Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-B/G)  
 Northbound Lane, South Abutment  
 10 metre embankment height, 1.25:1 side slopes  
 5 metre high, 2.5 metre long bench  
 N\_S10H1.SLP



Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

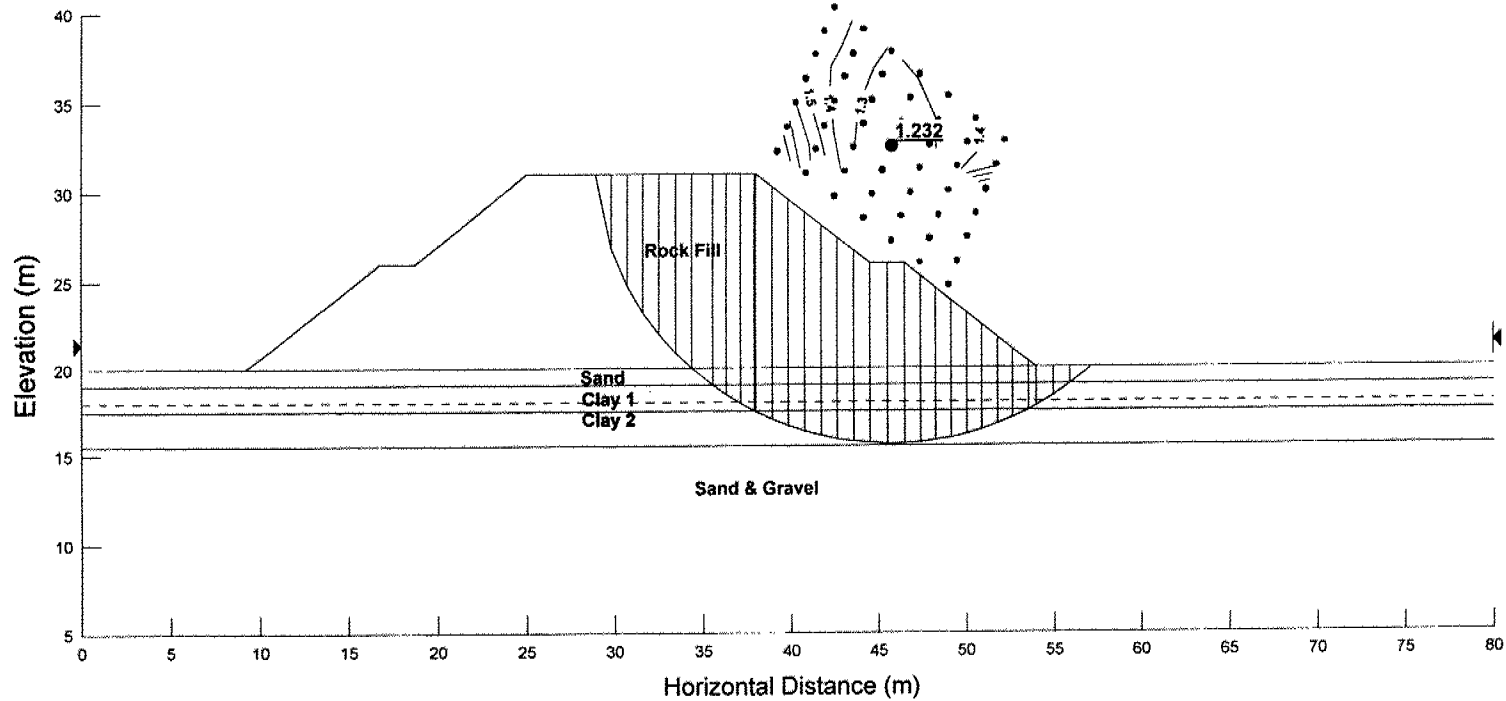
Sand  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 32

Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 19  
 Cv 60  
 Rate of Increase -16.67  
 Cv - Minimum 35  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 19  
 Cohesion 35  
 Phi 0

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39

Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-B/G)  
 Northbound Lane, South Abutment  
 11 metre embankment height, 1.25:1 side slopes  
 6 metre high, 2 metre long bench  
 N\_S11H.SLP



Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

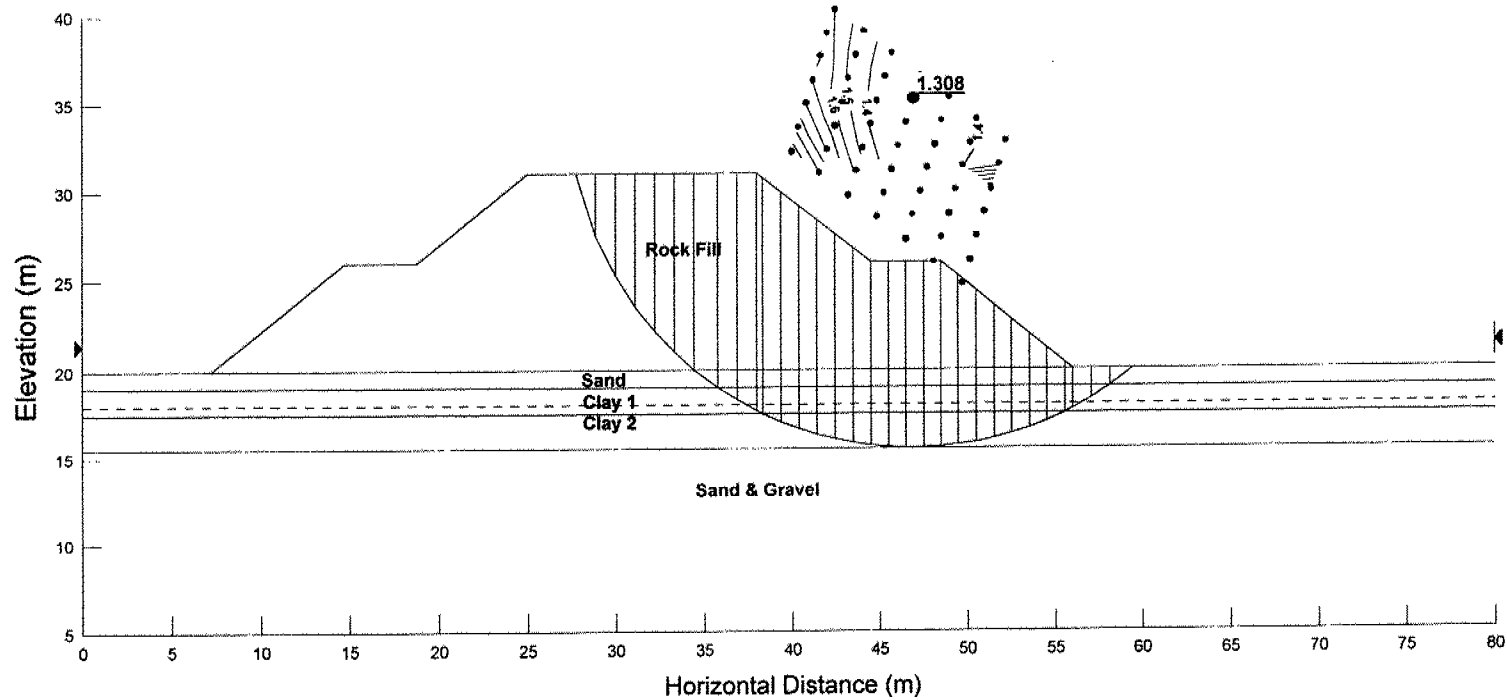
Sand  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 32

Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 19  
 Cv 60  
 Rate of Increase -16.67  
 Cv - Minimum 35  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 19  
 Cohesion 35  
 Phi 0

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39

Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-B/G)  
 Northbound Lane, South Abutment  
 11 metre embankment height, 1.25:1 side slopes  
 6 metre high, 4 metre long bench  
 N\_S11H1.SLP



Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Sand  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 32

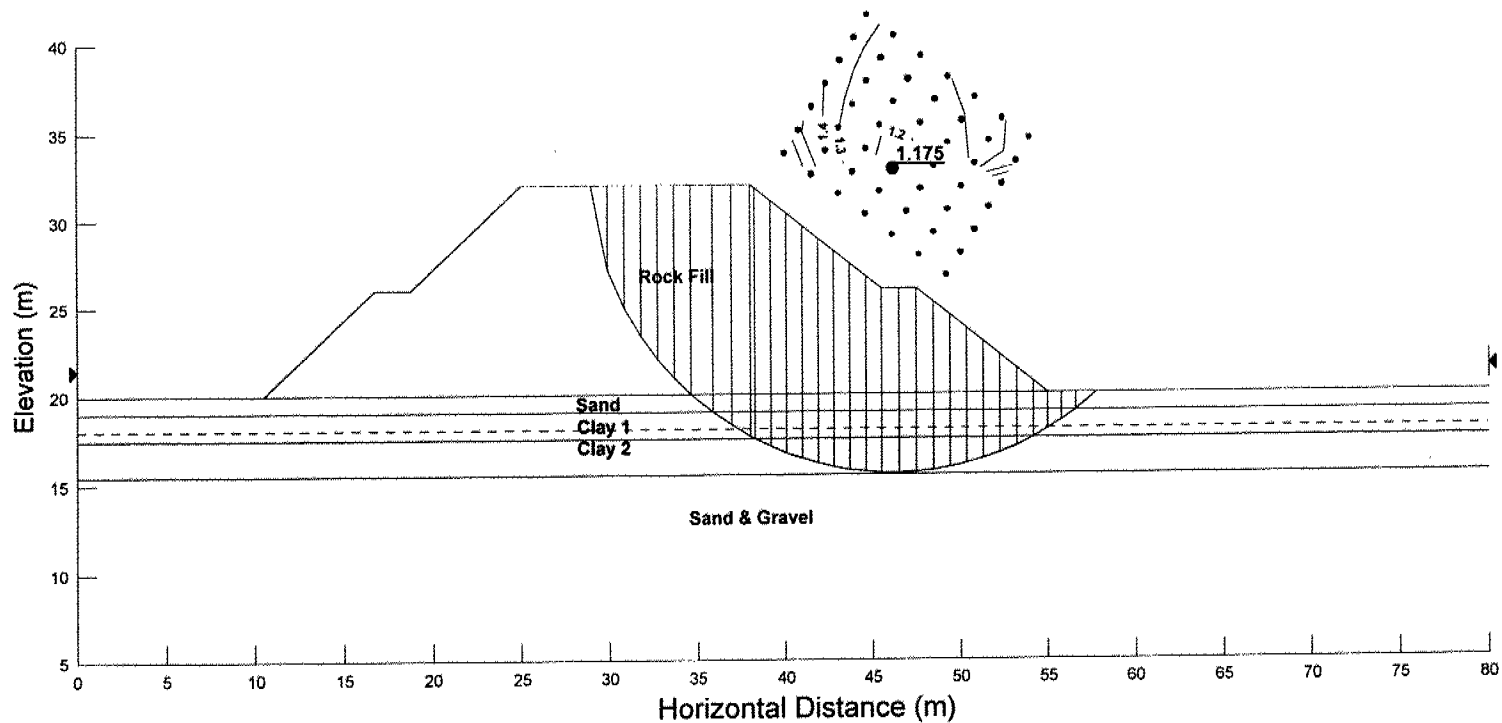
Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 19  
 Cv 60  
 Rate of Increase -16.67  
 Cv - Minimum 35  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 19  
 Cohesion 35  
 Phi 0

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39



Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-B/G)  
 Northbound Lane, South Abutment  
 12 metre embankment height, 1.25:1 side slopes  
 6 metre high, 2 metre long bench  
 N\_S12H.SLP



Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

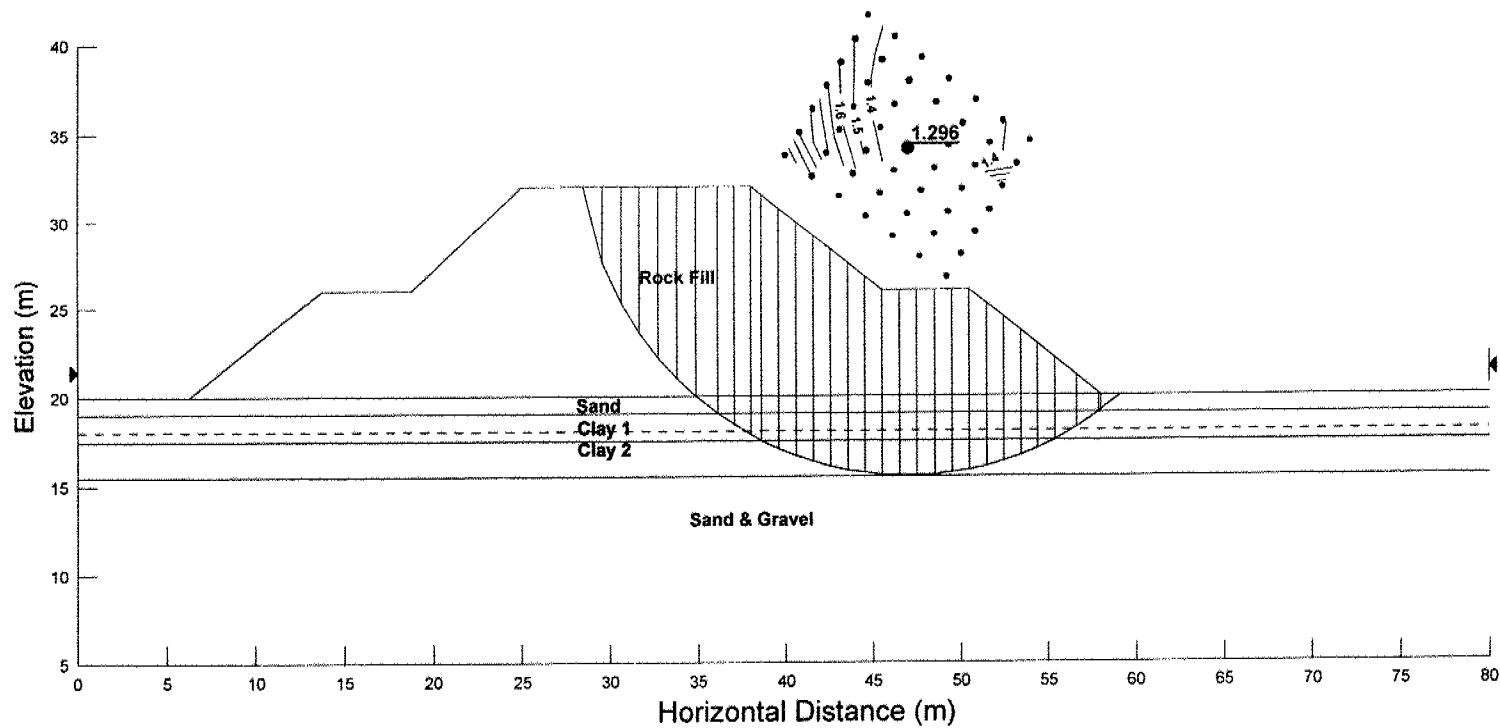
Sand  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 32

Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 60  
 Rate of Increase -16.67  
 Cv - Minimum 35  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 19  
 Cohesion 35  
 Phi 0

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39

Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-B/G)  
 Northbound Lane, South Abutment  
 12 metre embankment height, 1.25:1 side slopes  
 6 metre high, 5 metre long bench  
 N\_S12H1.SLP



Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Sand  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 32

Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 19  
 Cv 60  
 Rate of Increase -16.67  
 Cv - Minimum 35  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 19  
 Cohesion 35  
 Phi 0

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39

Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-B/G)  
 Northbound Lane, South Abutment  
 13 metre embankment height, 1.25:1 side slopes  
 5 metre high and 4 metre high, 2 metre long benches  
 N\_S13H.SLP

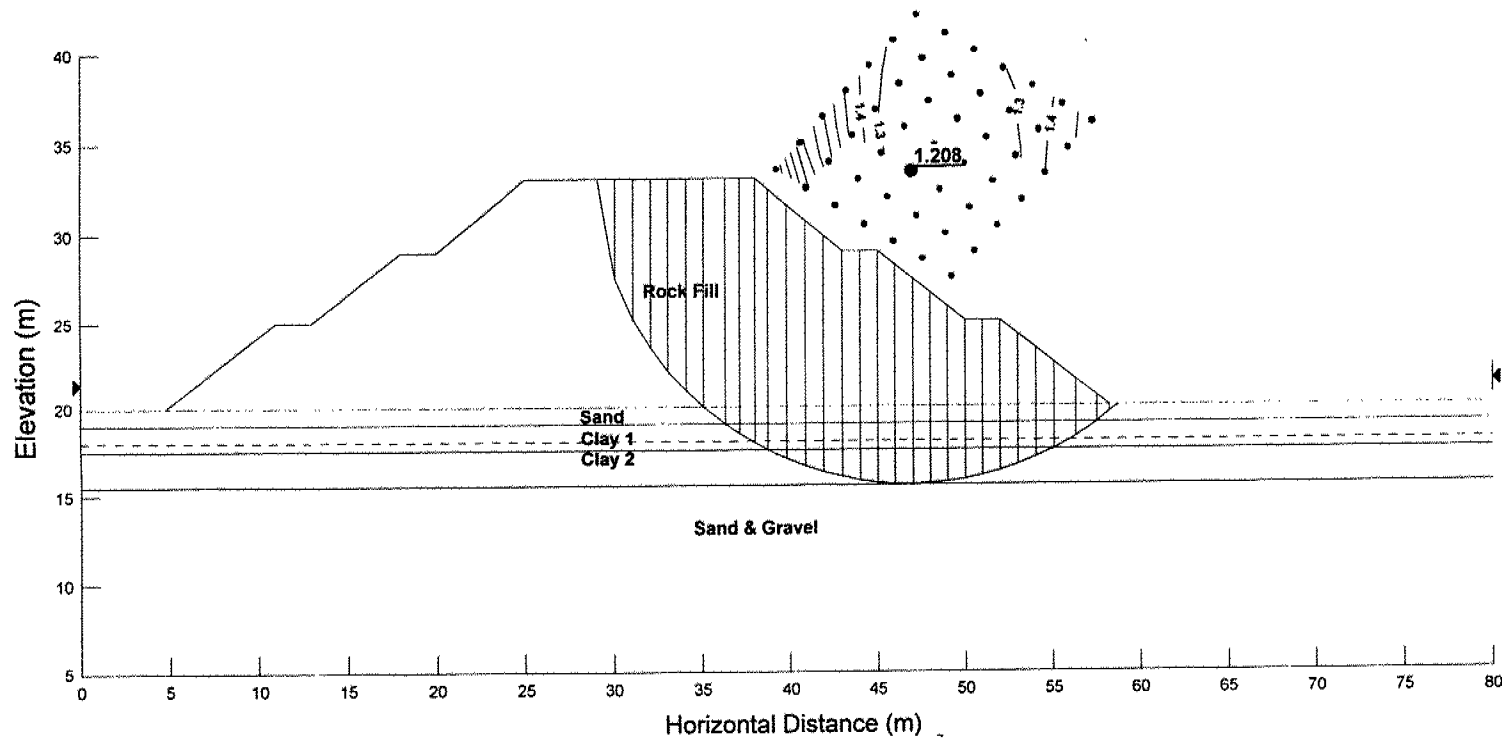
Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Sand  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 32

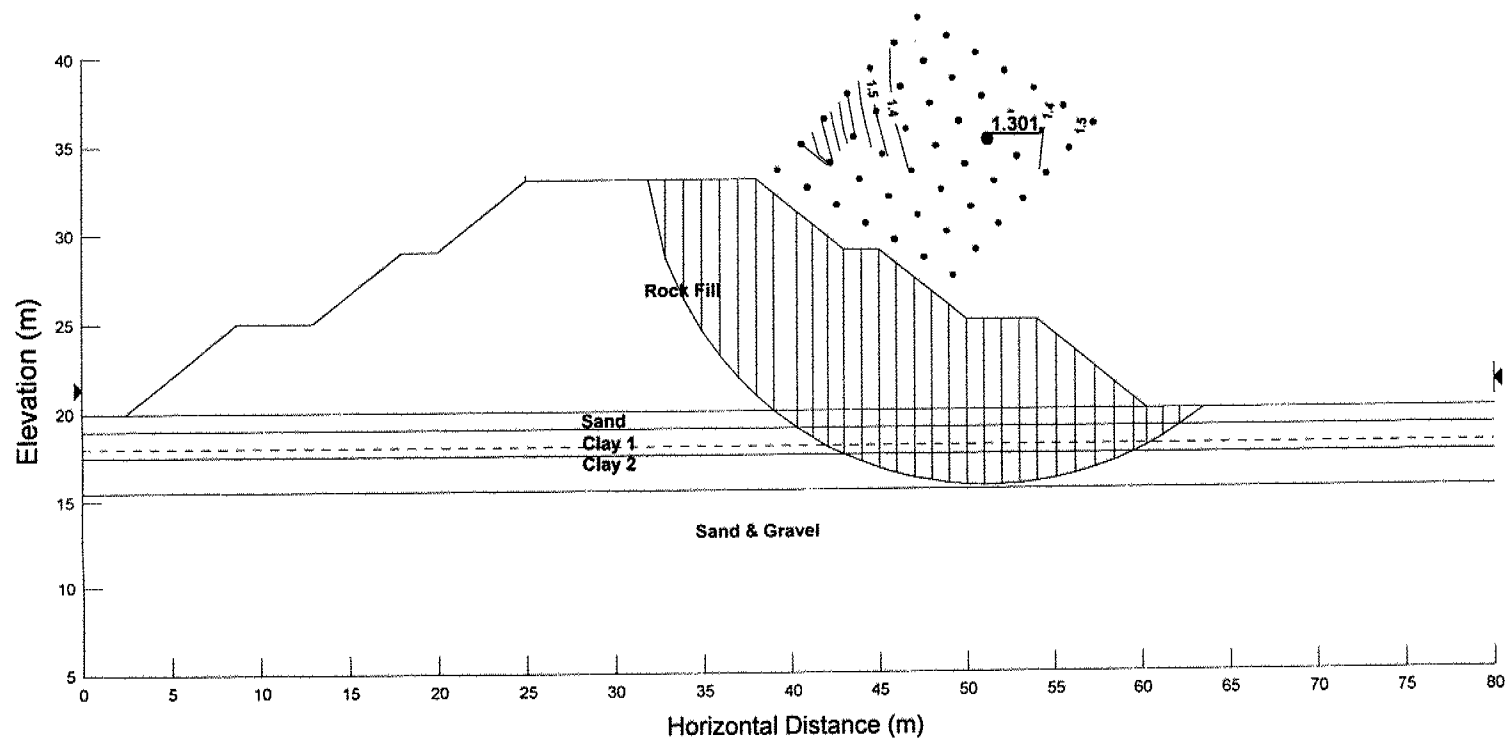
Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 19  
 Cv 60  
 Rate of Increase -16.67  
 Cv - Minimum 35  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 19  
 Cohesion 35  
 Phi 0

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39



Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-B/G)  
 Northbound Lane, South Abutment  
 13 metre embankment height, 1.25:1 side slopes  
 5 metre high, 4 metre long and 4 metre high, 2 metre long benches  
 N\_S13H1.SLP



Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Sand  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 32

Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 60  
 Rate of Increase -16.67  
 Cv - Minimum 35  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 19  
 Cohesion 35  
 Phi 0

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39

G.I.-30 SEPT. 1976

GEOCRES No. \_\_\_\_\_

DIST. 54 REGION \_\_\_\_\_

W.P. No. 774-93-00

CONT. No. \_\_\_\_\_

W. O. No. \_\_\_\_\_

STR. SITE No. \_\_\_\_\_

HWY. No. 11

LOCATION Trout CREEK By-PASS

North Bound & South Bound LANES

No of PAGES -           

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. \_\_\_\_\_

REMARKS: \_\_\_\_\_

\_\_\_\_\_

\_\_\_\_\_

\_\_\_\_\_

**Geotechnical Foundation Report  
Embankment Section  
Trout Creek ByPass  
North Bound & Sound Bound Lanes  
Station 12+350 to 12+850  
GWP No. 774-93-00  
District 54, Sudbury**

Prepared for:

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Thornhill, ON  
L3T 7N4

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SO7524G/IF  
March, 1999

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**SO7524G/IF**

**MEMORANDUM NO. 2**

**TO:** R.D. Kivi, P.Eng.  
Senior Project Manager  
Marshall, Macklin, Monaghan

**FROM:** I.W. Gore, P.Eng.  
Principal Engineer  
Trow Consulting Engineers Ltd.

E.A. Gonneau, P.Eng.  
Project Manager  
Trow Consulting Engineers Ltd.

**SUBJECT:** Geotechnical Foundation Report  
Embankment Section  
Trout Creek ByPass, North Bound & South Bound Lanes  
Station 12+350 to 12+850  
GWP No. 774-93-00  
District 54 Sudbury

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This memorandum addresses the geotechnical report for the design and construction of an embankment section between approximate stations 12+350 to 12+850 along the proposed NBL and SBL of the proposed Trout Creek ByPass, Highway 11, as part of GWP 774-93-00. The approximately 500 m long embankment is located from the existing McCarthy Street, extending southwesterly to the section where the existing ground rises, and the ByPass enters a "cut" section.

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## 1.0 Introduction

This embankment section for the NBL and SBL of the proposed Trout Creek ByPass, Highway 11, runs from approximate station 12+350 up to approximate station 12+850, a distance of 500 m.

In the initial section, i.e. between stations 12+600 to 12+850, the terrain is flat and contains many tag-alders within the existing ground level at approximate elevation 315 m. In this area, the embankment is approximately 4 m high at station 12+800, rising to 8 m high at station 12+600. Beyond station 12+850, up to the present McCarthy Street at station 13+750, the proposed embankment height is less than 4 m, and the terrain is flat and open.

The existing ground is level, southwesterly from station 12+600, then rises gradually from the flat land at elevation 315 m through densely wooded terrain, up to approximate elevation 324 m at station 12+470. Correspondingly, the proposed NBL and SBL embankment grades also rise to elevation 327 m at this location. Southwesterly from station 12+325, the existing grade then rises steeply, and the proposed NBL and SBL enter into cut, at approximate station 12+300.

## 2.0 Field Work

The field work comprised thirteen (13) sampled boreholes (BH's 1IF to 13IF, inclusive), together with five additional dynamic cone penetration tests driven adjacent to boreholes 1IF, 6IF, 11IF, 12IF and 13IF. The locations of the boreholes are included on the attached site plan, Drawing 1, and on the individual borehole logs. The drilling was completed using a track-mounted soils drill equipped with hollow and solid stem flight augers between September 14 to 17, 1998, inclusive.

Details of the soil strata encountered in the boreholes and cones are included in the attached logs, and plotted on the profile included on Drawing 1. Further information on soil descriptions are contained on Drawings 2A and 2B.

### 3.0 Subsurface Conditions

#### 3.1 Station 12+600 to 12+850 (BH's 5IF to 12IF)

Based on the borehole data, the subsoil conditions along this flat terrain portion are reasonably uniform and consist of the following soil strata:

- *Organics*

The organics comprise a thin, surficial deposit of topsoil generally 100 mm to 300 mm thick. At borehole 11IF, however, the topsoil veneer is locally thicker, i.e. 600 mm.

- *Sand*

The predominant soil stratum throughout this section consists of sand, which has a generally loose to compact consistency. The deposit is at least 6 m thick.

Based on the laboratory gradation analyses (see attached data), the sand is predominantly fine-grained with less than 10% silt and clay sizes.

Typical standard penetration test results ("N" values) vary from 2 to 27 (average 12). The lower "N" values (less than 5) are not considered to be representative, since sampling disturbance likely occurred during the drilling and in-situ testing. It is noted that the dynamic cone penetration tests (cones driven adjacent to borehole 6IF, 11IF, 12IF and 13IF) confirm consistent blows, exceeding 20 blows/300 mm penetration.

Moisture contents of the fine sand are in the range of 18% to 22%.

- *Clayey Silt*

A localized deposit of clayey silt was encountered underlying the sand in boreholes 6IF and 11IF.

In borehole 6IF, the clayey silt deposit is less than 2.5 m thick and occurs at depths between 6.5 m (~El. 308.5 m) and 8.8 m (~El. 300.2 m). In borehole 11IF, the deposit is slightly thicker, i.e. 3.6 m

thick, and occurs at depths between 5.5 m (~El. 310.4 m) and 9.1 m (~El. 306.8 m). The clayey silt is stratified with intermittent, horizontal silt layers and undrained shear strengths (as measured with in-situ vanes and laboratory shear tests) varying from 22 kPa to 30 kPa.

Natural moisture contents range from 40% to 55%, with Atterberg limits of 22% (plastic limit) to 38% and 42% (liquid limit).

The clayey silt is similar in characteristics and properties to the clayey deposit encountered at the adjacent North Interchange. For the North Interchange, extensive detailed laboratory analyses have been undertaken.

- *Silt*

A lower zone of loose to compact silt was encountered in borehole 11IF below the clayey silt. This silt stratum was intercepted at a depth of 9.2 m (~El. 306.8 m) and continues for approximately 4.5 m, i.e. down to a depth of 13.7 m (~El. 302.2 m). Standard penetration blows varied from 3 to 9 blows/300 mm, indicating a loose to compact consistency. Dynamic cone blows, through the deposit, exceed 40 blows/300 mm penetration. Laboratory shear strength tests and a field vane test (completed in "clayey" seams) confirmed undrained shear strength values exceeding 45 kPa. In-situ moisture contents of the deposit are in the range 25% to 35% with one Atterberg limit test confirming a plastic limit of 18% and a liquid limit of 28%.

- *Silty Sand and Gravel Till*

A 2 m thick zone of dense silty sand and gravel till ("N" blows >40) was encountered at the base of borehole 5IF, i.e., below the upper compact sand.

### **3.2 Stations 12+350 to 12+600 (BH's 1IF to 4IF)**

As the ground rises in this section, the upper veneer of sand is thinner and the lower levels of the boreholes encountered a dense glacial till. The properties and sequence of soil deposits is described below:

- *Organics*

The organics in this section are thin and comprise less than 100 mm of topsoil.

- *Sand*

The sand is generally a fine-grained deposit with traces of silt and occasional gravel inclusions. The thickness of the sand varies from less than 2.5 m in boreholes 3IF and 4IF to 5.4 m in borehole 1IF.

Standard penetration tests established "N" blows of 6 to 16, confirming a loose to compact condition. In-situ moisture contents are less than 20%.

- *Clayey Silt*

A localized, approximately 1.5 m thick, pocket of clayey silt was intercepted in borehole 1IF at a depth of 5.4 m (~El. 317.6 m). The clayey silt has the following characteristics:

- undrained shear strength ~50 kPa (laboratory shear test)
- in-situ moisture content - 36%
- Atterberg limits      Plastic Limit - 21%  
                                 Liquid Limit - 33%
- "N" value - 5 blows/300 mm
- dynamic cone penetration blows - >20 blows/300 mm

- *Sand and Gravel Till*

Sand and gravel till was encountered at the base of the boreholes and varies in thickness from about 1.0 m in borehole 3IF to slightly greater than 3.5 m thick in borehole 2IF.

Standard penetration blows ("N" values) are in the range of 15 to 50 blows/300 mm penetration (average 30 blows), with in-situ moisture contents of 8% to 20%.

## 4.0 Groundwater

In the embankment portion where the existing terrain is flat and poorly drained (boreholes 5IF to 13IF, stations 12+600 to 12+850), the groundwater table is at grade.

In the higher terrain (boreholes 1IF to 4IF, station 12+350 to 12+600), the groundwater table is slightly lower, i.e. at a depth of 1 m to 2 m below grade.

Seasonal fluctuations in the level of the groundwater table can be expected.

## 5.0 Recommendations

The proposed NBL and SBL will require embankment heights of up to about 8 m.

Since the majority of the subsoil is competent, granular soils (loose to compact sands and/or dense till), no stability nor long-term consolidation problems are envisaged. However, where the localized clayey silt deposit was encountered, estimated to be within a localized section beneath the NBL and SBL between stations 12+615 and 12+645, some precautionary measures must be considered. In these sections, the following procedures are recommended:

- ***Stability***

The embankment height(s) in this localized section is 7.5 m. The soil strata consists of 5.5 m of compact sand over 3.6 m of loose to compact, clayey silt. The undrained shear strength of the clayey silt is 22 kPa (minimum). Although the factor of safety, under these conditions, meets or exceeds 1.3, (see stability analysis, Figure 4 to 6), it is nevertheless recommended that the embankment be constructed gradually, increasing the height over a minimum of 3 months.

- ***Settlement***

The calculated settlement of the embankments, due to consolidation of the 3.7 m thick zone of clayey silt, is estimated to be in the order of 150 mm. However, 50% of this settlement will be completed within 3 months and 90% within a period of 7 months, after full height has been reached. As such,

all the anticipated consolidation settlement should occur within 8 months, once the embankment has been constructed to full height.

## 6.0 Embankment Design

The proposed embankments are 8 m in height or less. Since only fairly minor consolidation settlements are expected, i.e. a maximum of 150 mm within one localized section, which, in any event, should occur within 8 months after construction, it is recommended that the embankments be constructed either with rock fill having a side slope of 1.25H:1V, allowing for a 2.0 m wide, mid-height bench where the height exceeds 6 m, or the bouldery material contained within the cut area to the south of Trout Creek, having a side slope of 2.5H:1V, allowing for a 3.0 m wide, mid-height bench.

Settlement within the rock fill or till fill embankment itself should be expected over the first 2 to 3 years after construction. It is likely that this movement could be in the order of 1% to 2% of the height, i.e. up to about 150 mm beneath the 8 m high embankment section. This settlement could, however, be reduced significantly (to less than 50 mm) if the rock fill was restricted in size to a maximum of 600 mm diameter and then thoroughly compacted, using a 10 ton vibratory drum roller, with four or five passes in 1 m lifts of rock fill.

To allow for future potential grade raises, the rock fill embankments should be constructed a minimum of 1 m wider than standard on each side, to the bottom of the subgrade, in the section between stations 12+615 and 12+645.

## 7.0 Construction Considerations

The upper organic veneer 200 mm (average) of topsoil should be stripped off down to firm bottom before placement of the fill.

## 8.0 Closure

The field investigations were supervised by Mr. E.A. Gonneau, P.Eng., Project Manager. The memorandum report was written by Mr. I.W. Gore, M.Sc., P.Eng., Principal Geotechnical Engineer, and reviewed by Mr. S.E. Gonsalves, P.Eng.

Yours truly,

**TROW CONSULTING ENGINEERS LTD.**

  
I.W. Gore, M.Sc., P.Eng.  
Principal Engineer

  
S.E. Gonsalves, P.Eng.  
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Project Manager

Encl.  
Dist:

**NOTES ON SAMPLE DESCRIPTIONS**

1. All descriptions included in this report follow the I.S.S.M.F.E. as suggested in the Canadian Foundation Manual. The laboratory grain-size analysis also follows this classification system. Others may designate the unified classification system as their source; a comparison of the two is shown for your information. Please note that, with the exception of those samples where the grain-size analysis has been carried out, all samples are classified visually and the accuracy of visual examination is not sufficient to differentiate between the classification systems or exact grain sizing.

UNIFIED SOIL CLASSIFICATION	Fines (silt or clay)			Sand			Gravel			Cobbles	
				Fine	Medium	Coarse	Fine	Coarse			
I.S.S.M.F.E. SOIL CLASSIFICATION	Clay	Silt			Sand			Gravel			Cobbles
		Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	
Sieve Sizes											
<div><div>0.0010.0020.0030.0040.0060.0080.010.020.030.040.060.080.10.20.30.40.60.81.02.03.04.060.8102030406080</div><div>200401043/43in</div></div>											
Particle Size (mm)											

2. **FILL:** Where fill is designated on the borehole log, it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of the site fill material. All fills should be expected to contain obstructions such as large concrete pieces of subsurface basements, floors, tanks, etc.; none of these may have been encountered in the borehole. Since boreholes cannot accurately define the contents of fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact and correct composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant on-going and future settlements. Some fill material may be contaminated by toxic waste that renders it unacceptable for deposition in any but designated land fill sites. Unless specifically stated, the fill on this site has not been tested for contaminants that may be considered hazardous. This testing and a potential hazard study can be carried out if you so request. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common but are not detectable using conventional geotechnical procedures.
3. **TILL:** The term till on the borehole logs indicate that the material originates from a geological process associated with glaciation. As a result of this geological process, the till must be considered heterogeneous in composition and, as such, may contain pockets and/or seams of material such as sand, gravel silt or clay. As till often contains cobbles (60 to 200 mm) or boulders (over 200 mm), contractors may encounter them during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size, or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited areas; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till material.



## NOTES ON SAMPLE DESCRIPTIONS (Cont'd)



Project No: S07524G/IP

Drawing No: 2B

4. The following table gives a description of the soil based on particle sizes. With the exception of those samples where grain-size analyses have been performed, all samples are classified visually. The accuracy of visual examination is not sufficient to differentiate between this classification system or exact grain size.

Soil Classification		Terminology	Proportion
Clay	< 0.002 mm	"trace" (eg. trace sand)	1% - 10%
Silt	0.002 to 0.06 mm	"some" (eg. some sand)	10% - 20%
Sand	0.06 to 2 mm	adjective (eg. sandy)	20% - 35%
Gravel	2 to 60 mm	and (eg. and sand)	> 35%
Cobbles	60 to 200 mm	noun (eg. boulders)	> 35% and
Boulders	> 200 mm		main fraction

Classification system as suggested in the Canadian Foundation Engineering Manual, 3rd Edition, unless otherwise noted.

The compactness of cohesionless soils and the consistency of cohesive soils are defined by the following:

Cohesionless Soil		Cohesive Soil	
Compactness	Standard Penetration Resistance "N" Blows/0.3 m	Consistency	Undrained Shear Strength (kPa)
Very Loose	0 to 4	Very Soft	< 12
Loose	4 to 10	Soft	12 - 25
Compact	10 to 30	Firm	25 - 50
Dense	30 to 50	Stiff	50 - 100
Very Dense	Over 50	Very Stiff	100 - 200
		Hard	> 200

### 5. Rock Coring

Where rock drilling was carried out, the term RQD (Rock Quality Designation) is used. The RQD is an indirect measure of the number of fractures and soundness of the rock mass. It is obtained from the rock cores by summing the length of core recovered, counting only those pieces of sound core that are 100 mm or more in length. The RQD value is expressed as a percentage and is the ratio of the summed core lengths to the total length of core run. The classification based on the RQD value is given below.

RQD Classification	RQD
Very poor quality	< 25
Poor quality	25 - 50
Fair quality	50 - 75
Good quality	75 - 90
Excellent quality	90 - 100

$$\text{Recovery Designation \% Recovery} = \frac{\text{Length of Core Per Run} \times 100}{\text{Total Length of Run}}$$

# RECORD OF BOREHOLE BH-11F 1 OF 1

## METRIC

W.P. 774-93-00      LOCATION Station 12+387, on centreline of Northbound Lane      ORIGINATED BY S.M.  
 DIST 54      HWY 11      BOREHOLE TYPE Standard augers / Dynamic cone      COMPILED BY M.D.  
 DATUM Geodetic      DATE September 14, 1998      CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)		CONE PENETRATION TEST		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	20	40					
323.00 0.00	GROUND SURFACE													
	TOPSOIL, ~50 mm over SAND, fine to medium grading to fine silty sand with depth, brown then grey below ~3.5 m depth, moist then wet below ~2 m depth. (loose to compact)		1	SS	8									0% 92% 8%
			2	SS	16									
			3	SS	13									
317.60 5.40	CLAYEY SILT, grey, wet. (loose)		4	SS	5									
316.00 7.00	SILTY SAND & GRAVEL TILL, brown. (compact)		5	SS	14									
314.92 8.08	END OF BOREHOLE													
	Probable TILL													
313.25 9.75	END OF CONE TEST DUE TO 'BOUNCING' REFUSAL ON PROBABLE BEDROCK OR POSSIBLE BOULDER													
	Notes: 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Water level was at ~2.3 m & hole was open to ~5.1 m depth on completion. 3) Dynamic cone penetration test driven adjacent BH-11F.													



# RECORD OF BOREHOLE BH-2IF 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station ~12+450, on centreline of Northbound Lane ORIGINATED BY S.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / COMPILED BY M.D.  
 DATUM Geodetic DATE September 14, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80				wp — w — wl				
								SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL      FIELD VANE LAB SHEAR				WATER CONTENT (%) 10 20 30 40				
321.30	GROUND SURFACE															
0.00	TOPSOIL, ~75 mm over SAND, fine with SILT content, occasional stiff CLAYEY SILT layers, brown then grey below ~3 m depth, wet. (loose to compact)		1	SS	6											
			2	SS	10											
317.30	SILTY SAND & GRAVEL TILL, some cobble sizes, grey. (compact to dense)		3	SS	27										31% 44% 25%	
4.00			4	SS	44											
313.88	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER															
7.32	Notes: 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Water level was at ~2.0 m & hole was open to ~4.1 m depth on completion.															



# RECORD OF BOREHOLE BH-31F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 12+495, on centreline of Median ORIGINATED BY S.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / COMPILED BY M.D.  
 DATUM Geodetic DATE September 14, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION		
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60	80	wp	w	wl			10	20
320.00	GROUND SURFACE					320												
0.00	TOPSOIL, 50 mm over SAND, fine, brown, moist. (compact)																	
318.60	SAND & GRAVEL TILL, brown, moist. (dense)		1	SS	42	318												
317.71	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER																	
2.29	Notes: 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Borehole was dry & open to 0.8 m depth on completion.																	



# RECORD OF BOREHOLE BH-4IF 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station ~12+550, on centreline of Median ORIGINATED BY S.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / COMPILED BY M.D.  
 DATUM Geodetic DATE September 14, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				WATER CONTENT (%)				UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER TYPE			BLOWS/0.3m	20	40	60	80	wp	w	wl		
318.50 0.00	<b>GROUND SURFACE</b>														
	TOPSOIL, ~100 mm over SAND, fine, trace of SILT, brown to grey, wet at base. (loose to compact)														
316.50 2.00	<b>SILTY SAND &amp; GRAVEL TILL</b> cobbles & possible boulders, grey. (compact to dense)		1	SS	7										0% 77% 23%
			2	SS	27										
			3	SS	48										
312.75 5.75	<b>END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER</b>														
	<b>Notes:</b> 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Water level was at ~2.4 m & hole was open to ~3.1 m depth on completion.														



# RECORD OF BOREHOLE BH-51F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 12+600, on centreline of Median  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers /  
 DATUM Geodetic DATE September 14, 1998  
 ORIGINATED BY S.M.  
 COMPILED BY M.D.  
 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20   40   60   80				wp   ——— w ——— wl				
								SHEAR STRENGTH: Cu, KPa ● UNCONFINED QUICK TRIAXIAL   × FIELD VANE LAB SHEAR				WATER CONTENT (%) 10   20   30   40				
							20	40	60	80	10	20	30	40		
316.40	GROUND SURFACE															
0.00	TOPSOIL, ~100 mm over SAND, fine, trace of SILT, brown, moist to wet. (compact to dense)															
			1	SS	27											
313.90	SILTY SAND & GRAVEL TILL, grey. (dense)															
2.50			2	SS	48											
311.98	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER															
4.42	Notes: 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Water level was at ~0.6 m & hole was open to ~2.2 m depth on															



# RECORD OF BOREHOLE BH-6IF 1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station ~12+650, on centreline of Median

ORIGINATED BY S.M.

DIST 54 HWY 11

BOREHOLE TYPE Standard augers / Dynamic cone

COMPILED BY M.D.

DATUM Geodetic

DATE September 17, 1998

CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR		
315.00	GROUND SURFACE										
0.00	PEATY TOPSOIL, ~200 mm over SAND, mostly fine with a trace of SILT & occasional GRAVEL sizes, grey brown, wet. (loose to compact)		1	SS	7						
			2	SS	13						
			3	SS	2						
308.50	CLAYEY SILT, grey, wet. (loose)		4	SS	0						
6.50			5	SS	4						
306.16	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER										
8.84	Notes: 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Water level was at ~0.2 m & hole was open to ~4.0 m depth on completion. 3) Dynamic cone penetration test driven adjacent BH-6IF.										



# RECORD OF BOREHOLE BH-71F 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station ~12+700, on centreline of Median ORIGINATED BY S.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / COMPILED BY M.D.  
 DATUM Geodetic DATE September 17, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	20	40	60	80	wp		
315.00	GROUND SURFACE				315									
0.00	PEATY TOPSOIL, ~300 mm over SAND, fine with a trace of SILT content, grey, wet. (loose to compact)													
			1	SS	4									
			2	SS	17									
			3	SS	20									
			4	SS	27									
308.45	END OF BOREHOLE													
6.55	Notes: 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Water level was at ~0.1 m & hole was open to ~1.8 m depth on completion.													





# RECORD OF BOREHOLE BH-8IF 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station ~12+750, on centreline of Median ORIGINATED BY S.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / COMPILED BY M.D.  
 DATUM Geodetic DATE September 17, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20    40    60    80				wp ——— w ——— wl				
								SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)				
315.00	GROUND SURFACE															
0.00	PEATY TOPSOIL, ~300 mm over SAND, mostly fine with a trace of SILT, brown to grey, wet. (compact)															
			1	SS	10											
			2	SS	15											
			3	SS	13											
			4	SS	26											
308.45	END OF BOREHOLE															
6.55	Notes: 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Water level was at ~0.1 m & hole was open to ~2.6 m depth on completion.															



# RECORD OF BOREHOLE BH-9IF 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station ~12+800, on centreline of Median ORIGINATED BY S.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / COMPILED BY M.D.  
 DATUM Geodetic DATE September 17, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION					
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60	80	wp	w	wl			10	20	30	40	kN/m <sup>3</sup>
315.00	GROUND SURFACE																				
0.00	TOPSOIL, ~150 mm over SAND, mostly fine with a trace of SILT, grey. (loose to compact)																				
			1	SS	5																
			2	SS	16																
			3	SS	14																
			4	SS	12																
308.45	END OF BOREHOLE																				
6.55	Notes: 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Water level was at ~0.5 m & hole was open to ~1.9 m depth on completion.																				



# RECORD OF BOREHOLE BH-10IF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station +12+850, on centreline of Median ORIGINATED BY S.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / COMPILED BY M.D.  
 DATUM Geodetic DATE September 17, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60	80	wp	w		
314.80	GROUND SURFACE														
0.00	PEATY TOPSOIL, ~300 mm over SAND, mostly fine with a trace of SILT, brown to grey, wet below ~0.6 m depth. (compact)		1	SS	8										0% 97% 3%
			2	SS	15										
			3	SS	14										
			4	SS	12										
308.25	END OF BOREHOLE														
6.55	Notes: 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Water level was at ~0.7 m & hole was open to ~2.3 m depth on completion.														



# RECORD OF BOREHOLE BH-11IF 1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station ~12+630, on centreline of Southbound Lane

ORIGINATED BY S.M.

DIST 54 HWY 11

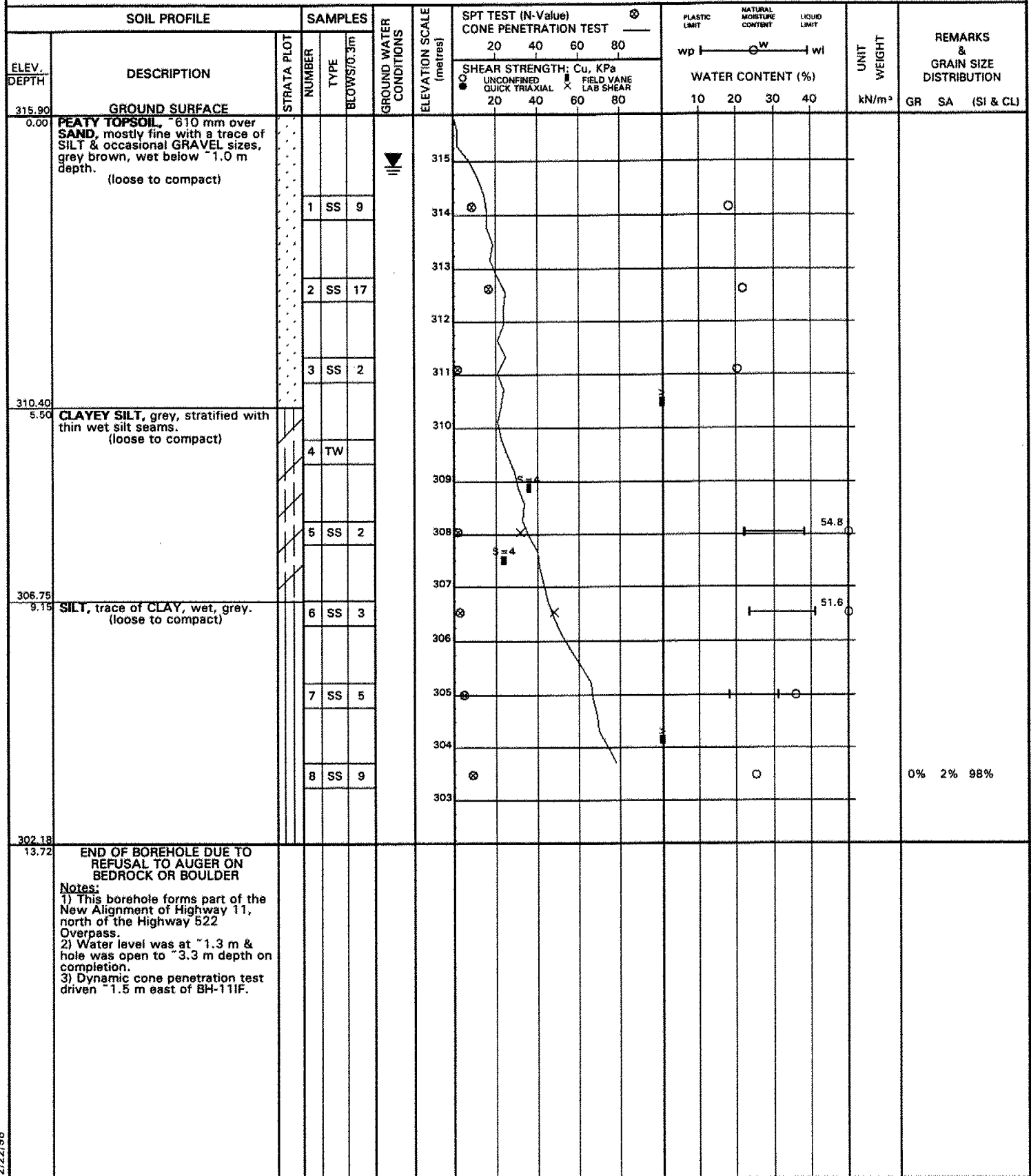
BOREHOLE TYPE Hollow stem augers / Dynamic cone

COMPILED BY M.D.

DATUM Geodetic

DATE September 15, 1998

CHECKED BY I.G.



# RECORD OF BOREHOLE BH-12IF 1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station 12+750, on centreline of Southbound Lane

ORIGINATED BY S.M.

DIST 54 HWY 11

BOREHOLE TYPE Hollow stem augers / Dynamic cone

COMPILED BY M.D.

DATUM Geodetic

DATE September 15, 1998

CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60	80	wp	w		
315.30	GROUND SURFACE														
0.00	PEATY TOPSOIL, ~360 mm over SAND, mostly fine with a trace of SILT, wet below ~0.8 m depth. (compact)		1	SS	14										
			2	SS	14										
			3	SS	11										
			4	SS	12										
307.22	END OF BOREHOLE		5	SS	10										
8.08	Notes: 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Borehole caved wet at ~0.9 m depth on completion.														



# RECORD OF BOREHOLE BH-13IF 1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station ~12+850, on centreline of Southbound Lane

ORIGINATED BY S.M.

DIST 54 HWY 11

BOREHOLE TYPE Hollow stem augers / Dynamic cone

COMPILED BY M.D.

DATUM Geodetic

DATE September 14, 1998

CHECKED BY I.G.

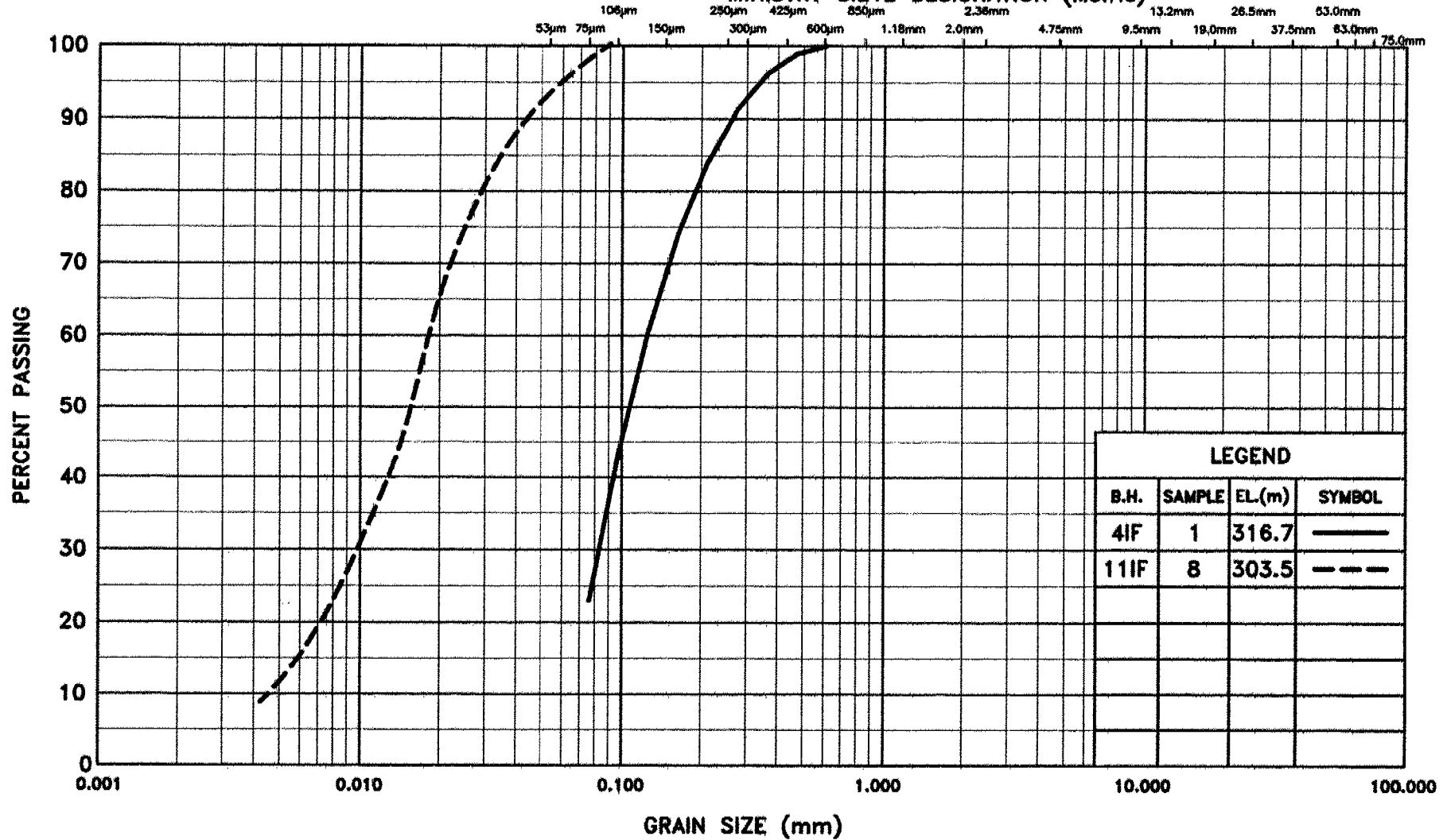
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)		CONE PENETRATION TEST		WATER CONTENT (%)		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR	PLASTIC LIMIT	NATURAL MOISTURE CONTENT		
314.80	GROUND SURFACE												
0.00	PEATY TOPSOIL, ~300 mm over SAND, mostly fine with a trace of SILT, wet below ~0.9 m depth. (compact)		1	SS	13								
			2	SS	17								
			3	SS	10								
			4	SS	5								
			5	SS	15								
306.72	END OF BOREHOLE												
8.08	Notes: 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 11 Overpass. 2) Borehole caved wet at ~1.0 m depth on completion. 3) Dynamic cone penetration test driven ~1.5 m west of BH-13IF.												



# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



### LEGEND

B.H.	SAMPLE	EL.(m)	SYMBOL
4IF	1	316.7	—
11IF	8	303.5	- - -

Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION  
SILTY SAND & SILT

FIGURE 1

W.P 774-93-00



PROJ. No. S07524GIF

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT

SAND

GRAVEL

FINE

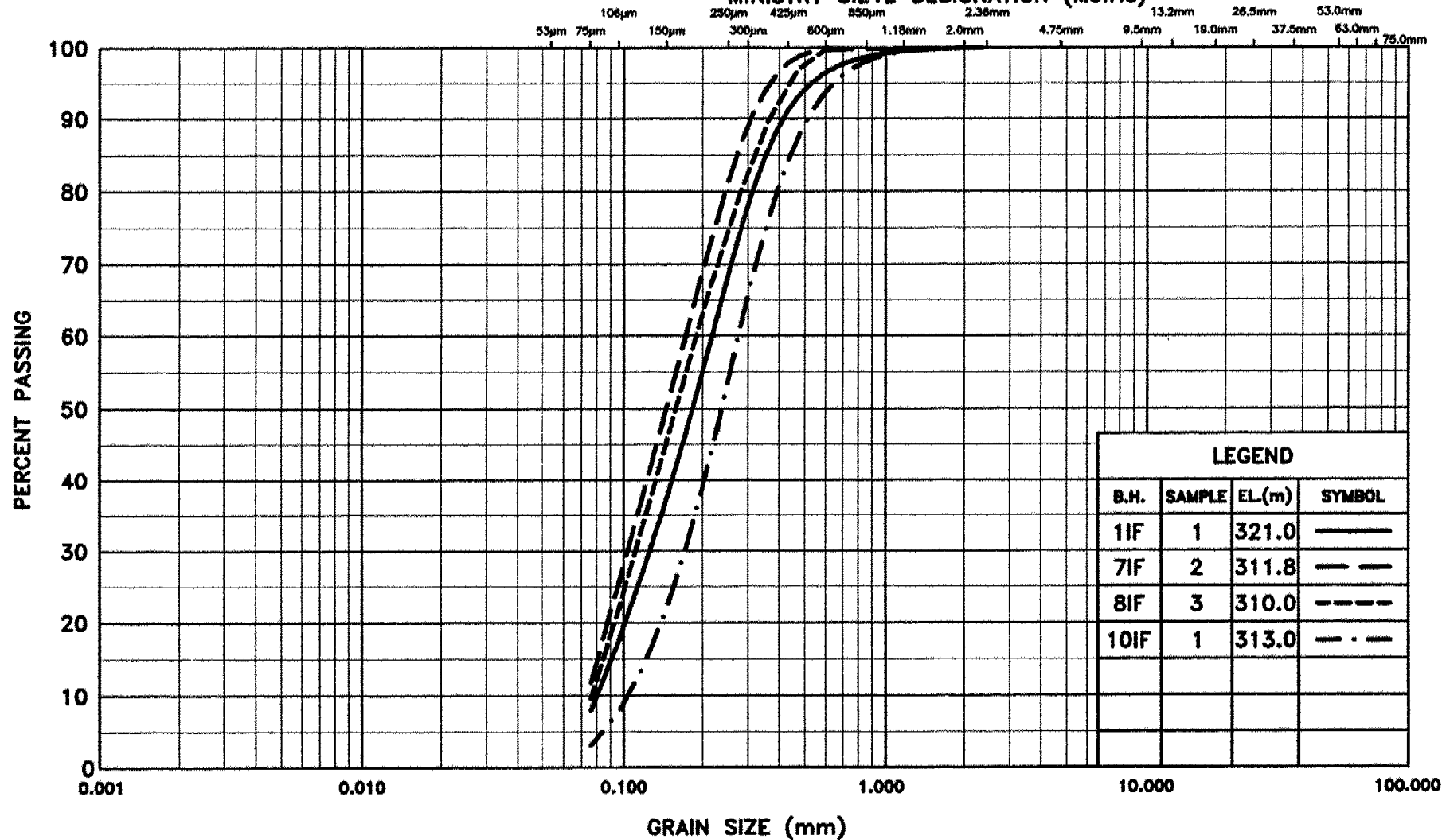
MEDIUM

COARSE

FINE

COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION

SAND

FIGURE 2

W.P 774-93-00



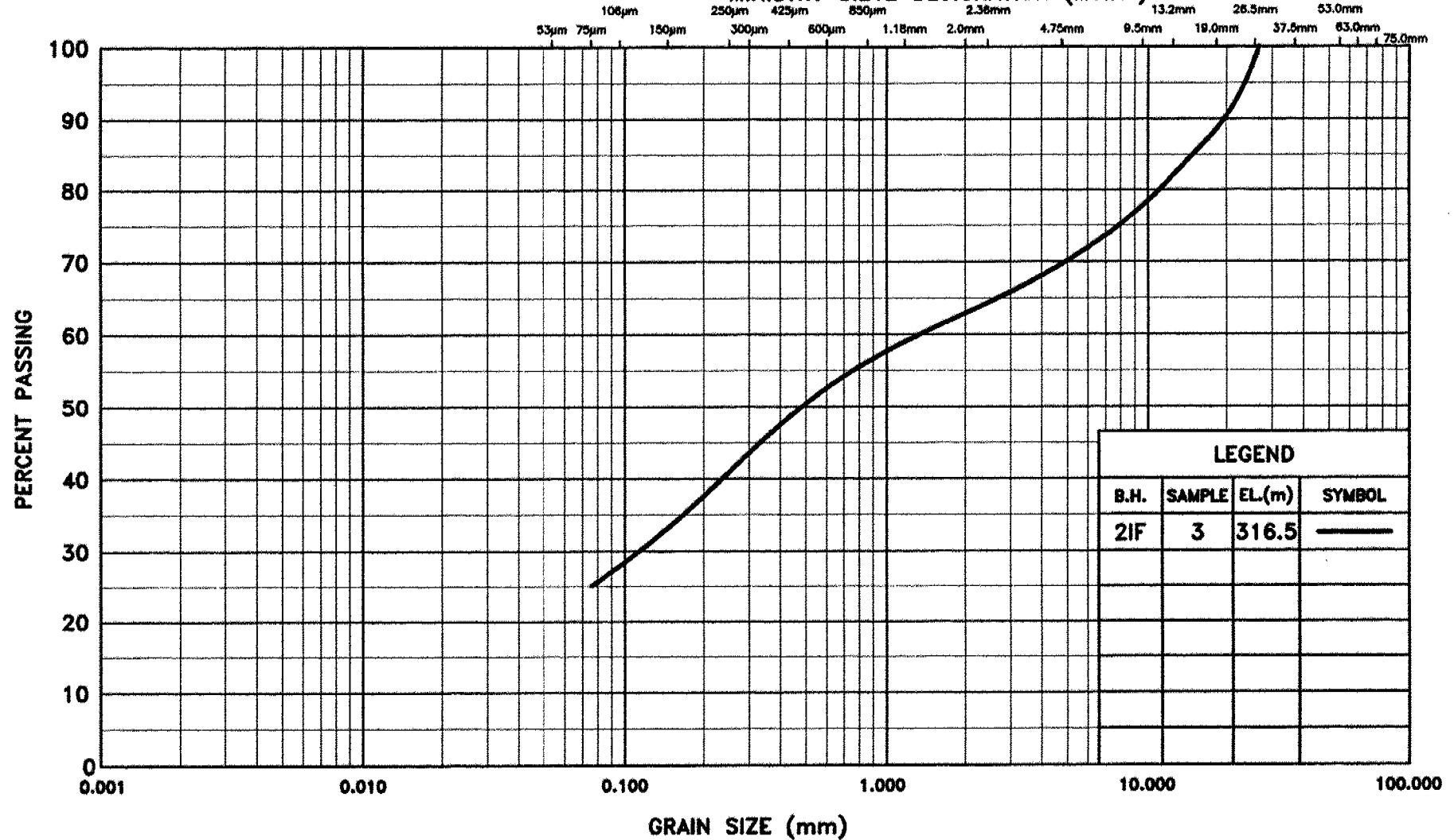
PROJ. No. S07524GIF



# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION

SAND & GRAVEL TILL

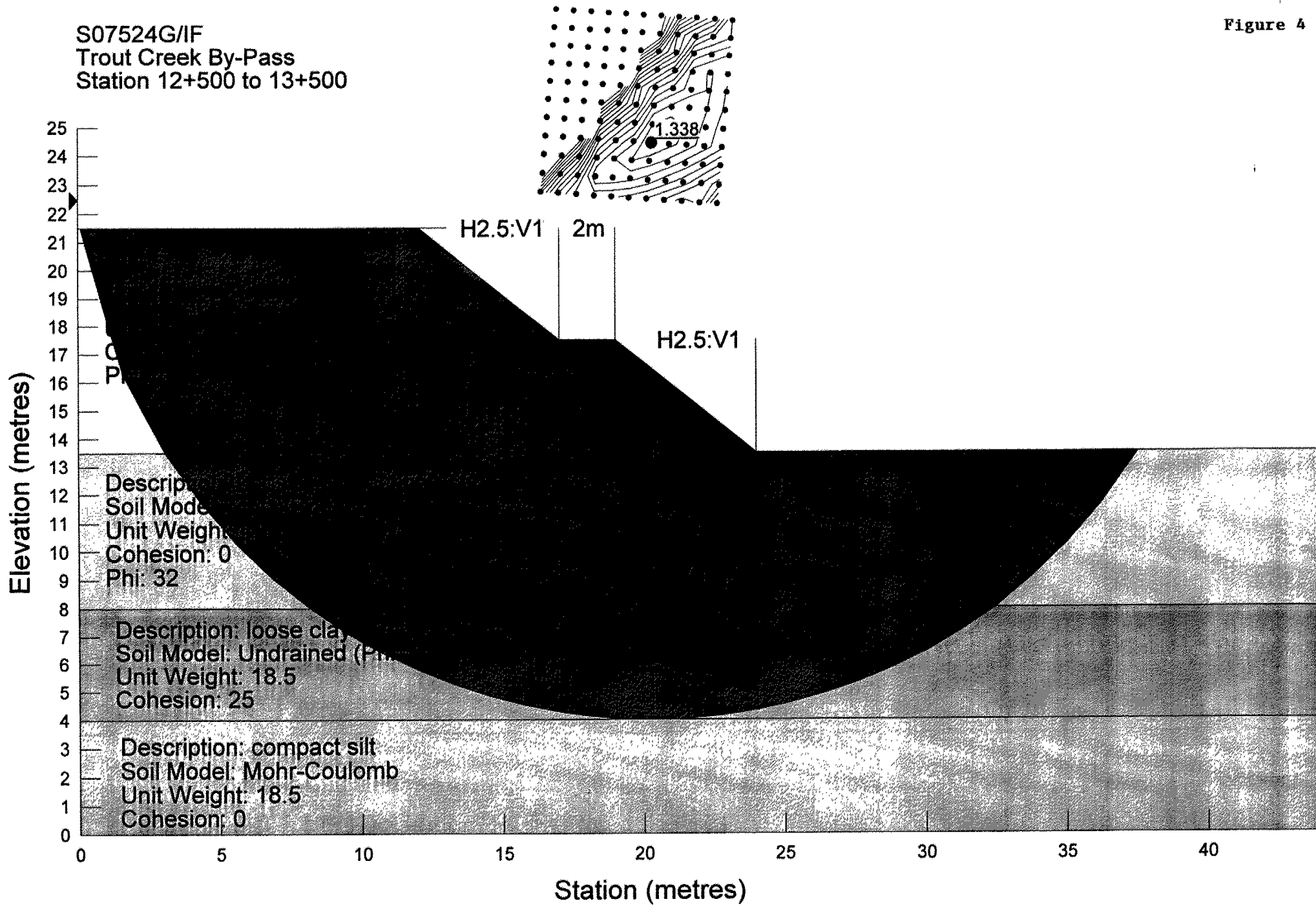
FIGURE 3

W.P 774-93-00



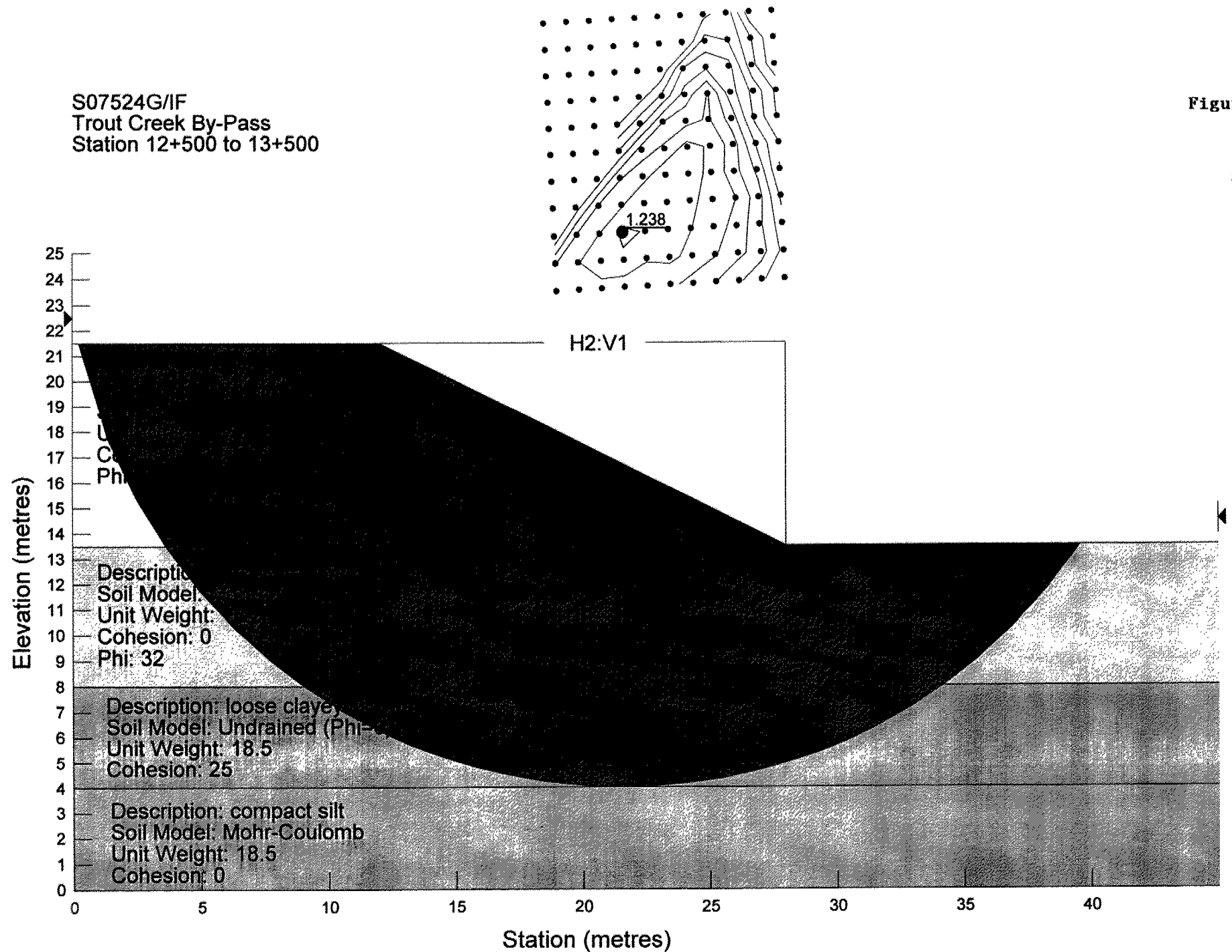
PROJ. No. S07524GIF

Figure 4



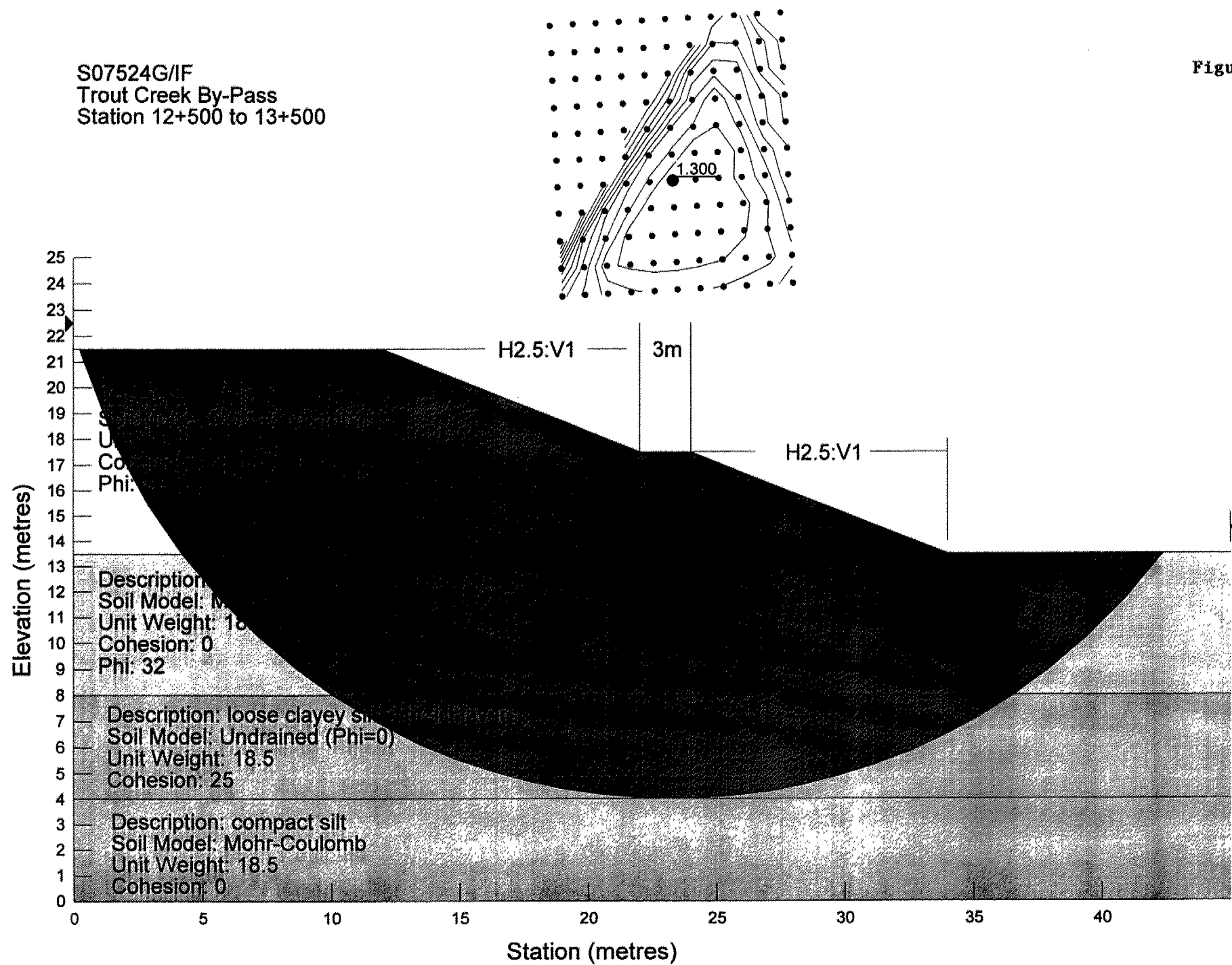
S07524G/IF  
Trout Creek By-Pass  
Station 12+500 to 13+500

Figure 5



S07524G/IF  
 Trout Creek By-Pass  
 Station 12+500 to 13+500

Figure 6



# OVERSIZE DRAWING(S)

# MEMORANDUM



See note pg. 2

To: E. Gallant, P. Eng.  
Senior Project Engineer  
Planning and Design, Northern Region

March 25, 1999

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

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

Re: Foundation Investigation Report Review  
Glen Roberts Road Connection(Stations 9+740 to 9+960)  
Northbound and Southbound Lanes (Station 12+350 to 12+850)  
Trout Creek By-Pass  
WP 774-93-00  
District 54, Sudbury

We have reviewed the draft reports prepared by Trow Consulting Engineers Ltd for the proposed abovementioned embankment sections. 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. 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..

## **Glen Roberts Road Connection(Station 9+740 to 9+960)**

### *General*

In general, the report does not follow the format that conventionally comprises Foundation Investigation and Design Reports. In this regard we forward the following comments:

1. an Introduction section should be included(rather than a memorandum)
2. A Site Description and Site Geology section should be included
3. an Investigation Procedure section should be included that describes the Field Investigation and the Laboratory Analyses
4. Subsurface Condition descriptions are not considered complete. The soil colour, SPT N values, consistency description, silty clay stress history (preconsolidated), Atterberg Limit chart that illustrates low, medium or high plasticity, gradation of cohesionless soils are not included in the text.
5. Groundwater - The method of groundwater measurement should be included in the specifications. Seasonal fluctuations should be described in the text.
6. there is some inconsistency regarding the extent of the embankment section(Stations 9+740 to 9+960 is on the title page whereas Stations 9+400 to 9+660 is identified in the opening

paragraph on page 1).

### *Embankment Design and Construction*

The method of stability analyses (total stress analysis, circular versus non circular, slope stability application software package, boundary conditions used, etc) and settlement calculation (stress distribution method, elastic settlement, primary consolidation settlement, etc) should be described in the report.

An alternative recommendation should be considered to achieve a safety factor of 1.3 during construction.

Recommendations should be included to reduce the magnitude of settlement within the fill embankment (rockfill material and placement guidelines).

It has been recommended that the rock fill be placed "gradually". The report should expand on this recommendation.

### Northbound and Southbound Lanes (Station 12+350 to 12+850)

#### *General*

In general, the report does not follow the format that conventionally comprises Foundation Investigation and Design Reports. In this regard we forward the following comments:

1. Memorandum should not be included in the report
2. A Site Description and Geology section should be included in the report..
3. An Investigation Procedure section should be included that contains Field Investigation and Laboratory Analyses procedures.
4. Atterberg Limit Tests have not been plotted on a Atterberg Limit Plasticity Chart.

### *Embankment Design and Construction*

The method of stability analyses (total stress analysis, circular versus non circular, slope stability application software package, boundary conditions used, etc) and settlement calculation (stress distribution method, elastic settlement, primary consolidation settlement, etc) should be described in the report.

A discussion of the different fill materials and the resulting different embankment geometries should be included in the report.

On Figure 4, the slope geometry should be clarified as 1.25H:1V rather 2.5H:1V.

Recommendations to realize the predicted settlement to minimize post construction settlements

in final  
AND REPORT WAS NOT  
SEALED.

Oct. 4/99.  
none of the changes  
were made  
D.O

Final Report

x  
x  
x  
x

x

x

x

should be included in the report. X

It has been recommended that the rock fill be placed "gradually". The report should expand on this recommendation.

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  
D. Smith



**Foundation Investigation  
and Design Report  
Bridge Structure, Highway 522 Underpass  
(Site 44-370)  
Trout Creek By-Pass  
(King's Highway 11)  
District 54, Sudbury  
GWP No. 774-93-00  
W.P. No. 770-93-01**

**Prepared for:**

**Marshall Macklin Monaghan  
80 Commerce Valley Dr., East  
Thornhill, Ontario  
L3T 7N4**

**Trow Consulting Engineers Ltd.**

**1074 Webbwood Drive  
Sudbury, Ontario P3C 3B7  
Telephone: (705) 674-9681  
Facsimile: (705) 674-8271**

**SO7524G/C  
July, 1999**

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### Appendix A

Site Plan & Soil Profiles	Dwgs. 1A, 1B & 1C
Notes on Sample Descriptions	Dwgs. 2A & 2B
Borehole Logs	
Rock Core Description	Table 1-1

### Appendix B

Grain Size Analyses

## Preface

Work Project GWP 774-93-00 is one of a series of projects for the four lane expansion of Highway 11. The project involves the four lane design of Highway 11, from 4.0 km south of Highway 522, northerly for 7.9 km. It will result in the construction of a westerly by-pass of the existing Highway 11 and the Town of Trout Creek.

This work project is located in the Townships of Laurier and Himsworth South, within the geographic District of Parry Sound. The project requires geotechnical input for the following major components:

- New pavement design for the entire length of the four lane by-pass, including associated service roads.
- New structure, Trout Creek South Interchange (underpass), Site 44-372.
- New structure, Trout Creek, Northbound Lanes, Site 44-371.
- New structure, Trout Creek, Southbound Lanes, Site 44-371.
- New structure, Highway 522 (underpass), Site 44-370.
- New structure, Trout Creek North Interchange (underpass), Site 44-369.

The following report deals with the new bridge structure at Highway 522, Site 44-370. Separate reports will be submitted for the additional components.

# **PART 1 FOUNDATION INVESTIGATION**

## **1.1 Introduction**

This submission presents the results of a geotechnical foundation investigation by Trow Consulting Engineers Ltd. (Trow) for the proposed crossing at Highway 522 and the proposed King's Highway 11 (Trout Creek By-Pass), at Site 44-370. It is Trow's understanding that a two span structure will be constructed, with the central pier located in the median of the proposed King's Highway 11. This report contains factual information (obtained from the field investigation) pertaining to the design parameters required for the bridge foundations and related earthworks.

## **1.2 Site Description and Geological Setting**

### **1.2.1 Site Description**

The site is located in the Township of South Himsforth at the proposed Highway 11 (Trout Creek By-Pass), and Highway 522 intersection, approximate Station 11+224 along the proposed Highway 11, which corresponds to Station 10+000, along Highway 522 at this location.

The proposed new, two-span bridge will be constructed to carry Highway 522 traffic over the four lanes of King's Highway 11. An approximately 8 m grade increase of Highway 522 at the bridge abutments is anticipated, in accordance with the proposed grading plan.

The terrain at the proposed bridge structure is relatively flat, although the grade of the existing Highway 522 rises gently towards both the west and east sides. The existing grade of Highway 522, at the bridge site, is at elevation 315 m and the road then rises gradually some 5 m, over a distance of approximately 240 m on the east side (up to Station 10+240), and 3 m, over approximately 200 m on the west side (up to Station 9 + 800).

The grade of Highway 522 will be raised to elevation 323 m at the bridge to accommodate the proposed four lanes of the by-pass. This arrangement will require approach embankments along Highway 522, approximately 240 m long (from the east side) and 200 m long (from the west side).

No bedrock outcrops are visible in the immediate vicinity of the proposed bridge; however, a rock cut is visible approximately 200 m along Highway 522 on the west side. There are mature trees, with heavy underbrush on either side of Highway 522, i.e. along the alignment of the proposed King's Highway 11.

### **1.2.2 Geological Setting**

According to OGS Maps 2544 and 2556, as well as Ontario Geological Survey Map P. 3160 (Quaternary geology, South River area), the site is located in what is known as the Central Gneiss Belt, i.e. mainly felsic igneous rocks of the Mesoproterozoic Group.

The overburden is expected to be relatively shallow, comprising, for the most part, of detritic sands and gravels with some prodeltaic deposits, mainly silts. A thin layer of basal, stoney, glacial till can be expected immediately over the bedrock.

## **1.3 Investigative Procedures**

### **1.3.1 General**

Part 1 of this report describes the investigative procedures adopted for the geotechnical assessment of the Highway 522 flyover structure at Trout Creek By-Pass, King's Highway 11. Properties of the overburden soils were obtained by in-situ and laboratory testing and the procedures employed during the investigation, are described below.

### **1.3.2 Field Investigation**

The field work for the investigation related to the proposed bridge structure was carried out between May 12 and May 13, 1998, and on May 25, 1998, and consisted of five(5) boreholes (BH's 1-CF to 5-CF), three(3) dynamic cone penetration tests (C-1CF to C-3CF) and four(4) additional auger probes (AP-1CF to AP-4CF). At least three (3) explorations were completed at each of the foundation elements. The probes, dynamic cones and boreholes were advanced to depths ranging from 3.7 m to 11.4 m.

The borehole, probe and dynamic cone penetration locations are shown on the attached site plan, Drawing 1, in Appendix A. These locations, as well as the surface elevations, were established by a survey crew from Marshall Macklin Monaghan, and are referenced to geodetic datum.

The boreholes, cones and probes were advanced through the overburden soils using a truck mounted CME-55 drill, equipped with solid and hollow stem augers, and operated by a soils drilling contractor, Marathon Drilling Limited. Soil samples were obtained by using a 51 mm O.D. split-spoon sampler in conjunction with standard penetration tests (ASTM D1586) at approximately 0.75 m and 1.5 m intervals. The standard penetration (N) values, together with the blows from the dynamic cone penetration tests, were recorded and used to provide an assessment of the compactness of the overburden soils. The recovered soil samples were used for identification and laboratory testing.

Upon completion, boreholes were backfilled with auger cuttings from the same boreholes, and compacted at regular intervals by applying back pressure with the auger. Where boreholes were advanced within the pavement surface, cold mix asphalt was placed to adequately patch the damaged area.

At three (3) of the borehole locations, i.e. at each of the three foundation elements, conventional rock coring techniques were used to advance the boreholes approximately 3 metres into the underlying bedrock. An "NQ" size core barrel and casing were used and core samples of the bedrock were retrieved for rock quality determination and classification.

Details of the soil and bedrock conditions encountered in the boreholes are included on the logs in the attached Appendix A. The additional two standard information sheets included with the logs, provide further details on soil descriptions for classification purposes.

### **1.3.3 Laboratory**

The laboratory testing program for select soil samples consisted of the following:

- Natural moisture content determinations
- Grain size distribution analyses
- Laboratory shear tests

The laboratory test results are summarized on the attached borehole logs in Appendix A. The grain size distribution for selected soil samples are presented in Appendix B.

## 1.4 Subsurface Conditions

The borehole locations are shown on the site plan, Drawing 1, in Appendix A. Also included in Appendix A are the borehole, probe and dynamic cone penetration logs. Soil sections, longitudinal, as well as at each of the three foundation elements, are plotted on Drawings 1A, 1B and 1C. Based on this information, the following different soil layers were encountered:

- fill
- silt
- sand/sand and gravel
- bedrock

A summary of the above soil strata encountered in the boreholes, and inferred from the probes and dynamic cone penetration tests, is presented below.

### 1.4.1 Fill

The fill at the test locations is associated with the road construction materials for the existing Highway 522. Beneath the present asphalt (~50 mm thick) and the base and subbase granulars (~300 mm thick), an underlying layer of sand fill was generally encountered, which extends to depths of 1 m to 2 m below the road grade. The sand fill is generally fine, although it contains random pieces of old asphalt, some gravel and/or cobble sizes, as well as minor organic staining and contamination. At borehole 3-CF, a sand stratum extends somewhat deeper, to about 3 m depth. Since this deposit also contains minor organic inclusions, it is also believed to be fill or possibly alluvial in origin, having been deposited by previous meanderings of an adjacent small creek, which runs beneath the existing Highway 522 in this area.

The compactness of the fill, based on the standard penetration resistance, "N", values ranged from 4 to 35 blows/300 mm, indicating a loose to dense state.



Grain size analyses on samples of the material confirm that the deposit is mainly a fine sand with a silt fraction of between 7% to 20%. Moisture contents vary from less than 10% above the water table to about 20% below the groundwater table.

#### 1.4.2 Silt

A deposit of silt was encountered in three boreholes (boreholes 2-CF, 3-CF and 5-CF). This silt stratum is absent beneath the east abutment (boreholes 1-CF and 4-CF). The silt contains some sand seams and odd layering where it is slightly cohesive. The standard penetration resistance “N” values ranged from 3 to 9 blows/300 mm, indicating a very loose to loose state of compaction. The thickness ranged from 1 m (borehole 3-CF) to 3 m (borehole 2-CF), and the moisture content from 25% to 35%.

#### 1.4.3 Sand and Gravel

A basal zone of sand and gravel was encountered in all five boreholes, with the exception of borehole 2-CF. The deposit is reasonably well-graded with up to 31% silt sizes in places. At some locations, the deposit appears to be weakly cemented, exhibiting a “till-like” structure. At borehole 4-CF, the sand and gravel contains odd, small pieces of wood and cobble sizes. At this particular location, the sand and gravel deposit is likely fill, associated with backfill around the existing, adjacent, 780 mm diameter, CSP culvert.

The standard penetration resistance “N” values, with the exception of borehole 4, ranged from 12 to 41 blows/300 mm, indicating a compact to dense condition. The thickness ranges from about 1 m at borehole 1-CF to 4 m at borehole 3-CF. Moisture contents range from 5% to 20%, although at borehole 4-CF, higher moisture contents of 25% to 30% were measured.

#### 1.4.4 Bedrock

Bedrock was confirmed by retrieving “NQ” size cores in the boreholes (BH’s 1-CF, 2-CF and 3-CF), i.e. at one borehole beneath each of the three foundation elements. Based on the borehole, probe and dynamic cone penetration tests, the bedrock level was established at the following depths:

- East Abutment (BH’s 1-CF and 4-CF, AP-3-CF, C-1CF)  
3.7 m (~El. 311 m) to 4.5 m (~El. 310.4 m)

- Centre Pier (BH 2-CF, AP-1-CF, AP-2-CF)  
5.0 m (~El. 310 m) to 5.3 m (~El. 309.1 m)
- West Abutment (BH's 3-CF and 5-CF, AP-4-CF, C-2CF)  
5.3 m (~El. 310 m) to 8.1 m (~El. 307.2 m)

Detailed descriptions of the rock are presented in Table 1-1 in Appendix A. Generally, the bedrock can be described as a pinkish, light grey, biotite-Hornblende gneiss. The rock is strong and unweathered for the most part.

Rock core recovery was 100% for all runs and the Rock Quality Designation (RQD) values ranged from 66% to 100%.

## 1.5 Groundwater Conditions

Information regarding the groundwater levels at the site was obtained by measuring the water levels in the open boreholes after completion of drilling. The groundwater table, at the time of the field work, was established at a depth of about 2 m to 2.5 m below the grade of Highway 522, which is close to the grade of the surrounding, poorly drained, relatively flat terrain.

Seasonal variations in the water table should be expected with higher levels during wetter periods of the year (such as spring thaw and late fall) and lower levels during drier periods.

## **Part 2    Engineering Discussions and Recommendations**

### **2.1    General**

The following subsections address geotechnical considerations pertaining to the proposed two-span bridge for the Highway 522 underpass of the Trout Creek By-Pass (King's Highway 11). A two-span bridge is proposed to carry Highway 522 traffic over the four lanes of the new By-Pass. The central pier will be located in the By-Pass median, with the abutments located on the west and east sides of the south and north bound lanes of Highway 11.

### **2.2    Foundations**

Based on the explorations, bedrock was encountered at depths of approximately 4 m at the east abutment (8 m from the proposed profile grade), 5 m at the central pier (7.5 m from the proposed profile grade), and 5 m to 8 m at the west abutment (7 m from the proposed profile grade). Foundation alternatives include piled foundations (integral abutments can be considered, although the rock is probably too shallow at the east abutment and pier), and/or spread foundations excavated to sound bedrock. At the west abutment, excavations of up to 8 m below grade will be required, i.e. where the deepest bedrock was encountered. Consequently, at the west abutment, it may be possible to establish the foundations at a higher level on the native, compact, granular soils.

Other options include installing a compacted granular mat over competent material (till and/or rock) to support the foundations, or possibly using caisson-type foundations. However, conventional augered caisson foundations are not normally practical in Northern Ontario because of potential, unknown, sharp irregularities in the bedrock surface, difficulties "seating" casing into the hard, strong, Precambrian rock, as well as the potential excavation difficulties with water seepages in the granular overburden and possible boulders near the rock contact.

Foundation options and design parameters are discussed in the following paragraphs.

## 2.2.1 Foundations on Bedrock

### 2.2.1.1 East Abutment

At the location of the east abutment (BH-1CF, 4-CF, cone 1-CF and AP3-CF), there is approximately 4 m of overburden soil overlying bedrock. As such, it would be feasible to excavate down to the rock and place the foundation directly on the bedrock surface. For the purpose of design, in accordance with the Ontario Highway Bridge Design Code, the following bearing capacities can be used for spread footings placed directly on the gneiss bedrock, subject to inspection by a qualified geotechnical engineer.

Table 2-1 Spread Footing Capacity on Bedrock	
	Spread Footing
Factored Bearing Capacity at ULS	7,500 kPa

The above Factored Bearing Capacity at ULS applies to spread footings placed directly on bedrock with a good Rock Mass Quality (RQD>75). The bearing capacity at SLS will not govern for a spread footing founded on bedrock, since the loads required to produce unacceptable settlements of the structure will be much larger than the recommended values for the factored capacity at ULS.

For the east abutment area, the borehole, cone and probe data indicate that the construction of spread footings on bedrock would require excavation and removal of approximately 4 m of overburden soil, i.e. down to approximate El. 310 m. The footing area must be cleared of all loose materials prior to placement of concrete and inspected by a qualified geotechnical engineer to verify the Rock Mass Quality.

As per Section 6-8.4.2 of the Ontario Highway Bridge design code, a reduction factor would normally be applied to the Ultimate Bearing Resistance at ULS (7,500 kPa) to account for the effects of inclined loadings. Recent comments, however, received from the Pavement and Foundation Section of MTO indicate that "*Although the OHBDC Code talks about bearing resistance reduction due to inclined loadings for footing on bedrock. The OHBDC committee has decided that no such reduction will be required if the footing is constructed on bedrock*". As such, for spread footings on

bedrock, the structural engineer should consult with the Ministry to confirm that a reduction factor for inclined loadings need not be applied.

#### 2.2.1.2 *Centre Pier*

The subsurface conditions for the centre pier location (BH-2CF, AP1-CF and AP2-CF) are similar to those discussed for the east abutment, although the overburden is slightly thicker, i.e. approximately 5 m. As a result, spread footings placed directly on bedrock using the bearing values given in table 2-2, below, are an option for the central pier foundations subject to inspection during construction by a qualified geotechnical engineer. The bedrock surface is expected at a depth of about 5 m below grade (~El. 310 m).

TABLE 2-2 Spread Footing Capacity on Bedrock	
	Spread Footing
Factored Bearing Capacity at ULS	7,500 kPa

The above Factored Bearing Capacity at ULS applies to spread footings placed directly on bedrock with a good Rock Mass Quality (RQD>75). The bearing capacity at SLS will not govern for a spread footing founded on bedrock, since the loads required to produce unacceptable settlements of the structure will be much larger than the recommended values for the factored capacity at ULS.

#### 2.2.1.3 *West Abutment*

The subsurface conditions for the west abutment location (Boreholes 5-CF, 3-CF, cone 2-CF and AP4-CF) are similar to those discussed for the centre pier and east abutment; however, on the south side (cone C-2CF and borehole 3-CF), the rock is locally deeper, i.e. at a depth of some 8 m below grade. Foundations placed directly on bedrock can be designed using the bearing values specified in Table 2-3, below, subject to inspection during construction by a qualified geotechnical engineer.

<b>TABLE 2-3</b> <b>Spread Footing Capacity on Bedrock</b>	
	Spread Footing
Factored Bearing Capacity at ULS	7,500 kPa

The above Factored Bearing Capacity at ULS applies to spread footings placed directly on bedrock with a good Rock Mass Quality ( $RQD > 75$ ). The bearing capacity at SLS will not govern for a spread footing founded on bedrock, since the loads required to produce unacceptable settlements of the structure will be much larger than the recommended values for the factored capacity at ULS.

Given the depth to bedrock at this abutment, and the high water table, necessitating 6 m to 8 m excavations below the groundwater table, spread footings located on the bedrock may not be the most prudent foundation alternative at this site, as construction difficulties may arise. As such, the other foundation options discussed in this report may be more suitable for this abutment.

As a further alternative, given the depth to rock at this abutment, foundations placed at a higher elevation on the compact and dense, native, sand/sand and gravel strata should also be considered. The compact and dense sand/sand and gravel stratum was encountered at a depth of about 4 m (elevation 311 m) beneath the west abutment location. Since the level of the sand and gravel stratum is about 2.5 m below the water table, dewatering will be required if this option is selected. Foundations placed on the undisturbed, dewatered, sand/sand and gravel stratum could be designed using the bearing capacity values specified in Table 2-4, below.

<b>TABLE 2-4</b> <b>Spread Footing Capacity on Sand/Sand &amp; Gravel</b>	
	Spread Footing
Factored Bearing Capacity at ULS	300 kPa
Bearing Capacity at SLS	150 kPa

Although founding the abutment on the underlying native sand/sand and gravel strata will require shallower excavations than founding the abutment on bedrock, excavations, will still be required below the water table. This may require dewatering, which may include sheet piling to accomplish.

As such, considering the low allowable bearing resistance values and the potential need for dewatering, this option is not recommended.

#### 2.2.1.4 *Anticipated Bedrock Elevations*

The following table summarizes the location and estimated bedrock elevations at the three foundation elements.

<b>Table 2-5</b> <b>Location and Estimated Elevation of Bedrock Foundation</b>			
<b>Location</b>	<b>Boreholes &amp; Probe Holes</b>	<b>Overburden Thickness (m)</b>	<b>Approximate Bedrock Elevation (m)</b>
East Abutment	Borehole 1-CF	3.7	311.0
	Borehole 4-CF	4.4*	310.4
	Probe Hole AP-3CF	4.1*	311.0
	Cone C-1CF	3.7*	311.4
	Cone C-3CF	5.1*	310.0
Centre Pier	Borehole 2CF	5.2	309.9
	Probe Hole AP-1CF	5.0*	310.0
	Probe Hole AP-2CF	5.3*	310.0
West Abutment	Borehole 3CF	8.1	307.2
	Borehole 5CF	5.3*	310.0
	Cone C-2CF	8.2*	307.5
	Probe Hole AP-4CF	6.7*	309.1

#### *\*Assumed bedrock level*

The above elevations are for preliminary design purposes and were estimated based on the factual borehole, dynamic cone and auger probe holes drilled near the abutment and pier locations. Interpolation between boreholes and probe holes is approximate, and as such, actual footing elevations will depend on the conditions encountered at the time of construction. The bedrock surface in Northern Ontario is known to be erratic. The rock surface at the footing base must be cleaned of all loosened or highly fractured rock and be inspected by a qualified geotechnical engineer to verify the Rock Mass Quality prior to placement of concrete.

### 2.2.2 Footings on Compacted Granular Pad

It may be feasible to establish footings on a Granular A (or equivalent) compacted granular pad. The existing upper loose soils should be subexcavated down to bedrock at both the east and west abutments, as well as at the central pier. As an alternative, at the west abutment, it may only be necessary to excavate down to 4 m (El. 311 m), i.e. down to the compact and/or dense, native, granular soil horizon, provided that the groundwater is controlled to prevent disturbance. The granular pad, when placed over the rock (pier and east abutment) and/or over sand and gravel (west abutment), should extend horizontally a minimum of 1.0 m beyond the plan limits of the footing and have side slopes no steeper than 1 horizontal to 1 vertical. The granular material should be compacted to 100 percent Standard Proctor Maximum Dry Density.

The bearing capacities recommended for the abutment footings placed on this compacted granular pad design, based on the Ontario Highway Bridge Design Code, are as follows:

<b>TABLE 2-6</b> <b>Spread Footing Capacity on Compacted Granular Mat</b>	
	Spread Footing
Factored Bearing Capacity at ULS	400 kPa
Bearing Capacity at SLS	200 kPa

As an alternative to using a granular pad, it would also be feasible to “upfill” over the bedrock, or possibly over the native sand and gravel at the west abutment, and up to underside of the proposed foundations, with lean concrete (typically 15 Mpa mix).

Footings placed on a granular pad beneath the groundwater table will require additional construction considerations. It is very difficult to compact the granular material to an acceptable Standard Proctor Maximum Dry Density by excavation and replacement methods underwater. Since the groundwater table was measured to be above the bedrock level at the time of the investigation, it is probable that dewatering will be required to ensure the granular material can be placed and compacted at optimum moisture levels. Alternatively, the granular material could be replaced by a nominal 400 mm clear stone beneath the groundwater table.



### 2.2.3 Frost Protection

Frost cover is not required for footings placed directly on bedrock. Due to the open nature of bridges, for footings placed on a granular pad, or the native granular soil for the west abutment, a minimum frost cover of 2 m should be provided.

### 2.2.4 Sliding Resistance

The computation of the sliding resistance of the spread footings shall be carried out in accordance with O.H.B.D.C. An unfactored friction angle,  $\phi$  of 32 degrees can be used for sliding along the bedrock and footing base and  $\phi$  of 35 degrees for sliding along granular soils (native sand/sand and gravel or engineered granular pad).

If the factored resistance against sliding failure is inadequate based on friction, then the footings normally could be anchored into bedrock by means of keys, dowels or sockets. However, given the hardness of the bedrock encountered at the site sockets and keys will likely be impractical, and developing adequate resistance against sliding of spread footings founded on the sloping bedrock at the site will likely require dowels. An unfactored coefficient of passive earth pressure,  $K_p'$ , equal to 3.7, can be used for design of a passive resistance key.

### 2.2.5 Piled Foundation

#### 2.2.5.1 *Capacity and Length*

Piling could be considered for the foundation elements at this site. However, because of the proximity of bedrock at the east abutment and central pier (<5 m below grade), piles may only be feasible for support at the west abutment, i.e. where bedrock is deeper. End bearing piles are normally only feasible when the length/width ratio exceeds 10, i.e. a length of 3 m for a 300 mm pile section, since it is very difficult to achieve adequate lateral support for the piles, and the piles will tend to rotate if the applied loading is eccentric.

Piles driven to bedrock can be designed based on the following Limit States design values in accordance with the O.H.B.D.C.

<b>Table 2-7</b> <b>Design Pile Capacities (kN)</b>		
	<b>HP 310x79</b>	<b>HP 310x110</b>
Factored Axial Resistance*	1430 kN	2000 kN
* <i>Note: Structural Office Policy Memo 98-01, April 15, 1998</i>		

Based on the attached borehole logs in Appendix A, the previous Table 2-5 shows a summary of the approximate bedrock elevation at the test locations at which piles would be expected to be founded. Drawings 1A, 1B and 1C in Appendix A show interpreted soil and rock subsurface profiles at the two abutments and pier.

It should be noted that the elevations given in Table 2-5 are approximate. Furthermore, although not experienced in the borings put down at this site, the bedrock elevation in this part of the country can be variable and may change rapidly over a very short distance.

#### **2.2.5.2 Construction**

All piles should be driven to bedrock. Since the boreholes indicate that the bedrock elevations are relatively uniform, the potential for irregular, steeply sloping bedrock at the foundation locations is considered to be low to moderate. The bedrock in this part of Northern Ontario, however, is known to be variable. As such, some minor problems may arise during pile seating. At some locations, the piles may have a tendency to skip over the bedrock surface resulting in alignment problems and deeper penetration. In the event that this problem occurs, somewhat longer piles may be required and, in some cases, piles may have to be added or replaced.

To minimize seating difficulties, rock injector points should be considered to facilitate proper seating. All piles must be fitted with reinforcing plates welded to the flanges as per OPSD 3301 to minimize pile damage. It is recommended that, during pile driving and upon initial contact with the bedrock, the pile driving energy should be reduced and subsequently increased incrementally until the piles have been sufficiently seated. OSLO, or similar rock points, installed and driven in accordance with OPSD 3304 and OPSS 903, respectively, may be considered. Once the locations and orientations of the piles have been determined (i.e. during the preliminary design stage), the use of such methods will be determined and recommendations will be provided by this office as required.

All lateral loads at the abutments should be supported using inclined piles.

### 2.2.5.3 *Horizontal Subgrade Reaction Parameters*

Should finite element modelling techniques be utilized, the Horizontal Subgrade Reaction Parameters can be calculated as follows:

$$k = k_1 Z/B$$

Where:  $k_1$  = coefficient of horizontal subgrade reaction for a 300 mm (1 ft.) Wide pile at 300 mm (1 ft.). The values of  $k_1$  are given below in tonnes/m<sup>3</sup>

$Z$  = depth

$B$  = width of pile

	Dry	Submerged
Sand and Gravel Fill (compact)	700	450
Silt (loose)	225	110
Sand and Gravel (compact)	700	450
Sand and Gravel (loose)	350	175

*These values are for design in the elastic range and are taken from published values.*

## 2.2.6 Caisson Foundations

### 2.2.6.1 *Bearing Capacity*

Where bedrock is deeper at the west abutment, a caisson type foundation system to rock could be considered. Caisson foundations placed directly on bedrock could be designed using the bearing values specified in Table 2-8, below.

<b>TABLE 2-8</b> <b>Caisson Foundation Capacity on Bedrock</b>	
	<b>Spread Footing</b>
Factored Bearing Capacity at ULS	5,000 kPa

The above Factored Bearing Capacity at ULS applies to caisson foundations placed directly on bedrock with a good Rock Mass Quality (RQD>75). The bearing capacity at SLS will not govern for a caisson founded on bedrock, since the loads required to produce unacceptable settlements of the structure will be much larger than the recommended values for the factored capacity at ULS.

#### **2.2.6.2 Construction**

As noted earlier in this report, caisson foundations, which are typically large diameter augered elements, which are cased to rock, are not normally feasible in Northern Ontario. The potential for sharp, unknown irregularities in the bedrock surface, difficulties “seating” and sealing casings at the hard strong bedrock contact, as well as the potential excavation difficulties with water seepages in the granular soils and possible boulders near the rock contact, render augered caissons difficult, if not impractical. As such, if caissons are considered at the west abutment, they will likely have to be installed using a backhoe type excavator with temporary braced shoring to support the open excavation sides and appropriate dewatering procedures..

### **2.3 Backfill**

Backfill to abutments or retaining walls should consist of free-draining granular materials such as Granular “A” and Granular “B” or rock fill. Computation of earth pressures shall be in accordance with Section 6.7.4 of the Ontario Highway Bridge Design Code. Unfactored properties for backfill materials are provided in the following table.

<b>Table 2-9</b> <b>Material Types and Unfactored Properties</b>					
<b>Material</b>	<b>Friction Angle, <math>\phi'</math></b>	<b><math>\gamma(\text{kN/m}^3)</math></b>	<b><math>K_a</math></b>	<b><math>K_p</math></b>	<b><math>K_o</math></b>
Granular A	35 degrees	22.5	0.27	3.7	0.43
Granular B	30 degrees	21.1	0.33	3.0	0.50
Rock Fill	35 degrees	18.0	0.27	3.7	0.43

Note:  $K_a$  is the earth pressure coefficient corresponding to the active state.

$K_p$  is the earth pressure coefficient corresponding to the passive state.

$K_o$  is the earth pressure coefficient at rest.

If rock fill is used as backfill behind abutments, the particle size should be limited to no greater than 300 mm and the backfill must be placed carefully in a manner that does not cause damage to the abutments or other structural components of the bridge.

As shown in figure 6.7.4.3 of the Ontario Highway Bridge Design Code, the lateral earth pressure, as a result of compaction shall be increased by 16 kPa.

## 2.4 Excavations

### 2.4.1 Overburden

Excavations through overburden soil will be required if spread foundations are used. The overburden granular soils are classified as Type 3 soils and the maximum depth of excavation anticipated at the site is approximately 8 metres at the west abutment. As such, excavations in accordance with the Occupational Health and Safety Regulations for Construction Projects for Type 3 soils will be adequate, provided the groundwater in the overburden soil is removed. If appropriate dewatering is not done, the soil would have to be classified as a Type 4 soil and any excavation greater than 1.2 m should then be sloped to 3 horizontal to 1 vertical, starting from the base of the excavation, or appropriate shoring provided.

#### 2.4.2 Bedrock

Any removal of bedrock required for the foundations (spread footings and/or at the base of caissons) will require drilling and blasting procedures.

### 2.5 Approach Embankments

No stability or significant settlement problems are anticipated for the approach embankments established over the essentially granular soils. Topsoil and compressible organics (if present) must be removed from the plan limit of the approach embankments. Based on Trow's adjacent borings for the pavement design of Highway 11, it is likely that the surficial organics will be about 300 mm to 600 mm thick. If rock fill is used to construct the approach embankments, the side slopes and forward slopes should be constructed at a maximum gradient of 1.25H:1V. If Granular "A" or Granular "B" is used, the forward and side slopes should be constructed at 2H (minimum):1V.

The geotechnical conditions are such that integral abutment design could be considered, if feasible, from structural, practical and economical considerations. It should be noted, however, that the depth to bedrock at the east abutment and pier is less than approximately 5 m.

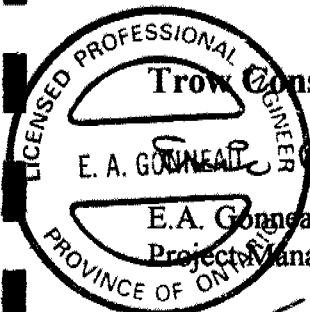
### 2.6 General

The information presented in this report is based on a limited investigation designed to provide information to support an overall assessment of the current geotechnical conditions at the site of the proposed Highway 522/King's Highway 11 Trout Creek By-Pass. The conclusions presented in this report reflect site conditions existing at the time of the investigation. It is noted that the soil boundaries indicated on the logs are inferred from discontinuous sampling and observations during drilling. These boundaries are intended to reflect transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change.

This report has been prepared by Mr. I.W. Gore, P.Eng., and Mr. E.A. Gonneau, P.Eng., and reviewed by Mr. S.E. Gonsalves, P.Eng. The field investigation was performed by Mr. I. Dumpis, C.E.T.

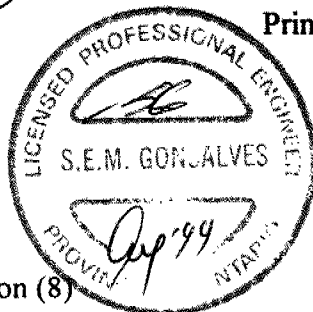
We trust this report is satisfactory for your purposes. Should you have any questions, please do not hesitate to contact this office.

**Trow Consulting Engineers Ltd.**



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Planning & Design  
Mr. E. Gallant

Marshall Macklin Monaghan (1)  
Mr. R.D. Kivi, P.Eng.

**APPENDIX A**

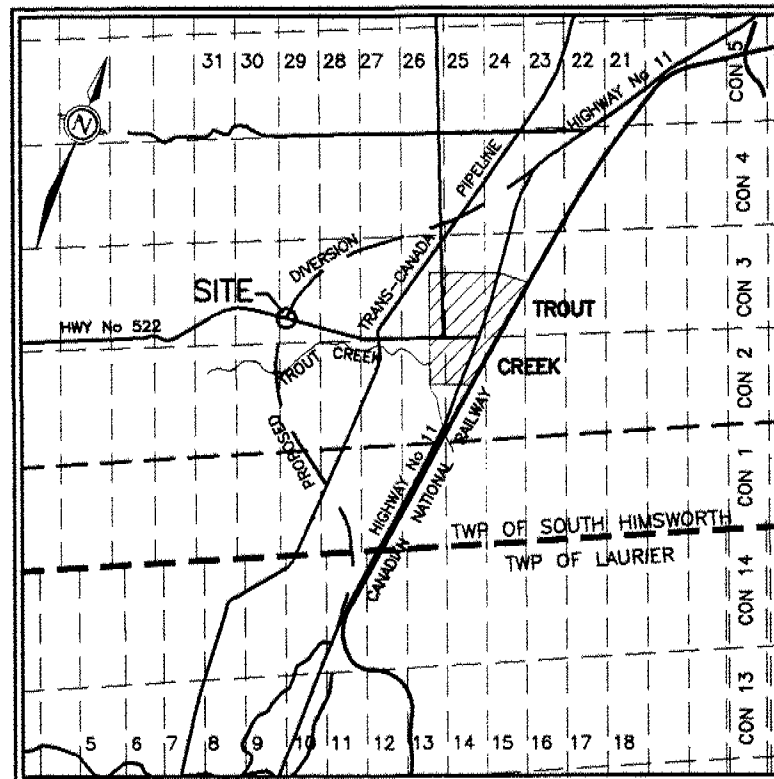
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PLATE No 509-11/178-0  
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 WP No 774-93-00

SHEET



# KEY PLAN

1 km 0 1 km



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 SUDBURY, ONTARIO

**Trow**

PROJ. No. S07524GCF DWG. No. 1

MINISTRY OF TRANSPORTATION  
 ENGINEERING OFFICE  
 SURVEYS AND PLANS SECTION

## KEY PLAN

PROPOSED CROSSING

AT

SEC HIGHWAY 522

AND

PROPOSED C/L MEDIAN HWY 11

GEOG TWP SOUTH HIMSWORD  
 LOT 29

DIST OF PARRY SOUND  
 CON 3

SCALE  
 1:400

DISTRICT  
 PARRY SOUND

REGION  
 NORTHERN

ETR  
 509-11

SURVEY DATE 97/10

PLAN DATE 97/10

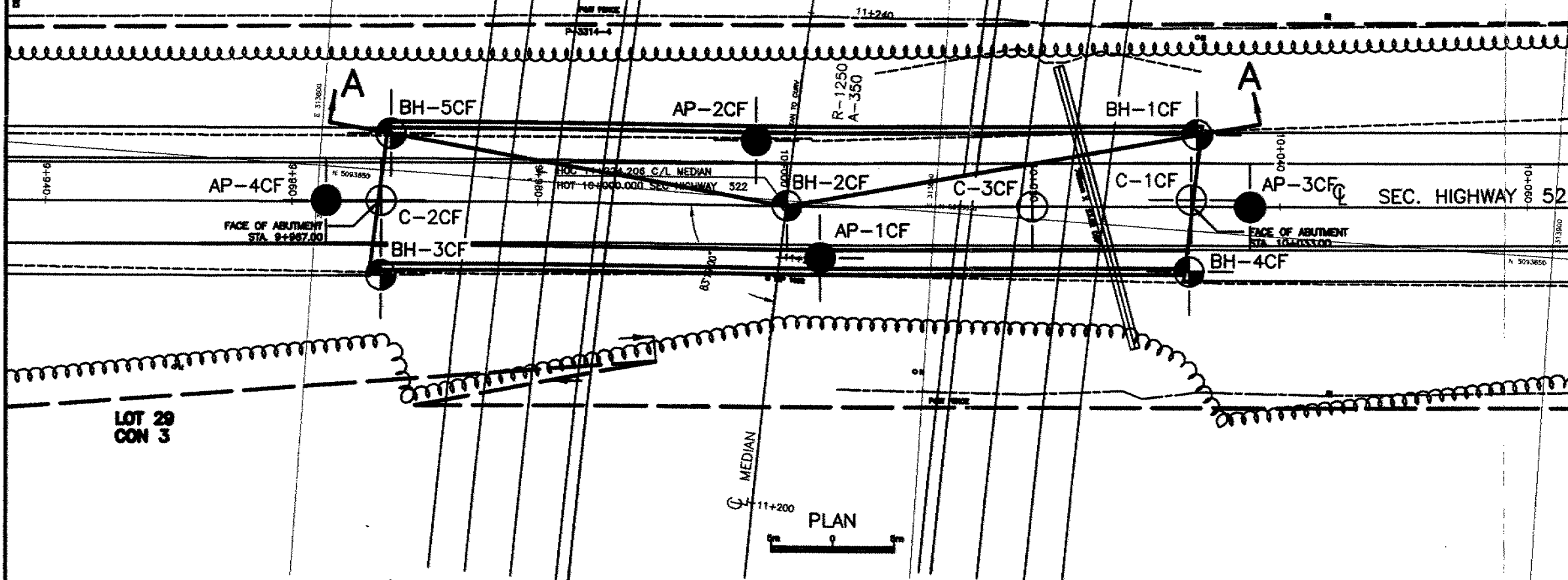
SITE 44-370

PLANE-509-11-12

DEAN AND STANTON LTD.

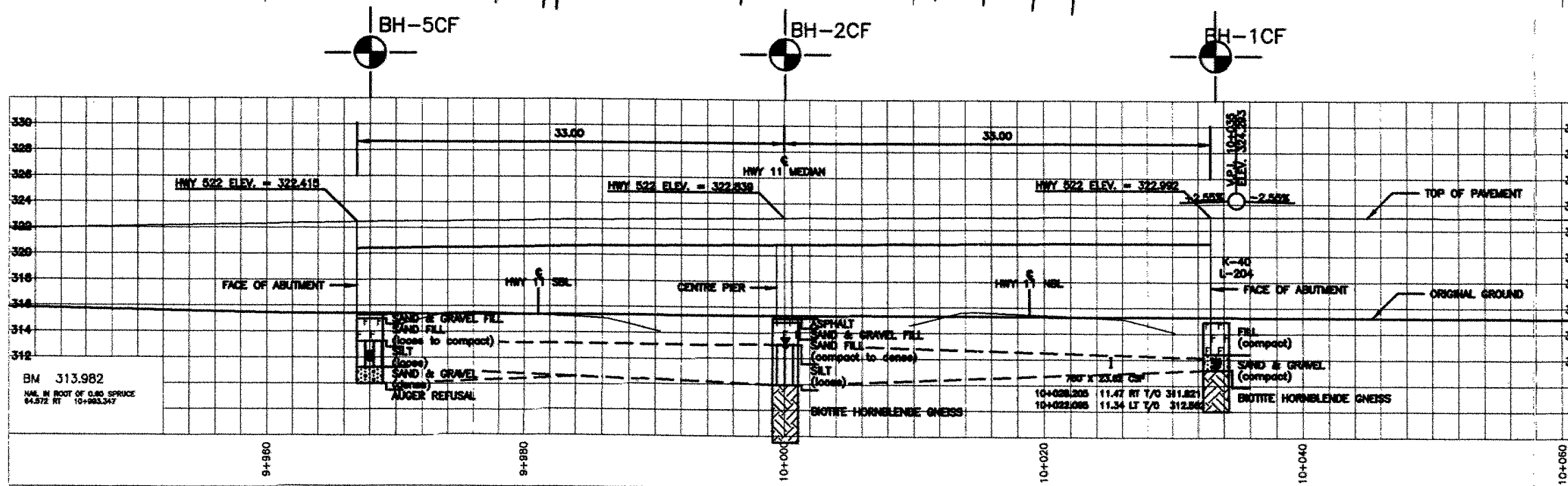
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CONT No  
WP No 774-93-00

SHEET



LEGEND			
	BOREHOLE		
	AUGER PROBE		
	DYNAMIC CONE PENETRATION TEST		
	GROUND WATER LEVEL		
No.	ELEV.	CO-ORDINATES	
		NORTH	EAST
BH-1CF	314.90	5 093 657.8	313 870.8
BH-2CF	315.13	5 093 649.6	313 838.0
BH-3CF	315.37	5 093 641.5	313 805.6
BH-4CF	314.98	5 093 648.7	313 870.9
BH-5CF	315.24	5 093 653.0	313 805.6
C-1CF	315.14	5 093 652.5	313 870.7
C-2CF	315.67	5 093 647.5	313 805.2
C-3CF	315.18	5 093 651.0	313 857.8
AP-1CF	315.93	5 093 645.6	313 841.0
AP-2CF	314.93	5 093 654.7	313 835.1
AP-3CF	315.16	5 093 652.3	313 875.4
AP-4CF	315.74	5 093 647.2	313 800.8

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SECTION A-A  
ON PROFILE OF SEC HIGHWAY 522

TROW CONSULTING ENGINEERS LTD.  
SUDBURY, ONTARIO  
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MINISTRY OF TRANSPORTATION  
ENGINEERING OFFICE  
SURVEYS AND PLANS SECTION

SECTION A-A

PROPOSED CROSSING  
AT  
SEC HIGHWAY 522  
AND  
PROPOSED C/L MEDIAN HWY 11

GEOG TWP SOUTH HIMS WORTH DIST OF PARRY SOUND  
LOT 29 CON 3

SCALE 1:400	DISTRICT PARRY SOUND	REGION NORTHERN
ETR 509-11		
SURVEY DATE 97/10	PLAN DATE 97/10	
SITE 44-370	PLANE-509-11-12	

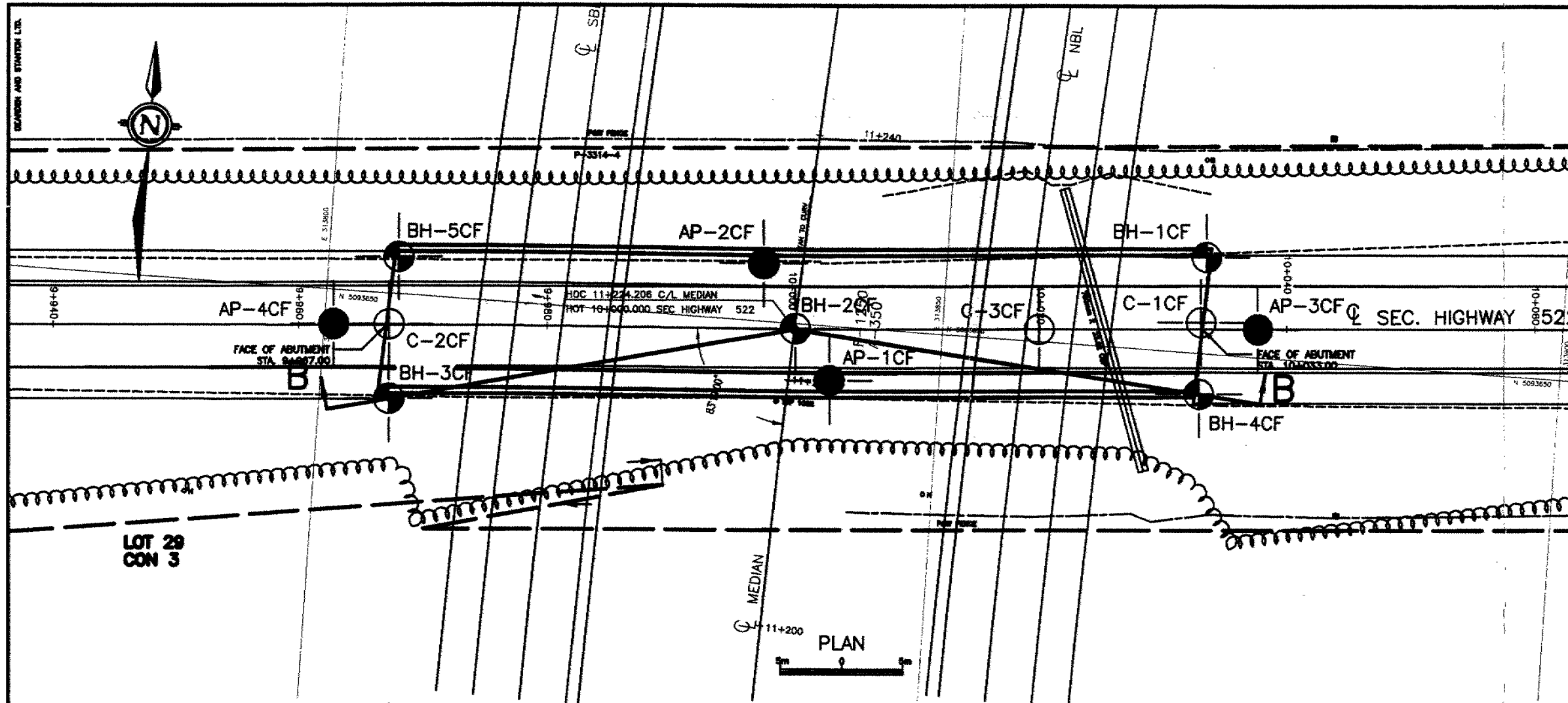
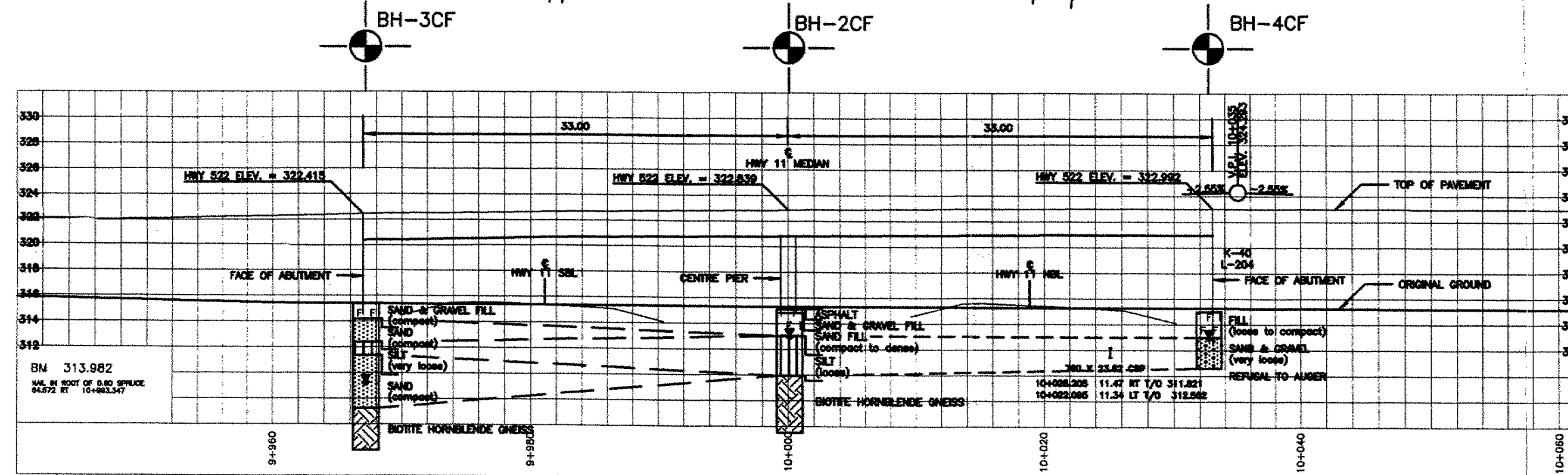


PLATE No 509-11/178-0  
 DRAWING No 05090011178  
 CONT No  
 WP No 774-93-00

SHEET

LEGEND				
	BOREHOLE			
	AUGER PROBE			
	DYNAMIC CONE PENETRATION TEST			
	GROUND WATER LEVEL			
No.	ELEV.	CO-ORDINATES		
		NORTH	EAST	
BH-1CF	314.90	5 093 657.8	313 870.8	
BH-2CF	315.13	5 093 649.6	313 838.0	
BH-3CF	315.37	5 093 641.5	313 805.6	
BH-4CF	314.98	5 093 648.7	313 870.9	
BH-5CF	315.24	5 093 653.0	313 805.6	
C-1CF	315.14	5 093 652.5	313 870.7	
C-2CF	315.67	5 093 647.5	313 805.2	
C-3CF	315.18	5 093 651.0	313 857.8	
AP-1CF	315.93	5 093 645.6	313 841.0	
AP-2CF	314.93	5 093 654.7	313 835.1	
AP-3CF	315.16	5 093 652.3	313 875.4	
AP-4CF	315.74	5 093 647.2	313 800.8	



SECTION B-B  
ON PROFILE OF SEC HIGHWAY 522

METRIC  
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 AND/OR MILLIMETRES  
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TROW CONSULTING ENGINEERS LTD.  
 SUDBURY, ONTARIO  
 PROJ. No. S07524GCF DWG. No. 1B

MINISTRY OF TRANSPORTATION  
 ENGINEERING OFFICE  
 SURVEYS AND PLANS SECTION

SECTION B-B

PROPOSED CROSSING  
 AT  
 SEC HIGHWAY 522  
 AND  
 PROPOSED C/L MEDIAN HWY 11

GEOG TWP SOUTH HINSDALE DIST OF PARRY SOUND  
 LOT 29 CON 3

SCALE 1:400	DISTRICT PARRY SOUND	REGION NORTHERN
ETR 509-11		
SURVEY DATE 97/10	PLAN DATE 97/10	
SITE 44-370	PLANE-509-11-12	

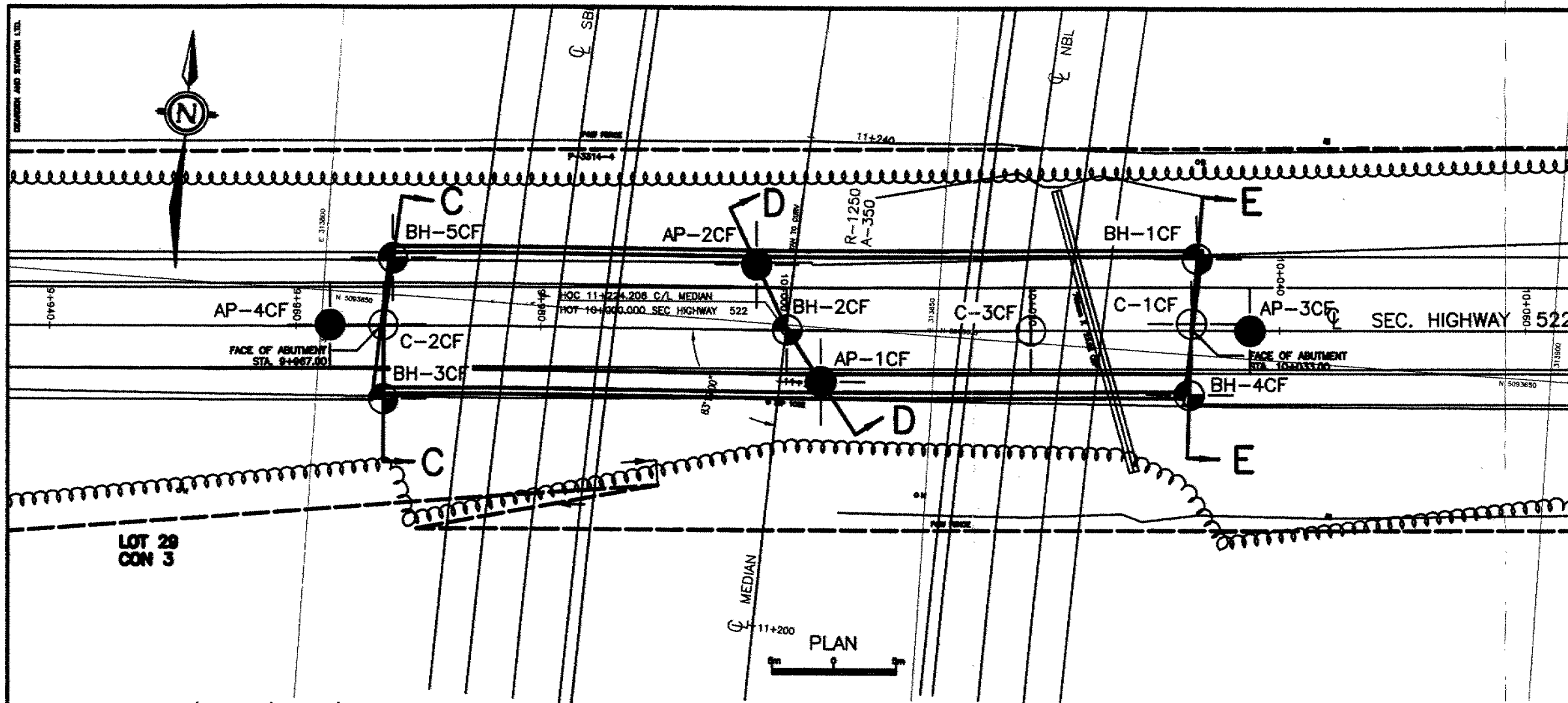
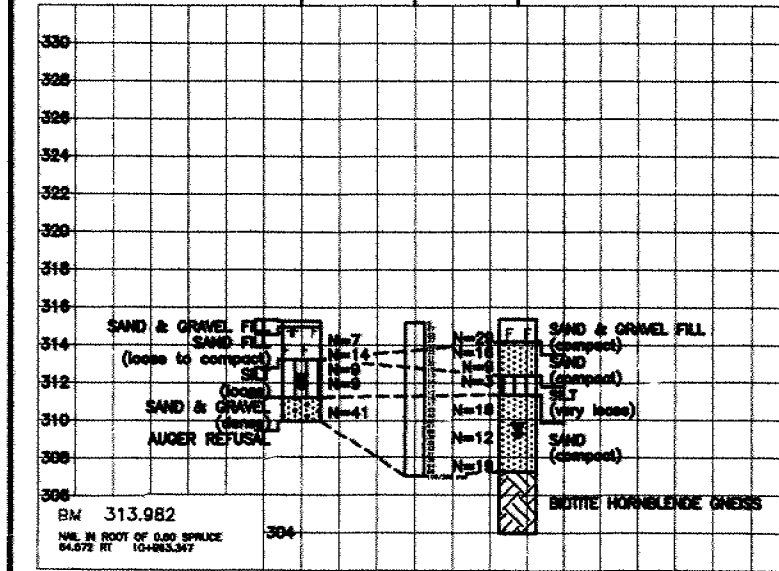
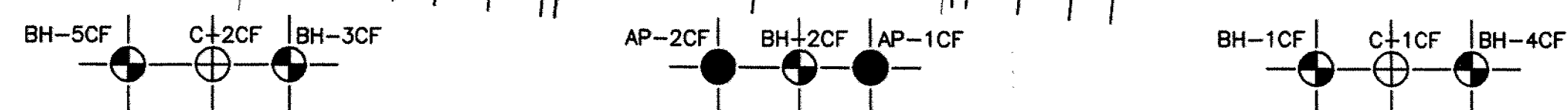
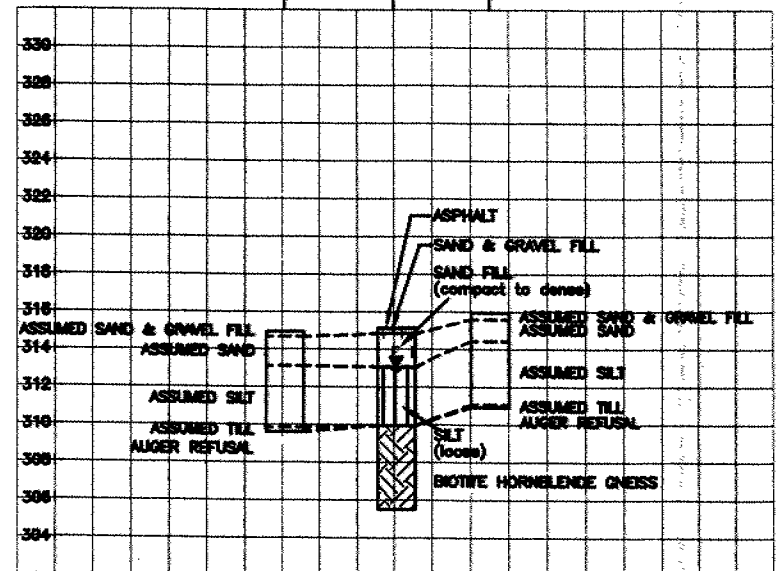


PLATE No 509-11/178-0  
 DRAWING No 05090011178  
 CONT No  
 WP No 774-93-00  
 SHEET

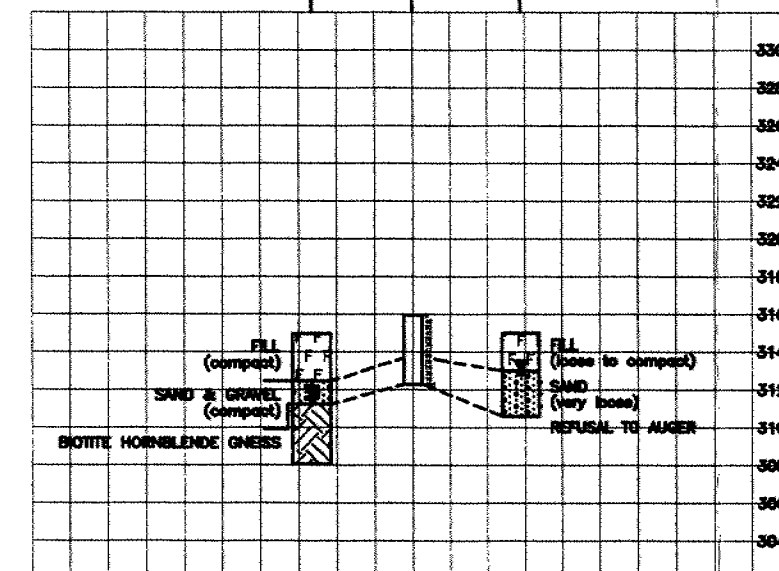
LEGEND				
	BOREHOLE			
	AUGER PROBE			
	DYNAMIC CONE PENETRATION TEST			
	GROUND WATER LEVEL			
No.	ELEV.	CO-ORDINATES		
		NORTH	EAST	
BH-1CF	314.90	5 093 657.8	313 870.8	
BH-2CF	315.13	5 093 649.6	313 838.0	
BH-3CF	315.37	5 093 641.5	313 805.6	
BH-4CF	314.98	5 093 646.7	313 870.9	
BH-5CF	315.24	5 093 653.0	313 805.6	
C-1CF	315.14	5 093 652.5	313 870.7	
C-2CF	315.67	5 093 647.5	313 805.2	
C-3CF	315.18	5 093 651.0	313 857.8	
AP-1CF	315.93	5 093 645.6	313 841.0	
AP-2CF	314.83	5 093 654.7	313 835.1	
AP-3CF	315.16	5 093 652.3	313 875.4	
AP-4CF	315.74	5 093 647.2	313 800.8	



SECTION C-C



SECTION D-D



SECTION E-E

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

TROW CONSULTING ENGINEERS LTD.  
 SUDBURY, ONTARIO  
 PROJ. No. S07524GCF DWG. No. 1C  
 MINISTRY OF TRANSPORTATION  
 ENGINEERING OFFICE  
 SURVEYS AND PLANS SECTION  
 SECTION C-C, D-D & E-E  
 PROPOSED CROSSING  
 AT  
 SEC HIGHWAY 522  
 AND  
 PROPOSED C/L MEDIAN HWY 11  
 GEOG TWP SOUTH HINSWORTH DIST OF PARRY SOUND  
 LOT 29 CON 3  
 SCALE 1:400 DISTRICT PARRY SOUND REGION NORTHERN  
 ETR 509-11  
 SURVEY DATE 97/10 PLAN DATE 97/10  
 SITE 44-370 PLANE-509-11-12

**NOTES ON SAMPLE DESCRIPTIONS**

- All descriptions included in this report follow the I.S.S.M.F.E. as suggested in the Canadian Foundation Manual. The laboratory grain-size analysis also follows this classification system. Others may designate the unified classification system as their source; a comparison of the two is shown for your information. Please note that, with the exception of those samples where the grain-size analysis has been carried out, all samples are classified visually and the accuracy of visual examination is not sufficient to differentiate between the classification systems or exact grain sizing.

UNIFIED SOIL CLASSIFICATION	Fines (silt or clay)			Sand			Gravel			Cobbles																						
				Fine	Medium	Coarse	Fine	Coarse																								
	Clay	Silt			Sand			Gravel			Cobbles																					
		Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse																						
	I.S.S.M.F.E. SOIL CLASSIFICATION	Sieve Sizes																														
	0.001	0.002	0.003	0.004	0.006	0.008	0.01	0.02	0.03	0.04	0.06	0.08	0.1	0.2	0.3	0.4	0.6	0.8	1.0	2.0	3.0	4.0	6.0	8.0	10	20	30	40	60	80		
	Particle Size (mm)																															

- FILL:** Where fill is designated on the borehole log, it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of the site fill material. All fills should be expected to contain obstructions such as large concrete pieces of subsurface basements, floors, tanks, etc.; none of these may have been encountered in the borehole. Since boreholes cannot accurately define the contents of fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact and correct composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant on-going and future settlements. Some fill material may be contaminated by toxic waste that renders it unacceptable for deposition in any but designated land fill sites. Unless specifically stated, the fill on this site has not been tested for contaminants that may be considered hazardous. This testing and a potential hazard study can be carried out if you so request. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common but are not detectable using conventional geotechnical procedures.
- TILL:** The term till on the borehole logs indicate that the material originates from a geological process associated with glaciation. As a result of this geological process, the till must be considered heterogeneous in composition and, as such, may contain pockets and/or seams of material such as sand, gravel silt or clay. As till often contains cobbles (60 to 200 mm) or boulders (over 200 mm), contractors may encounter them during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size, or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited areas; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till material.

# NOTES ON SAMPLE DESCRIPTIONS (Cont'd)



Project No: S07524G/C

Drawing No: 2B

4. The following table gives a description of the soil based on particle sizes. With the exception of those samples where grain-size analyses have been performed, all samples are classified visually. The accuracy of visual examination is not sufficient to differentiate between this classification system or exact grain size.

Soil Classification		Terminology	Proportion
Clay	< 0.002 mm	"trace" (eg. trace sand)	1% - 10%
Silt	0.002 to 0.06 mm	"some" (eg. some sand)	10% - 20%
Sand	0.06 to 2 mm	adjective (eg. sandy)	20% - 35%
Gravel	2 to 60 mm	and (eg. and sand)	> 35%
Cobbles	60 to 200 mm	noun (eg. boulders)	> 35% and
Boulders	> 200 mm		main fraction

Classification system as suggested in the Canadian Foundation Engineering Manual, 3rd Edition, unless otherwise noted.

The compactness of cohesionless soils and the consistency of cohesive soils are defined by the following:

Cohesionless Soil		Cohesive Soil	
Compactness	Standard Penetration Resistance "N" Blows/0.3 m	Consistency	Undrained Shear Strength (kPa)
Very Loose	0 to 4	Very Soft	< 12
Loose	4 to 10	Soft	12 - 25
Compact	10 to 30	Firm	25 - 50
Dense	30 to 50	Stiff	50 - 100
Very Dense	Over 50	Very Stiff	100 - 200
		Hard	> 200

## 5. Rock Coring

Where rock drilling was carried out, the term RQD (Rock Quality Designation) is used. The RQD is an indirect measure of the number of fractures and soundness of the rock mass. It is obtained from the rock cores by summing the length of core recovered, counting only those pieces of sound core that are 100 mm or more in length. The RQD value is expressed as a percentage and is the ratio of the summed core lengths to the total length of core run. The classification based on the RQD value is given below.

RQD Classification	RQD
Very poor quality	< 25
Poor quality	25 - 50
Fair quality	50 - 75
Good quality	75 - 90
Excellent quality	90 - 100

$$\text{Recovery Designation \% Recovery} = \frac{\text{Length of Core Per Run}}{\text{Total Length of Run}} \times 100$$

# RECORD OF BOREHOLE BH-1CF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 093 657.8 N, 313 870.8 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow Stem Augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE May 12, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80				wp ——— w ——— wl					WATER CONTENT (%)  10 20 30 40
								SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL X FIELD VANE LAB SHEAR									
314.90	GROUND SURFACE															GR SA SI + CL	
0.00	FILL, mostly silty sand & gravel with a few cobble sizes, occasional pieces of asphalt, brown, moist. (compact)	F															
		F	1	SS	28												
312.40		F															
2.50	SAND & GRAVEL, pockets of sand, some cobble sizes & possible boulders, brown, wet. (compact)	S	2	SS	13												
311.18		S															
3.72	BIOTITE HORNBLENDE GNEISS, pinkish grey, excellent rock quality, unweathered.	G	3	NQ												Rec 100% RQD 98%	
		G	4	NQ												Rec 100% RQD 100%	
		G	5	NQ												Rec 100% RQD 100%	
308.01	END OF BOREHOLE																
6.89	Notes: 1) This borehole forms part of Highway 522 Underpass Foundation Investigation. 2) Borehole located at station 10+033.3, offset ~6.5 m left of centreline as referenced to Highway 522.																





# RECORD OF BOREHOLE BH-2CF 1 OF 1

METRIC

W.P. 774-93-00

LOCATION 5 093 649.6 N, 313 838.0 E

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Hollow Stem Augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE May 13, 1998

CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20	40	60	80					
315.13	GROUND SURFACE															
314.89	ASPHALT, ~50 mm thick over															
0.30	SAND & GRAVEL FILL															
	SAND FILL, some gravel		1	SS	35											0% 80% 20%
	inclusions, traces of organics,		2	SS	25											
	brown, moist.															
	(compact/dense)															
313.03																
2.10	SILT, fine sand inclusions in parts		3	SS	9											
	with occasional fine clay layers,															
	grey, wet.		4	SS	5											
	(loose)															
	Some cobble sizes at base.		5	SS	5											
309.95																
5.18	BIOTITE HORNBLLENDE GNEISS,		6	NQ												Rec 99% RQD 68%
	pinkish grey, fair to excellent rock															
	quality, slightly weathered to		7	NQ												Rec 100% RQD 75%
	unweathered.															
			8	NQ												Rec 100% RQD 93%
			9	NQ												Rec 100% RQD 82%
305.50																
9.63	END OF BOREHOLE															
	Notes:															
	1) This borehole forms part of															
	Highway 522 Underpass															
	Foundation Investigation.															
	2) Borehole located at station															
	10+000.0, on centreline as															
	referenced to Highway 522.															





# RECORD OF BOREHOLE BH-3CF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 093 641.5 N, 313 805.6 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow Stem Augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE May 12, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT		UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20    40    60    80		wp    —    w    wl				WATER CONTENT (%) 10    20    30    40
								SHEAR STRENGTH: Cu, KPa ● UNCONFINED QUICK TRIAXIAL    × FIELD VANE LAB SHEAR						
315.37	GROUND SURFACE													
0.00	SAND & GRAVEL FILL, occasional lumps of asphalt, brown, moist. (compact)	F	1	SS	29		315							
314.17			2	SS	16		314							
1.20	SAND, brown, wet, traces of organics, (possible FILL). (compact)		3	SS	9		313							
312.37			4	SS	3		312							
3.00	SILT, grey, trace of clay, wet. (very loose)		5	SS	18		311							
311.37			6	SS	12		310							
4.00	SAND, with gravel sizes, brown, wet occasional cobbles. (compact)		7	SS	19		309							
			8	NQ			308							
307.23			9	NQ			307							
8.14	BIOTITE HORNBLENDE GNEISS, pinkish grey, good to excellent rock quality, slightly weathered to unweathered.		10	NQ			306							
							305							
303.97							304							
11.40	END OF BOREHOLE													
<b>Notes:</b> 1) This borehole forms part of Highway 522 Underpass Foundation Investigation. 2) Borehole located at station 9+966.8, ~5.0 m right of centreline as referenced to Highway 522.														



# RECORD OF BOREHOLE BH-4CF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 093 646.7 N, 313 870.9 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow Stem Augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE May 13, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE		20	40	60	80					
314.98	GROUND SURFACE													
0.00	FILL, sand & gravel, occasional cobble sizes, some silt, brown, moist. (loose to compact)	TI	1	SS	4									
312.98		TI	2	SS	15									
2.00	SAND & GRAVEL, organic inclusions with wood, some cobbles, dark grey, wet. (very loose)	TI	3	SS	7									
		TI	4	SS	1									
310.59	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER													
4.39	Notes: 1) This borehole forms part of Highway 522 Underpass Foundation Investigation. 2) Borehole located at station 10+032.2, "5.8 m right centreline as referenced to Highway 522. 3) Borehole caved wet at "3.8 on completion.													



# RECORD OF BOREHOLE BH-5CF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 093 653.0 N, 313 805.6 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow Stem Augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE May 13, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20    40    60    80		wp ——— w ——— wl				
								SHEAR STRENGTH: Cu, KPa ● UNCONFINED QUICK TRIAXIAL    X FIELD VANE LAB SHEAR		WATER CONTENT (%) 10    20    30    40				
315.24	GROUND SURFACE													GR   SA   SI + CL
314.93	SAND & GRAVEL FILL	F					315							
0.30	SAND FILL, traces of organics, brown, moist. (loose to compact)	F	1	SS	7		314	⊗		○				2%   91%   7%
313.24			2	SS	14		313	⊗		○				
2.00	SILT, occasional seams of firm clay, brown to grey. (loose)		3	SS	9		312	⊗			○			0%   22%   78%
311.24			4	SS	9		311	⊗			○			
4.00	SAND & GRAVEL, brown, moist. (dense)		5	SS	41		310		⊗	○				8%   61%   31%
309.97														
5.27	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER													
Notes: 1) This borehole forms part of Highway 522 Underpass Foundation Investigation. 2) Borehole located at station 9+967.3, offset ~6.0 m left of centreline as referenced to Highway 522. 3) Borehole caved wet at ~4.8 m depth on completion.														



# RECORD OF BOREHOLE C-1CF

1 OF 1

METRIC

W.P. 774-93-00

LOCATION 5 093 652.5 N, 313 870.7 E

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Dynamic cone test / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE May 13, 1998

CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60	80	wp	w		
315.14 0.00	GROUND SURFACE Dynamic cone test only.														
311.48 3.66	END OF CONE TEST DUE TO BOUNCING REFUSAL ON BEDROCK OR BOULDER  Notes: 1) This cone test forms part of Highway 522 Underpass Foundation Investigation. 2) Cone test located at station 10+033.0, on centreline as referenced to Highway 522.														



# RECORD OF BOREHOLE C-2CF

1 OF 1

METRIC

W.P. 774-93-00

LOCATION 5 093 647.5 N, 313 805.2 E

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Dynamic cone test / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE May 13, 1998

CHECKED BY I.G.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)		CONE PENETRATION TEST		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR	wp					
315.67 0.00	GROUND SURFACE Dynamic cone test only.														
						315									
						314									
						313									
						312									
						311									
						310									
						309									
						308									
307.47 8.20	END OF CONE TEST DUE TO BOUNCING REFUSAL ON BEDROCK OR BOULDER  Notes: 1) This cone test forms part of Highway 522 Underpass Foundation Investigation. 2) Cone test located at station 9+967.0, on centreline as referenced to Highway 522.														



# RECORD OF BOREHOLE C-3CF

1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 093 651.0 N, 313 857.8 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Dynamic cone test / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE May 25, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m					
315.18 0.00	GROUND SURFACE Dynamic cone test only.						20 40 60 80					
315												
314												
313												
312												
311												
310.07 5.11	END OF CONE TEST DUE TO BOUNCING REFUSAL ON BEDROCK OR BOULDER  Notes: 1) This cone test forms part of Highway 522 Underpass Foundation Investigation. 2) Borehole located at station 10+020.0, on centreline as referenced to Highway 522. 3) Augered first 0.3 m through dense fill before driving cone test.											



# RECORD OF BOREHOLE AP-1CF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 093 645.6 N, 313 841.0 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard Augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE May 13, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT wp	NATURAL MOISTURE CONTENT w	LIQUID LIMIT wl	WATER CONTENT (%)	UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA SI + CL
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20	40	60	80						
315.93	GROUND SURFACE																
0.00 315.56 0.37	ASSUMED SAND & GRAVEL FILL																
	ASSUMED SAND						315										
314.41 1.52							314										
	ASSUMED SILT						313										
							312										
311.05 4.88	ASSUMED FILL						311										
310.93 5.00	END OF PROBE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER																
Notes: 1) This auger probe forms part of Highway 522 Underpass Foundation Investigation. 2) Auger probe located at station 10+002.5, offset 5.0 m left of centreline as referenced to Highway 522.																	



# RECORD OF BOREHOLE AP-2CF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 093 654.7 N, 313 835.1 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard Augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE May 13, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60					
314.93	GROUND SURFACE														
314.93 0.30	ASSUMED FILL														
	ASSUMED SAND														
313.10 1.83	ASSUMED SILT														
309.84 5.09	ASSUMED TILL														
309.80 5.33	END OR PROBE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER														
<p>Notes:</p> <p>1) This auger probe forms part of Highway 522 Underpass Foundation Investigation.</p> <p>2) Borehole located at station 9+997.5 offset 5.0 m left of centreline as referenced to Highway 522.</p>															





# RECORD OF BOREHOLE AP-3CF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 093 652.3 N, 313 875.4 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard Augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE May 13, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT				UNIT WEIGHT  KN/m³	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20    40    60    80				wp ——— w ——— wl					
								SHEAR STRENGTH: Cu, KPa ● UNCONFINED      × FIELD VANE QUICK TRIAXIAL      LAB SHEAR				WATER CONTENT (%) 10    20    30    40					
315.16	GROUND SURFACE																
0.00	ASPHALT, 45 mm over						315										
	ASSUMED SAND						314										
313.64 1.52	-----  ASSUMED SILT						313										
							312										
311.35 3.81	----- ASSUMED SILTY SAND & GRAVEL																
311.08 4.08	END OF PROBE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER																
<b>Notes:</b> 1) This auger probe forms part of Highway 522 Underpass Foundation Investigation. 2) Borehole located at station 10+037.0, on centreline as referenced to Highway 522.																	



## 1 OF 1

METRIC

W.P. 774-93-00

LOCATION 5 093 647.2 N. 313 800.8 E

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Standard Augers / CME-55

COMPILED BY M.D

**DATUM** Geodetic

DATE May 13, 1998

CHECKED BY I.G.

[illegible]

S07524G/C

**TABLE 1-1  
ROCK CORE DESCRIPTION**

BH#	Core Recovery				Core Description	
	Core #	Depth (m)	% CR*	% RQD**	Depth (m)	Description
<b>HIGHWAY 522 BRIDGE FOUNDATION</b>						
1-CF	3	3.72 to 4.72	100	98	3.72 to 6.89	<b>Biotite Horneblende Gneiss</b> - light grey to pinkish white, fine to medium grained, strong, unweathered, fractures very widely spaced, dipped at 80 to 90° from vertical, planar, smooth
	4	4.72 to 5.88	100	100		
	5	5.88 to 6.89	100	100		
2-CF	6	5.18 to 5.94	100	66	5.18 to 9.63	<b>Biotite Hornblende Gneiss</b> - light grey to pinkish white, with pegmatitic quartz inclusions, medium to coarse grained, strong, unweathered, fractures moderate to very close spread, dipping at 45° from vertical planar, smooth
	7	5.94 to 6.71	100	75		
	8	6.71 to 7.47	100	93		
	9	7.47 to 9.63	100	82		
3-CF	8	8.14 to 9.05	100	75	8.14 to 11.40	<b>Biotite Hornblende Gneiss (Garnetiferous)</b> , pinkish white to light grey, medium grained, strong unweathered, fractures moderately spaced, dipping at 0 to 10° and 80 to 90° from vertical, planar, smooth
	9	9.05 to 10.52	100	76		
	10	10.52 to 11.40	100	94		

\*CR

Core Recovery %

\*\*RQD

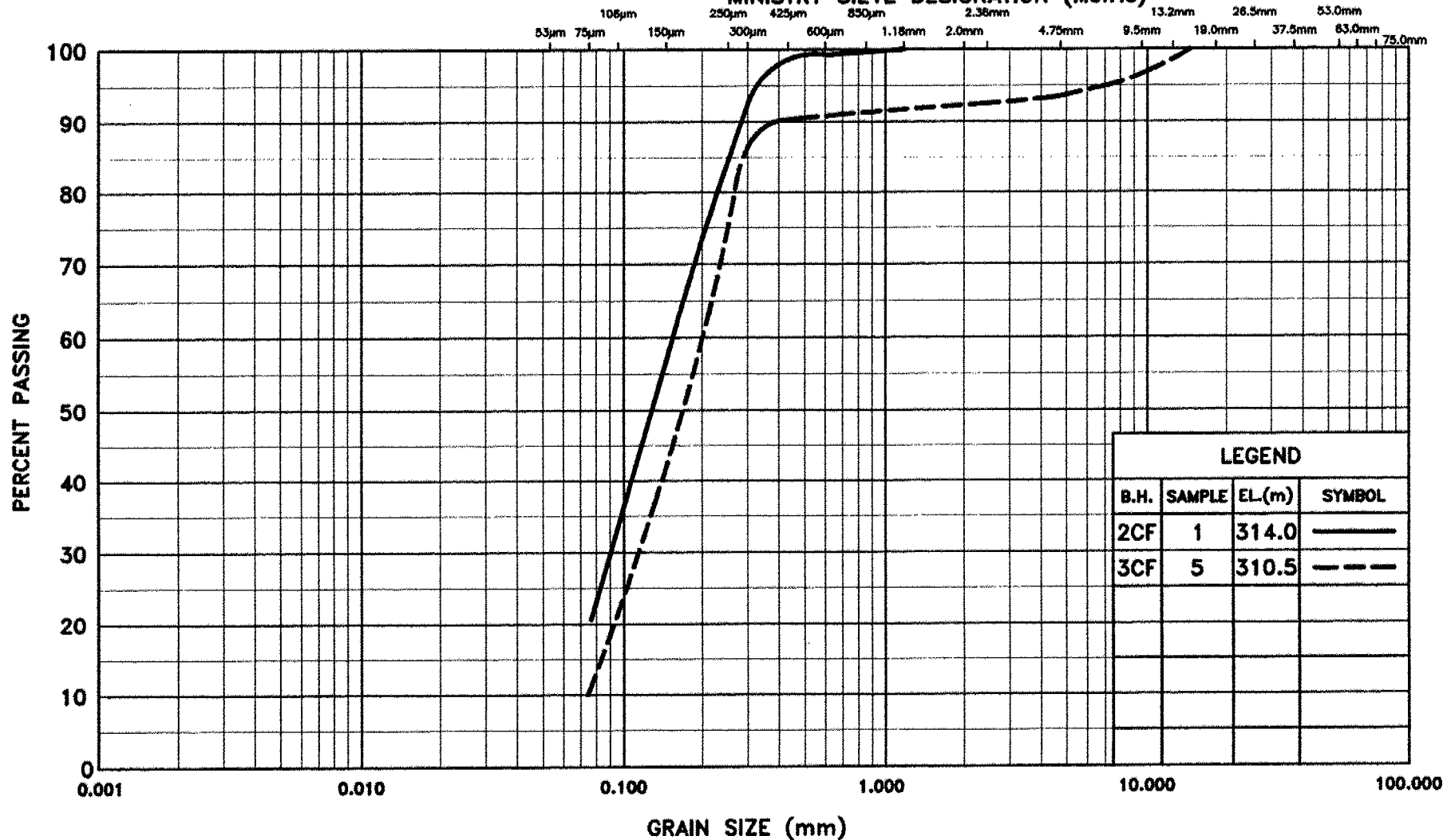
Rock Quality Designation %

## APPENDIX B

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION

FINE SAND

FIGURE 1

W.P. 774-93-00

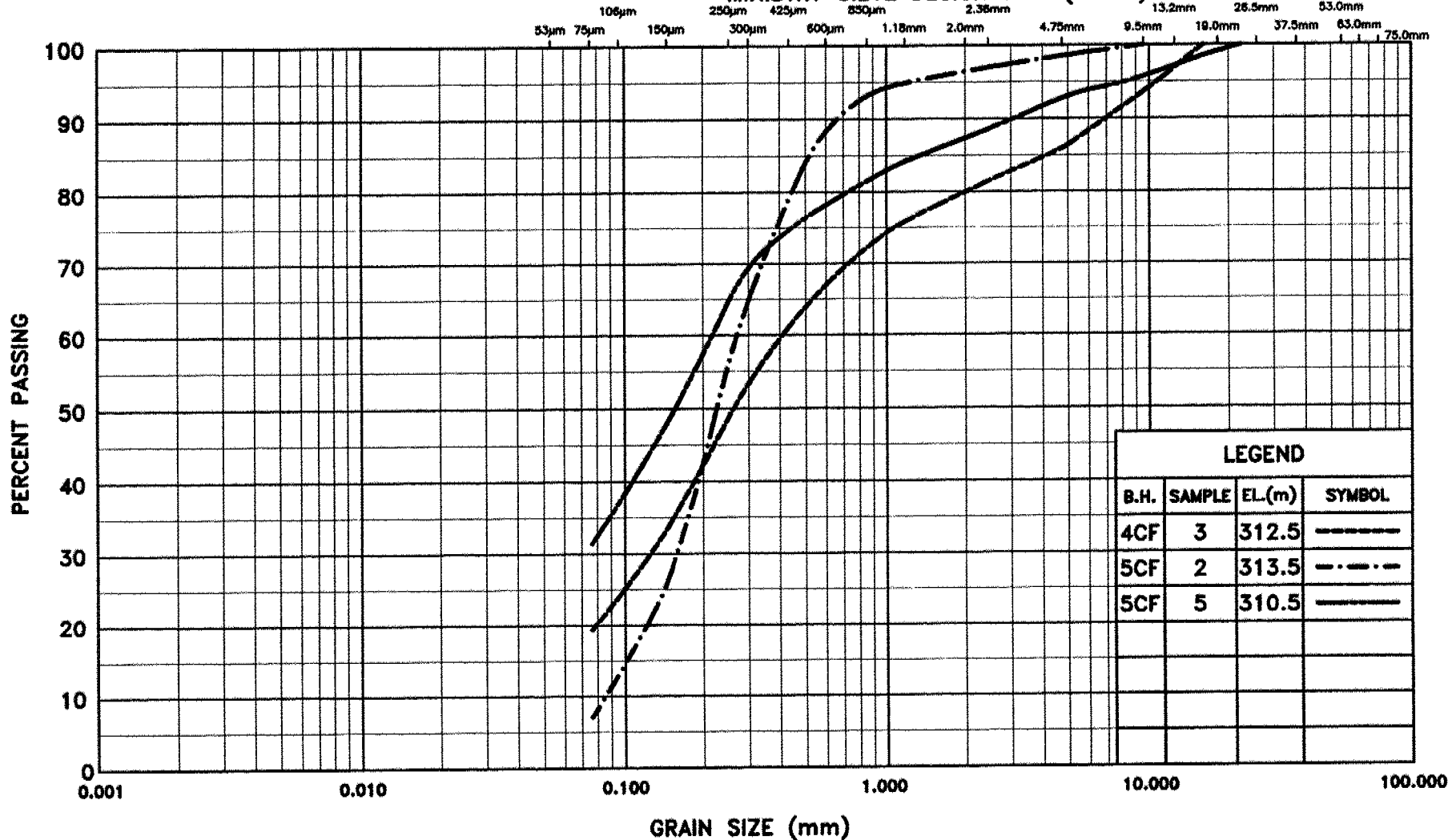


PROJ. No. S07524GCF

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION  
FINE/MEDIUM SAND, with gravel

FIGURE 2

W.P. 774-93-00

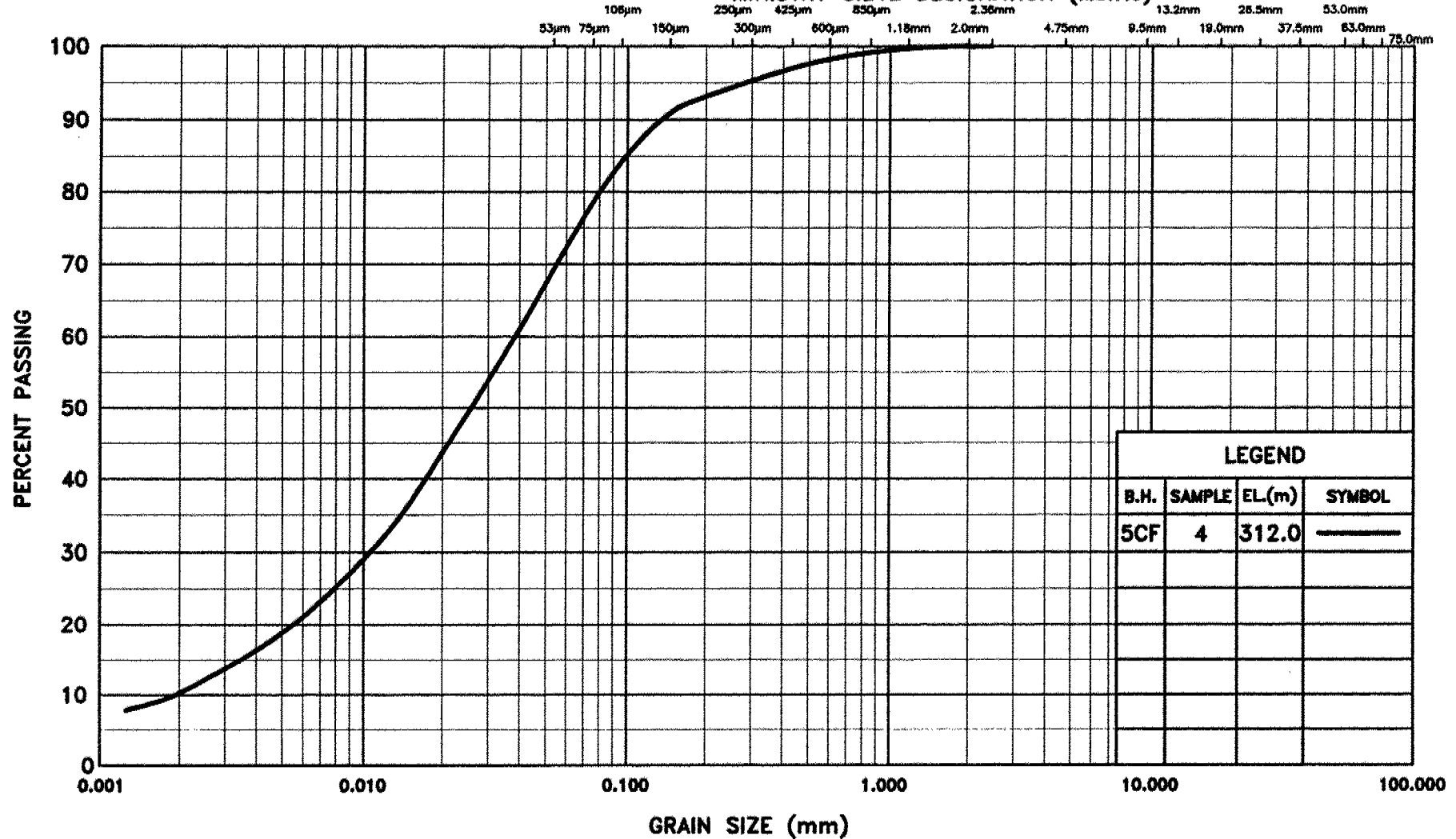


PROJ. No. S07524GCF

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

### GRAIN SIZE DISTRIBUTION

BH-5, SAMPLE 4: SILT, some sand, trace of clay

FIGURE 3

W.P. 774-93-00



PROJ. No. S07524GCF

# MEMORANDUM



To: E. Gallant, P. Eng.  
Senior Project Engineer  
Planning and Design, Northern Region

September 10, 1998

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

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

Re: Foundation Investigation Report Review  
Hwy 522, Bridge Structure  
Trout Creek By-Pass  
WP 774-93-00  
District 54, Sudbury

As requested we have reviewed draft reports prepared by Trow Consulting Engineers Ltd for the proposed Hwy 522 bridge structure and the Hwy 522 bridge structure approach embankments. 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. 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..

Two separate reports have been submitted: one for the bridge structure and one for the approach embankments. We have no comments for the Discussion and Recommendations component for the approach embankment report. Comments for the factual component of the report are applicable to both the bridge structure and approach embankment report.

## **Factual Component of Report(Hwy 522 Bridge Structure & Approach Embankment Reports)**

The Site Description and Geology, Investigation Procedure and Description of Subsurface and Groundwater Conditions have been partially addressed. The Consultant should, however, address the following:

1. Explanation of lab shear tests conducted on selected soil samples.
2. Borehole backfilling procedures.
3. Borehole Plan Drawing, stratigraphical profiles and drawings are not to MTO standards.
4. Section 1.4.1 - the report states that "...it is also believed to be fill or possibly alluvial in origin,...". The Consultant shall conclude whether the material is fill or not.(applicable to bridge report only)
5. Section 1.5 - Groundwater Conditions. Seasonal fluctuations are not addressed.



**Foundation Investigation & Design Report  
Bridge Structure & Approaches  
Trout Creek (Site 44-371S)  
SOUTHBOUND LANES  
Trout Creek By-Pass, King's Highway 11  
District 54, Sudbury, Ontario  
GWP No. 774-93-00**

Prepared For:

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Thornhill, Ontario  
L3T 7N4

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## Preface

Work Project GWP 774-93-00 is one of a series of projects for the four lane expansion of Highway 11. This project involves the four lane design of Highway 11, from 4.0 km south of Highway 522, northerly for 7.9 km. It will result in the construction of a by-pass of the existing Highway 11 to the west of the Town of Trout Creek.

This highway section is located in the Townships of Laurier and South Himsworth, within the geographic District of Parry Sound, as shown on the Key Plan, Figure A1 in Appendix A. The project requires geotechnical input for the following major components:

- New pavement design for the entire length of the four lane by-pass, including associated service roads.
- New structure, Trout Creek South Interchange (underpass), Site 44-372.
- New structure, Trout Creek Northbound Lanes, Site 44-371N.
- **New structure, Trout Creek Southbound Lanes, Site 44-371S.**
- New structure, Highway 522 (underpass), Site 44-370.
- New structure, Trout Creek North Interchange (underpass), Site 44-369.

This report deals with the new bridge structure for the **southbound lanes** at the proposed Trout Creek crossing, Site 44-371S, as well as the approach fills. Separate reports will be submitted for the additional components.

Part 1 of this report, Foundation Investigation, describes the geotechnical investigation methodology and the results derived, as pertaining to the design parameters required for the bridge foundations and related earthworks.

Part 2 of this report, Engineering Discussion and Recommendations, addresses the geotechnical engineering design considerations pertaining to the bridge foundations and the approaches.

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## Appendix A

Figure A1.	Key Plan
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Figure A5.	Estimated Consolidation Settlement - NORTH APPROACH

## Appendix B

Borehole Logs and Rock Descriptions

## Appendix C

Results of Laboratory Testing (grain size, Atterberg Limits, consolidation tests)

## Appendix D

Stability Analysis Printouts

## Rear Pockets

- Drawing No. 1. Bridge Site Plan & Profile
- Drawing No. 2. Bridge Site Plan & Sections

## **Part 1 Foundation Investigation**

### **1.1 Introduction**

This report presents the results of a geotechnical foundation investigation by Trow Consulting Engineers Ltd. (Trow) for the proposed bridge structure and approaches for the southbound lanes (SBL) at the Highway 11, Trout Creek crossing at Site 44-371S.

An original investigation was completed at this site in June and July of 1998 for an originally proposed 105 m long ( $\pm$ ) three span structure. However, for technical reasons based on the results of the original investigation, as discussed more fully in Part 2 of this report, a longer (242 m), five span bridge was selected. Accordingly, a supplementary investigation was completed during September and November, 1998 which investigated the subsurface conditions at the revised pier and abutment locations of the new proposed five span structure, where conditions were not previously investigated as part of the original work.

This report contains the results of the supplementary investigation, plus the relevant results of the original investigation, compiled for the five span structure arrangement of the Highway 11, southbound lane crossing of Trout Creek. It is Trow's understanding that the 5 span structure will be located with the central span crossing Trout Creek. The structure will include an approximately 14 metre high south abutment, 22 metre high centre span, and 7 metre high north abutment. The span lengths will be approximately 40, 54, 54, 54, 40 metres in length, creating a bridge with a length of about 242 metres.

### **1.2 Site Description and Geological Setting**

The site is located in Lot 30, Concession 2, Township of South Himsforth, District of Parry Sound, along the banks of Trout Creek, about 2 km west of the Town of Trout Creek, and 750 metres south of Highway 552, as shown on Figure A1, in Appendix A.

Generally, the terrain at the site is sloping towards the creek and is well drained. However several drainage gullies run parallel and perpendicular to the creek, with steep embankments on either side of the gullies, creating a highly variable terrain in the vicinity of the proposed structure. The relief

can vary at slopes steeper than 1H:1V within the site. There are mature trees with heavy underbrush across the site.

According to OGS Maps 2544 and 2556, as well as Ontario Geological Survey Map P.3160 (Quaternary Geology, South River Area), the site is located in what is known as the Central Gneiss Belt, with mainly felsic igneous rocks of the Mesoproterozoic Group.

The overburden soils have been mapped as glaciolacustrine deposits consisting of laminated, rhythmically bedded (also referred to as varved) to massive silts and clays. The southernmost portion of the site edges onto an area mapped as a bedrock drift complex with relatively thin, primarily cohesionless soil cover (till).

## 1.3 Investigative Procedures

### 1.3.1 General

Part 1 of this report describes the investigative procedures used for the geotechnical assessment of the southbound lanes crossing of Trout Creek, within the Trout Creek By-Pass, King's Highway 11. Properties of the overburden soils were obtained by *in situ* and laboratory testing and the procedures employed during the investigation are described below.

### 1.3.2 Field Investigation

The initial field investigation, or explorations, for the originally proposed three span arrangement was carried out between June 9 and July 2, 1998. Originally these explorations investigated only the pier and abutment locations, as well as the immediate approach embankments. With the discovery of clay at this site additional explorations were advanced, while initially on site, to outline the vertical and areal extent of the clay within the approach embankments. A further investigation of the north approach was completed September 23 and 24, 1998. The supplementary investigation of the revised, five span, arrangement occurred between November 10 and 24, 1998.

The locations of the boreholes, cones and probes, completed as part of these investigations are shown on Figures A2a and A2b, in Appendix A, as well as on Drawings, No. 1 and No. 2, located in the pockets at the end of this report. These locations, as well as the surface elevations, were established

from the terrain model for the project, and/or a survey crew from Marshall Macklin Monaghan, and are referenced to geodetic datum.

The investigation of the piers and abutments of the five span arrangement includes 12 boreholes (BH-1OF to BH-10OF, BH-4EF, BH-20EF), eight dynamic cone penetration tests (C-1EF, C-6EF, C-11OF to C-14OF, C-4EF, C-7EF), and nine auger probes (AP-1OF to AP-8OF, AP-1EF). All of these explorations were advanced to auger refusal or to refusal of the penetration cone ( $N > 100$ ). Six of the boreholes were advanced into the bedrock to obtain core, as described below.

The investigation of the approaches included two additional boreholes (BH-17EP, BH-18EP) at the south approach and four additional boreholes (BH-20EP to BH-23EP) at the north approach. All of these boreholes were advanced to auger refusal, except for BH-23EP, which was terminated at a depth of about 14 m.

Other explorations completed in the vicinity of the SBL structure as part of the original three span investigation included nine boreholes, three dynamic cone penetration tests, and one auger probe, in addition to those explorations completed as part of the NBL investigations (see foundation report for the NBL structure).

The boreholes, cones and probes were advanced through the overburden soils using a Bombardier mounted CME-55 drill and a Dietric 50 drill, equipped with solid and hollow stem augers, and supplied by a soils drilling contractor, Master Soil Investigations Limited. At some of the borehole locations, a bulldozer was used to prepare the site for drill access.

Soil samples were obtained by using a 51 mm OD split-spoon sampler in conjunction with standard penetration tests at approximately 0.75 m and 1.5 m intervals. The standard penetration ( $N$ ) values, together with the blows from the dynamic cone penetration tests, were recorded and used to provide an assessment of the compactness or consistency of the overburden soils. Sampling and testing procedures were in general accordance with ASTM D1586.

Several undisturbed, nominal 50 mm diameter, 'Shelby' tube samples were also obtained in the cohesive deposits. Field vane testing was completed in the boreholes and the auger probes throughout the cohesive soils to measure the *in situ* undrained shear strength of the cohesive soils. The field vane used had dimensions of 152 mm long by 70 mm diameter, not including the pointed 45° point. Torque measurement was by using two calibrated scales on a calibrated lever arm threaded to the drill rods. The testing was in general accordance with ASTM D2573.

The recovered soil samples were taken to Trow's Sudbury laboratories for additional examination, identification and laboratory testing.

At each bridge foundation element, conventional rock coring techniques were used to advance one of the explorations approximately 3 metres into the underlying bedrock. At Pier WP5, two boreholes were cored. A BQ size core barrel and casing was used and core samples of the bedrock were retrieved for rock quality determination and classification. The seven boreholes advanced into the underlying bedrock included BH-2OF, BH-3OF, BH-5OF, BH-7OF, BH-8OF, BH-4EF, and BH-9OF.

Details of the soil and bedrock conditions encountered in the boreholes are included on the logs in the attached Appendix B.

### **1.3.3 Laboratory**

The laboratory testing program for selected soil samples consisted of the following:

- natural moisture content
- unit weight
- grain size distribution
- laboratory shear tests (lab vane, penetrometer)
- Atterberg limits
- 1-d consolidation test

The laboratory test results are summarized on the attached borehole logs in Appendix B and are also presented in Appendix C. Many of the results are also discussed in the following descriptive sections.

## **1.4 Subsurface Conditions**

The borehole locations are shown on the site plans, Figures A2a and A2b in Appendix A, as well as on Drawings, No. 1 and No. 2. Centerline soil profiles are also shown on the Figures and Drawing No. 1 and the soil cross sections at the locations of the foundation elements are shown on Drawing No. 2.





The borehole, auger probe and dynamic cone penetration (CPT) logs are provided in Appendix B. In general, the following main soil layers were encountered, with increasing depth:

- topsoil
- sand
- silty sand
- silty clay
- silty sand and gravel
- bedrock

A summary of the soil strata encountered in the boreholes, and inferred from the probes and dynamic cone penetration tests, is presented below.

#### **1.4.1 Topsoil**

The majority of boreholes encountered a surficial layer of up to about 300 mm of topsoil. At some of the borehole locations, a bulldozer was used to prepare the site for drill access, thus the topsoil in these areas was removed in the process and the logs do not indicate the presence of topsoil.

#### **1.4.2 Sand**

A 300 mm to 600 mm thick layer of loose to compact, fine to medium grained brown sand was encountered beneath the topsoil at Boreholes, BH-9OF and BH-10OF, at the location of the north abutment, and was underlain by silty clay, as described below.

#### **1.4.3 Silty Sand**

Brown to grey, very loose to loose (N-values of 1 to 5) silty sand was encountered beneath the topsoil in Boreholes, BH-6OF and BH-7OF at the location of Pier WP4, adjacent to the north bank of Trout Creek. The soil contained organics such as roots and pieces of wood and may be a recent stream alluvium. At BH-6OF, the thickness was about 4 m and was underlain by the silty sand and gravel, described below. In BH-7OF, the silty sand was underlain by silt clay at a depth of about 2.3 m.

#### **1.4.4 Silty Clay**

Beneath the upper sand, topsoil, or silty sand, a stratum of silty clay was encountered as the principal soil in all boreholes at the locations of the foundation elements except at the south abutment (WP1),

BH-4OF at Pier WP2, and BH-6OF at Pier WP4. At the south abutment, the native soil consists of silty sand and gravel, described below. Silty sand and gravel was also encountered in BH-4OF (Pier WP2). In BH-6OF (Pier WP4), silty sand, as described above, was encountered.

The silty clay, where encountered, is relatively thin (1.5 m) in BH-3OF at the location of Pier WP2, and increases in thickness to between about 12 m to 14 m on the north side of Trout Creek, in the area of the north abutment and approach.

Generally, the silty clay is thinly laminated with silt (varved). The individual layers, or laminations, varied in thickness from a few millimetres to a few centimetres, but in general were less than about one centimetre. The proportions of clay to silt and clayey silt varied also, but in general the clay portion dominated.

The natural moisture content of the clay varies from about 20% to over 50% (depending on the silt content) and consistency. Atterberg limit determinations provided the following approximate ranges: Plastic Limit, 18 to 23; Liquid Limit, 28 to 50; Plasticity Index, 8 to 30. These data indicate that, in general, the clay can be described as a medium plasticity silty clay (CI). Locally, clayey silt soil (CL) was encountered. Typically, the silty clay is drier near the top and bottom, but there does not appear to be a depth relationship relative to the Atterberg Limits. The laboratory test data are shown on the borehole logs, on Figure A3 in Appendix A, and in Appendix C.

Standard penetration test (SPT) values ranging from about 1 to over 20 were obtained in the silty clay. The higher values were generally obtained within the upper metre, or so. *In situ* field vanes and laboratory shear vane testing, as well as the SPT values, indicate that the silty clay has a stiff to very stiff crust with undrained shear strengths exceeding 100 kPa. The silty clay consistency, however, reduces with depth to soft to firm to stiff, with undrained shear strengths of about 20 kPa to 70 kPa, at depths of about 3 m to 5 m. Sensitivity, the measure of peak shear strength to remoulded shear strength, ranged from about 2 to 16, with an average of about 7, indicating the clay is moderately sensitive. The undrained shear strength profile which includes shear strength data from all boreholes is shown on Figure A4. The strength profile shown on Figure A3 is based on the strength data from boreholes at the locations of the abutments and the design profile assumed is based on the actual vane test results and the SPT values.

Based on all the above, and with reference to Figure A3, it is evident that the clay is heavily overconsolidated in the upper 3 m to 4 m, becoming moderately to lightly overconsolidated with increasing depth. The preconsolidation pressure near the top of the stratum is estimated at about 400 kPa, on average. The overconsolidation ratio (OCR) is estimated as ranging from over 30 near

the top to about 3 at a depth of about 3 m. Thereafter with increasing depth, the OCR decreases gradually to about 1.6 at a depth of 14 m.

A one-dimensional consolidation test was performed on a sample of the silty clay extruded from a thin walled Shelby tube, obtained from BH-21EP. The results are presented graphically and numerically in Appendix C. The data are also summarized below in Table 1-1, along with the value ranges used in our subsequent analyses.

<b>Table 1-1. Consolidation Parameters for Silty Clay</b>		
	<b>BH-21EP, 3 m</b>	<b>Values for Analyses</b>
Compression ratio, $C_c' (= C_c/(1+e_0))$	0.08	0.08 - 0.20
Recompression ratio, $C_r' (= C_r/(1+e_0))$	0.006	0.008 - 0.02
Coefficient of consolidation (recompression), $C_{vr}$	40	25 - 60 (avg 40)
Coefficient of consolidation (virgin), $C_v$	7	5 - 12 (avg 8)
Coefficient of secondary compression, $C_{\alpha\epsilon}^*$	0.003 - 0.004	0.003 - 0.005
Notes: Coefficients of consolidation in units of $m^2/year$		
* $C_{\alpha\epsilon} = \Delta H/H_0$ (= secondary compression ratio, in some literature)		

#### 1.4.5 Silty Sand and Gravel

A basal deposit of loose to dense, brown silty sand and gravel was encountered above the bedrock (or refusal) surface in almost all boreholes. Standard penetration indices (N-values) ranged from about 5 to over 100. Its thickness varied from less than about 0.5 m to over 3.5 m. Where bedrock is relatively shallow, such as at the south abutment, the sand and gravel was encountered as the uppermost soil overlying the bedrock.

#### 1.4.6 Bedrock

The presence of bedrock was established by retrieving "BQ" size cores at each of the six foundation element locations, for depths of between about 3.1 m and 3.5 m. Detailed descriptions of the rock are presented in Table 1-1 in Appendix B, following the borehole logs. Generally, the bedrock can be described as a pink and light grey, biotite-hornblende gneiss. The rock is strong and unweathered for the most part.

Rock core recovery was 100% for all runs and the Rock Quality Designation (RQD) values for individual core runs ranged from about 35% to 96%. The average RQD for the rock core recovered was about 80%, based on the 23 core runs. In accordance with the MTO classification system, the rock quality can be described as poor to excellent, with an average of fair. It is noted that the RQD values are likely conservative; it is expected that higher values would be obtained using NQ core.

Table 1-2, below, lists the bedrock depths and elevations as well as those of refusal, at the locations of each of the six foundation elements. It can be seen that the bedrock and refusal depths and elevations are quite variable, even within short distances at the individual element locations. Refusal (to augering or dynamic cone penetration testing (CPT)) is inferred to be due to probable bedrock, but it is noted that refusal may be due to cobbles, boulders, or very dense soil. The bedrock depths and elevations have been positively established only at the locations where the bedrock has been cored.

<b>Table 1-2. Bedrock and Refusal Depths and Elevations at Boreholes</b>					
<b>Location</b>	<b>Borehole</b>	<b>Orig. Gr. Elev. (m)</b>	<b>Bedrock or Refusal Elev (m)</b>	<b>Overburden Thickness (m)</b>	<b>Comment RQD (by run)</b>
<b>WP1 South Abutment</b>	BH-1OF	310.76	307.56	3.20	auger refusal
	BH-2OF	311.74	308.62	3.12	B/R cored 80%, 80%, 90%
	AP-1OF	312.05	309.15	2.90	auger refusal
	AP-2OF	312.05	310.17	1.68	auger refusal
	AP-3OF	311.24	308.56	2.68	auger refusal
	AP-4OF	311.83	309.09	2.74	auger refusal
<b>WP2 Pier</b>	BH-3OF	305.90	302.55	3.35	B/R cored 75%, 97%
	BH-4OF	306.84	303.94	2.90	auger refusal
	AP-5OF	306.65	302.54	4.11	auger refusal
	AP-6OF	307.39	304.71	2.68	auger refusal
	AP-7OF	306.10	302.38	3.72	auger refusal
	AP-8OF	305.77	302.72	3.05	auger refusal
<b>WP3 Pier</b>	BH-5OF	303.79	298.52	5.27	B/R cored 35%, 50%, 60%
	AP&C-1EF	301.02	295.84	5.18	auger/cone refusal
	C-6EF	300.50	295.90	4.60	CPT refusal

Table 1-2. Bedrock and Refusal Depths and Elevations at Boreholes					
Location	Borehole	Orig. Gr. Elev. (m)	Bedrock or Refusal Elev (m)	Overburden Thickness (m)	Comment RQD (by run)
WP4 Pier	BH-6OF	299.89	293.95	5.94	auger refusal
	BH-7OF	300.22	293.90	6.32	B/R cored 70%, 92%
	C-11OF	300.01	293.33	6.68	CPT refusal
	C-12OF	300.16	293.94	6.22	CPT refusal
	C-13OF	300.57	293.96	6.61	CPT refusal
	C-14OF	300.12	294.06	6.06	CPT refusal
WP5 Pier	BH-8OF	303.23	294.94	8.29	B/R cored 65%, 78%
	BH-4EF	305.36	295.45	9.91	B/R cored 80%, 90%
	BH-20EF	303.20	293.90	9.30	auger refusal
	C-7EF	303.70	294.51	9.19	CPT refusal
WP6 North Abutment	BH-9OF	312.07	297.65	14.42	B/R cored 82%, 93%
	BH-10OF	311.62	297.96	13.66	auger refusal

## 1.5 Groundwater Conditions

Information regarding the groundwater levels at the site was obtained by measuring the water levels in the open boreholes after completion of drilling and by tactile examination of the recovered samples. The measured or inferred groundwater levels are shown on the borehole logs and the stratigraphic sections. In general, the groundwater table at the times of the field work was between about 1 m and 4 m in depth. It appears to follow the topography and this suggests that local subsurface drainage would be towards Trout Creek.

## Part 2 Engineering Discussion and Recommendations

### 2.1 Introduction

The following subsection addresses the geotechnical design considerations pertaining to the proposed five span bridge for the Southbound Lanes crossing of Trout Creek, as well as the approaches.

An original investigation was completed at this site in June and July of 1998 for an originally proposed 105 m long ( $\pm$ ) three span structure and revealed significant clay deposits on either side of Trout Creek which caused concern for the stability of the approach embankments, which were about 20 m high. Various design alternatives were considered including removal of the clay to the underlying bedrock, extensive berming of the embankments, the use of lightweight fill within the embankments, retaining walls, and a lengthening of the structure to limit the impact of the clay deposits. A subsequent cost benefit analysis of the design alternatives proposed by Trow, performed by Marshall Macklin Monaghan, indicated the preferred alternative was to lengthen the structure.

It was considered that the lengthened structure would effectively span the clay deposit, so that the approach embankments could either be located on a thinner or absent clay layer, or alternatively the lengthened structure would extend to the point where the height of the approach embankments could be reduced, and thus reduce or eliminate the complications encountered by placing high approach embankments on thick clay deposits.

Upon MTO acceptance of the lengthened structure alternative, a supplementary investigation was completed during September and November, 1998 as described in Part 1 of this report. The supplementary investigation examined the soil conditions at the revised pier and abutment locations of the new proposed five span structure, where conditions were not previously investigated as part of the original work.

The five span bridge is proposed to carry southbound Highway 11 traffic over Trout Creek and its valley. It is Trow's understanding that the bridge will be located with the central span crossing Trout Creek. The structure will include an approximately 14 metre high south abutment, 22 metre high centre span, and 7 metre high north abutment. The span lengths will be approximately 40, 54, 54, 54, 40 metres in length, creating a bridge with a length of about 242 metres.

It is understood that the project is scheduled for completion within a two year time frame. Accordingly, some of the discussions and design recommendations presented herein are based on this time frame.

## **2.2 Foundations**

In general, because of the presence of loose to compact sand and silt and relatively weak and compressible clay at the locations of the foundation elements, it is not considered feasible to support the proposed bridge foundations on spread footings constructed on the native mineral soils. For all foundation elements, driven steel H-piles are considered to be the preferred alternative. Alternate types of foundations may, however, be considered for the support of the bridge piers and abutments. The alternate types that are considered applicable to the site and proposed layout include large diameter reinforced concrete caissons and spread footings on rock or structural fill. Not all of these foundation types are applicable to all six foundation elements.

The following sections present the foundation design recommendations for the six foundation elements of the proposed bridge.

### **2.2.1 Steel H-Piles (all locations)**

All abutments and piers are recommended to be supported on steel H-piles driven to the bedrock surface, using the ULS capacities for HP310x110 and HP310x132 sections, as given in Table 2-1, below.

Current design practice requires that consideration be given to downdrag loads that may be generated as a result of soil consolidation due to fills, groundwater table fluctuations and the soil stresses induced by pile installation. We consider that, even after primary consolidation of the clay soils due to the fill placement is complete, the potential exists for downdrag loads due to events such as significant groundwater lowering and the long term process of secondary compression. At this site, secondary compression is the likely mechanism. Secondary compression of the clay foundation soils is discussed further in a subsequent section of this report. Accordingly, we have considered the effects of downdrag loads on the pile capacities given in the following sections, where applicable.

<b>Table 2-1. H-Pile Design Pile Capacities (kN)</b>						
	<b>HP 310x110</b>			<b>HP 310x132</b>		
Factored Structural Capacity (OHBDC)	3285			3890		
Factored Axial Resistance (MTO*)	2000			2300 (est)		
Pile Location ---->	South Abutment W1	Piers W2, W3, W4, W5	North Abutment W6	South Abutment W1	Piers W2, W3, W4, W5	North Abutment W6
Factored Downdrag Load	-	-	1200	-	-	1500
<b>Factored Axial Capacity at ULS**</b>	<b>2000</b>	<b>2000</b>	<b>2000</b>	<b>2300</b>	<b>2300</b>	<b>2300</b>
Notes: MTO* = Structural Office Policy Memo 98-01, April 15, 1998 ** Factored axial capacity at ULS is the lesser of, a) factored structural capacity minus factored drag load, or b) factored axial resistance. SLS capacity not applicable to piles driven to bedrock						

The axial capacity at SLS is not applicable to steel piles driven to bedrock because very high loads, in excess of the ULS capacity, are required to produce unacceptable deformations. The pile structural capacity will govern.

The capacities given are axial capacities, for both vertical and inclined piles. No reduction is required for inclined piles. Notwithstanding the above, the horizontal and vertical loads acting on all piles must be resolved axially, such that the vector sum of these loads does not exceed the ULS capacity provided above.

For design purposes, the modulus of horizontal subgrade reaction for steel H-piles can be taken as 8,000 kN/m<sup>3</sup> for the cohesive soils (silty clays) and 40,000 kN/m<sup>3</sup> for the cohesionless soils. It is expected, however, that inclined piles will be required to accommodate the lateral loads. These can be designed using the same axial capacities given in Table 2-1.

A minimum soil embedment depth of 3 m below the pile cap is recommended. Pile caps should be provided with at least 2 m of soil cover for frost protection. Local grade raises may be required in order to provide this cover.

If the underside of the pile caps cannot be provided with a minimum of 2 m earth cover, insulation will be required. Insulation should consist of rigid board extruded polystyrene, meeting CAN/CGSB-51.20-M87 (Type 4), such as DOW SM™. The insulation is recommended to be placed



beneath the pile caps, prior to placement of concrete. Since the insulation will not carry any significant loads, high strength/low compressibility insulation (such as *DOW HI40™*, etc.) is not required. Products other than those made by *DOW CORNING* may be used, provided they meet the above noted specification.

The insulation thickness and lateral extension beyond the edges of the pile caps will depend on the depth of placement (i.e., underside of pile cap), in accordance with Table 2-2, below. A minimum soil cover of 300 mm is recommended over the top of the insulation.

<b>Table 2-2. Pile Cap Insulation Dimensions</b>		
<b>Depth (mm)</b>	<b>Thickness (mm)</b>	<b>Lateral Extension (mm)</b>
500	90	1500
1000	50	1000
1500	25	500

As discussed in following subsections of this report, substantial settlements of the north approach fill will occur as will some associated lateral strains. Accordingly, piles should not be installed until after the majority of the fill settlement has occurred, based on monitoring during construction. This is particularly important for inclined piles, that would be subjected to bending stresses.

All piles should be driven to bedrock. Pile tip elevations can be estimated from Table 1-2 which provides the bedrock or refusal elevations encountered at the boreholes drilled at the various foundation elements. The boreholes indicate that the bedrock elevations are quite erratic and the potential for irregular steeply sloping bedrock at the foundation locations is considered to be high at most locations. As such, problems may arise during pile seating. At some locations, the piles may have a tendency to skip over the bedrock surface resulting in alignment problems and deeper penetration. In the event that this problem occurs, somewhat longer piles may be required and, in some cases, piles may have to be added or replaced.

To minimize seating difficulties, rock injector points should be used to facilitate proper seating. All piles must be fitted with reinforcing plates welded to the flanges as per OPSD 3301 to minimize pile damage. It is recommended that, during pile driving and upon initial contact with the bedrock, the

pile driving energy should be reduced and subsequently increased incrementally until the piles have been sufficiently seated.

### 2.2.2 Concrete Caissons (all locations)

As an alternative foundation system, concrete caissons installed on or into the bedrock can be considered for all locations. However, they will likely only be practical for the foundations at the south abutment (WP1) and Piers WP2 and WP3, where the bedrock surface is generally within about 1.7 m to 5.3 m below original grade. The load capacity will be derived by end bearing, in accordance with the values given in Table 2-3. As for steel H-piles, the effects of downdrag loads must be considered.

<b>Table 2-3. Concrete Caisson Design Capacities</b>	
Factored Downdrag Load (north abutment only)*	3800 kN per m of pile diameter
Factored Bearing Resistance at ULS	8000 kPa
Notes: SLS resistance not applicable to caissons on bedrock * Factored Downdrag Load to be applied to the factored dead loads	

In order to provide an adequate socket, the caisson should be installed at least one pile diameter into the bedrock, or be heavily dowelled. While these units can provide high capacities, because of the irregular and potentially steeply sloping bedrock surface expected at this site, caisson installation may prove difficult, thus expensive. This is complicated by the fact that in most cases the bedrock is overlain by silty sand and gravel that may create dewatering and stability problems during work at the base of the caissons.

### 2.2.3 Spread Footings

Spread footings for the south abutment and Piers, WP2 and WP3, can be considered for design and construction on bedrock or on structural fill, as described in the following sections. We do not consider spread footings to be feasible alternatives for the remaining foundation elements, either because of a great depth of excavation or low SLS bearing resistance for footings near the ground surface.

### 2.2.3.1 Spread Footings on Bedrock (WP1, WP2, WP3)

An alternate foundation for the south abutment (WP1) and Piers WP2 and WP3 is a spread footing constructed on the bedrock. The elevations of footings can be estimated from Table 1-2. This alternative is not considered practical for the other locations because of the excavation depths exceeding 6 m. The factored bearing resistance at ULS for footings on unweathered bedrock is **8,000 kPa**. The bearing resistance at SLS does not apply because of the much higher pressures required to produce unacceptable deformations.

In order to evaluate the sliding resistance of spread footings on bedrock, the unfactored coefficients of friction for mass concrete on clean bedrock can be taken as 0.7. If the factored resistance against sliding failure is inadequate based on friction only, steel dowels will be required for footings on bedrock.

The ULS capacity of spread footings must be reduced for the effects of inclined loads. The reduction factors given in Table 2-4, below, can be used for footings on bedrock. Interpolation is possible. These factors must be applied to the ULS bearing resistance given previously.

<b>Table 2-4. ULS Reduction Factors for Inclined Loads on Spread Footings</b>	
<b>Ratio of Horizontal to Vertical Load</b>	<b>Footings on Rock</b>
0.1	0.86
0.2	0.76
0.3	0.66
0.4	0.58
The ULS reduction factors for inclined loads have been taken from Figure 6-8.4.2 of the OHBDC	

### 2.2.3.2 Spread Footings on Structural Fill (WP1 - South Abutment)

Spread footings can be designed for construction on structural fill at the south abutment. Structural fill should be constructed after removal of the overburden soils, where shallow, or it can be placed on the stripped native soils, as described below. For spread footing support, it is recommended that the structural fill consist of OPSS Granular A, placed in small lifts and adequately compacted (100%

standard Proctor). Alternatively, a relatively fine well graded rockfill, with a maximum size of 300 mm can be used. This finer graded rockfill should be placed in lifts limited to about 500 mm and adequately compacted with heavy vibratory rollers (minimum 6 passes, 10 tonnes).

At the south abutment, a spread footing abutment foundation, if considered, should be designed for construction in the approach fill, with a depth of about 2 m below the slope face. This would place the base of the footing at a distance of about 7 m to 8 m above original ground, at an elevation of about 318 m.

If the native granular soils are left in place, the structural fill supporting the foundation should have a thickness at least equal to the width of the footing. In addition, the structural fill should be constructed to occupy a zone, down and out from the footing edges at a slope of no steeper than 1H:2V, in order to accommodate the footing stresses.

The ULS resistance values given in Table 2-5, below, can be used for design. For 25 mm immediate settlement, the SLS resistance is greater than at ULS. Accordingly, the ULS resistance governs the design. Consolidation settlement is not considered an issue at this location since cohesive soils were not encountered in the investigation.

<b>Table 2-5. Spread Footing ULS and SLS Bearing Resistance</b>	
Factored Bearing Resistance at ULS on Unweathered Bedrock <sup>1</sup>	8000 kPa
Factored Bearing Resistance at ULS on Structural Fill <sup>2</sup>	1000 kPa
Bearing Resistance at SLS - Initial Elastic Settlement - Structural fill <sup>3</sup>	> 1000 kPa
Notes: 1. SLS resistance not applicable to footings on bedrock 2. Thickness of structural fill greater than footing width 3. Structural fill placed on native granular soils after removal of organics/topsoil, and bottom of footing about 7 m to 8 m above original ground (approx el. 318 m).	

For the determination of the sliding resistance of spread footings, the unfactored coefficient of friction for mass concrete on granular structural fill can be taken as 0.6. If the factored resistance against sliding failure is inadequate based on friction only, a soil key can be considered for footings on structural fill, making use of the passive soil resistance. Passive earth pressure coefficients are provided in Section 2.3.

The ULS resistance of spread footings must be reduced for the effects of inclined loads. For footings on granular structural fill, Figure 6-8.4.2 of the OHBDC may be used for the applicable footing depth to effective width ratio. These factors must be applied to the ULS resistance given previously in Table 2-5.

## 2.3 Backfill

Backfill for abutments or retaining walls should consist of free draining granular materials such as Granular A, Granular B, or rock fill. Computation of earth pressures shall be in accordance with Section 6.7.4 of the Ontario Highway Bridge Design Code. Unfactored properties for backfill materials are provided in the following Table 2-6.

<b>Table 2-6. Fill Types and Unfactored Geotechnical Properties</b>					
<b>Material</b>	<b>Friction Angle, <math>\phi'</math></b>	<b><math>\gamma</math> (kN/m<sup>3</sup>)</b>	<b><math>K_A</math></b>	<b><math>K_P</math></b>	<b><math>K_0</math></b>
Granular A	35 degrees	22	0.27	3.7	0.43
Granular B	30 degrees	21	0.33	3	0.5
Rock Fill	42 degrees	20	0.2	5	0.33
Note: Values given for $K_A$ and $K_P$ are for horizontal backfill, and will vary for other configurations. $K_A$ is the earth pressure coefficient corresponding to the active state. $K_P$ is the earth pressure coefficient corresponding to the passive state. $K_0$ is the earth pressure coefficient at rest.					

The mobilization of full active or passive resistance requires a measurable and perhaps significant wall movement or rotation, in general accordance with that indicated in Figure C6-7.1 of the OHBDC. Therefore, unless the structural element can tolerate these deflections, the at rest earth pressure should be used in design. Alternatively, partially mobilized active and passive earth pressure coefficients may be used, in accordance with the deflections given in the Figure.

The effects of compaction surcharge should be taken into account in the calculation of active and at rest earth pressures. In accordance with Section 6-7.4.3 of the OHBDC, the lateral pressure due to compaction should be taken as at least 16 kPa at the surface, and its magnitude should be assumed to diminish linearly with depth to zero at the depth where the active (or at rest) pressure is equal to

16 kPa. This pressure distribution should be added to the calculated active (or at rest) pressure. Notwithstanding, lighter compaction equipment and smaller lifts should be used adjacent to walls to prevent overstressing.

If rock fill is used as backfill behind abutments, the particle size should be limited to no greater than 300 mm and the backfill must be placed carefully in a manner that does not cause damage to the abutments or other structural components of the bridge.

Weep holes should be provided in the abutments and a filter wrapped, 150 mm diameter perforated PVC drainage pipe should be installed behind the abutments, at or slightly above the base.

## 2.4 Excavations and Dewatering

All work associated with design and construction relative to excavations shall be in accordance with Part III of Ont. Reg. 213/91 of the Occupational Health and Safety Act. Where the width of the base of the excavation is less than twice its depth, conformance with this regulation is required.

The existing organic zone (topsoil) with a depth ranging to about 300 mm will have to be removed from beneath the approach fill footprints. For pile caps at the pier locations, excavations of at least 2 m depth will likely be required, in order to provide the recommended frost cover. Since the ground surface is uneven at the pier locations, deeper excavations will be required to place the pile cap at a common elevation, unless the finished site grades are raised with fill or the caps are provided with insulation. Based on the borehole information, excavations would generally proceed through the loose to compact upper sand and into the stiff to firm silty clay at most locations.

At the locations of Piers WP2, WP3 and WP5, excavations should be relatively straight forward, since they will likely terminate above the prevailing groundwater level. The upper sand and silty clay is considered a Type 3 soil and excavations should be cut back to at least 1H:1V. If minor groundwater seepage occurs and loosens/softens the soil, flatter slopes will be required. Dewatering of the excavations should be possible by pumping from sumps within the excavations.

At the location of Pier WP4, on the north side of Trout Creek, the excavation will likely be carried out entirely within the loose to compact silty sand and would terminate close to or below the prevailing water level of Trout Creek. These soils would then be classified as Type 4 soils and excavations should be sloped back at 3H:1V. Pumping from sumps within the excavation should



suffice here as well, in order to maintain a safe and workable area, although more aggressive effort will likely be required. In order to stabilize the base of the excavation if it becomes loosened due to groundwater infiltration, a 300 mm layer of crushed clear stone may be required to improve working conditions.

If the native soils are to be removed and replaced with structural fill at the south abutment, or to place foundations on the bedrock at Piers WP2 and WP3, excavations of between about 1.7 m and 5.3 m will be required, based on the results of the investigation. The soils to be excavated will consist of sand, and silty sand and gravel, as well as silty clay in some locations. The water table would be encountered well above the 5.3 m depth. Accordingly, excavations in this area should be sloped back at 3H:1V, or flatter. Aggressive pumping from sumps will likely be required. Alternatively, sheeted and braced excavations could be considered, but this may prove problematic because of the variable bedrock surface.

Excavations carried out within granular structural fill in the approaches can likely be completed using a 1H:1V cut since it will be above the water table.

It is recommended that a non-standard special provision (NSSP) for dewatering be provided in the contract documents.

## **2.5 Bridge Approach Fills**

The construction of the bridge approaches will require embankment fills of up to about 14 m height at the south abutment and about 6 m height at the north abutment. The soils at the south abutment are predominantly granular, with a maximum thickness of about 3.5 m, based on the results of the investigation. At the north abutment, the principal soils consist of stiff to firm silty clay, to depths of up to about 14 m. The two principal design and construction considerations are embankment stability and consolidation settlement. These two issues are discussed in the following sections.

In all of the following discussions, it is assumed that all organic material (topsoil) is removed from beneath the embankments and the embankments are constructed on the native mineral soils. Fill heights should be measured from the top of the native mineral soil.

## 2.5.1 Embankment Stability

Highway embankments can be constructed using structural fill of various acceptable soil materials. Typically, however, in this part of the province they are constructed using rockfill as the principal component. A typical MTO section will consist of a minimum (steepest) slope of 1.25H to 1V to a maximum height of 6 m. For embankment sections greater than 6 m height, a 2 m wide horizontal bench is required before the next 6 m, 1.25H to 1V lift is constructed. A 14 m crest width has also been assumed, based on the drawings provided.

Slope stability analyses have been performed using SLOPE/W (ver. 3.x), based on Bishop's Method, using total stress parameters, for the cohesive soils. This analysis would apply to rapid construction (short term stability) and factors of safety can be expected to increase with time. The undrained shear strength profile shown on Figure A3 was used to provide the shear strength parameters for the silty clay soils. Table 2-7, below, lists the parameters used.

<b>Table 2-7. Geotechnical Parameters for Slope Stability Analyses</b>			
	$\gamma_{\text{total}}$ (kN/m <sup>3</sup> )	$c_u$ (kPa)	$\phi'$
Rockfill	20	0	42°
Sand	20	0	32°
Silty clay	19.5	variable (see Fig. A3)	0
Sand and Gravel	21.5	0	35°
Notes: Embankment crest width 14 m.			

Appendix D contains many of the SLOPE/W graphical printouts, for the various analyses performed and discussed below.

### 2.5.1.1 South Approach Stability

The south approach embankment will rest on essentially cohesionless soils. The results of the analyses performed on the embankment cross-sections, for an approach height of 14 m above original ground, indicates that the factor of safety against a foundation failure is about 1.7. Accordingly, the section as proposed is acceptable.



### 2.5.1.2 North Approach Stability

The north approach will rest on essentially cohesive soils. The results of the total stress analyses performed on the embankment cross-sections, for heights of 5 m to 8 m, resulted in safety factors ranging from about 2.9 to 2.0, which are considered more than adequate.

An analysis was also performed on the longitudinal design section (front slope) as shown on Figure A2b and Drawing No. 2. The calculated factor of safety for this slope is 1.40, which is considered adequate.

### 2.5.2 Consolidation Settlement of Approach Embankments (North Approach)

Long term consolidation settlement will result only at the north approach since cohesive soils are present as the principal foundation soils. The soils at the south approach are primarily cohesionless and consolidation settlement will not occur. Accordingly, the following discussion will apply only to the north approach.

#### 2.5.2.1 Magnitudes of Consolidation Settlement

For the north approach embankment, consolidation settlement calculations have been performed using the effective stress profiles shown on Figure A3 and compression ratios ( $C_c' = C_c/[1+e_0]$ ) ranging from 0.08 to 0.20. The values used were established from the consolidation test data, previous experience at the north and south interchanges of this project, as well as from geotechnical literature. Recompression indices ( $C_r' = C_r/[1+e_0]$ ) ranging from about 0.008 to 0.02 were used. The stress increases in the foundation soils due to the embankment loadings have been calculated based on the Boussinesq elastic solutions. Recompression and virgin consolidation has been calculated based on the assumed existing overburden pressure and preconsolidation stress profiles shown on Figure A3.

The results of the calculations for embankment centerline and crest settlement using rockfill to construct the embankments are provided in Table 2-8, below and are shown graphically in the top panel of Figure A5. The thickness of the compressible silty clay soil ranges up to about 14 m. Because of the unevenness of the original ground surface at the approaches, the height of fill may vary slightly. Accordingly, the analyses have been performed taking this into account by using a varying fill height.

**Table 2-8. Estimated Embankment Consolidation Settlement - North Approach**

Embankment Ht	Centerline Settlement (mm)	Crest Settlement (mm)
5 m	35	30
6 m	40	35
7 m	70	45
8 m	115	75

Notes: Embankment crest width 14 m. Values rounded to nearest 5 mm

The loadings imposed by the fill will approach and may exceed the indicated preconsolidation pressure, such that the silty clay soils will be normally consolidated following the consolidation process due to the fill. Examination of the top panel of Figure A5 indicates that the settlement is expected to increase significantly as the embankment height exceeds about 6 m, where the settlement curve steepens. This will result in greater settlements due to any future additional loadings, such as grade changes, for example. Accordingly, it may be prudent to overbuild the embankment by 1 or 2 m, allow consolidation to take place and subsequently remove the excess fill height (*i.e.* preloading). The preload should be left in place for about a year, unless monitoring indicates that it can be removed sooner. This will result in a slight overconsolidation of the foundation silty clay soils.

If consideration is to be given to the use of light weight fill, for preliminary design purposes, the total consolidation settlements can be reduced **approximately** by proportion of rockfill replaced by lightweight polystyrene (eg. 30% replacement of fill height with polystyrene will result in about 70% of the indicated settlement).

### 2.5.2.2 Time Rate of Consolidation Settlement

The time rate of consolidation settlement has been calculated for the various cases of embankment height for vertical drainage only. It is also assumed for the purposes of calculation, all embankments are constructed to full height in about a one month construction period. Coefficients of consolidation (virgin),  $C_v$ , of 8 m<sup>2</sup>/year, and (recompression),  $C_{vr}$ , of 40 m<sup>2</sup>/year, have been used in the analyses, based on the results of the consolidation tests and the geotechnical literature.

The bottom panel of Figure A5 shows the calculated consolidation rate for the various fill heights at the north approach. Primary consolidation should be complete anywhere between about 18 months to over 24 months, depending on the fill height.

### 2.5.3 Secondary Compression of Clays

Calculations have been performed to estimate the magnitudes of secondary compression of the foundation clays, following the completion of primary consolidation. For purposes of analysis and discussion, the primary consolidation is assumed to be essentially complete within one to two years from the start of construction of the embankments. The calculations are based on use of a coefficient of secondary compression,  $C_{\alpha\epsilon}$ , of 0.004, based on the results of the consolidation tests, previous experience, and the geotechnical literature.

The calculations indicate that the secondary settlement may be about 30 mm to 50 mm in the first 10 years.

### 2.5.4 Rockfill and Rockfill Settlement

Rockfill used in the embankments and approaches should consist of hard, durable, angular quarried rock. Rockfill should be tested for acid drainage potential and checked against the appropriate regulatory requirements for acceptability.

For the bridge abutments that are to be supported by driven steel H-piles, as discussed in a previous section, these will have to be installed after all the embankment fill has been placed. The driving of steel piles through rockfill will be difficult, if not impossible, unless care is taken to ensure that the rockfill meets a tight size range, generally smaller than 75 mm. For the majority of the embankments and approach fills, however, larger rockfill can be used.

The use of large rockfill, loosely placed or end dumped and compacted by conventional construction traffic only may result in long term settlements of up to about 2% of the fill height. Accordingly, a 10 m high rockfill may settle up to 200 mm following completion of construction. This is primarily due to the rockfill "adjusting" under highway service conditions.

In order to minimize the long term settlements associated with the rockfill itself, consideration can be given to specifying the use of a finer graded rockfill, with a maximum size of 400 mm. This finer graded rockfill should be placed in lifts limited to about 800 mm and compacted with heavy vibratory rollers. While the production of the finer graded rockfill and its placement, as described

above, carries with it a cost premium, the long term performance of the finished highway pavement will be improved.

It is recommended that an NSSP for rockfill material and placement requirements be included in the contract documents.

## **2.6 Instrumentation and Construction Monitoring**

Construction of embankments should be monitored during construction using a properly designed and effective system of instrumentation, consisting primarily of settlement points. Lateral slope movements should also be monitored both during and following construction of the embankments. This will provide indications of the rate of settlement, such that construction timing of the foundations can be modified, if required.

## **2.7 Closing Comments**

The recommendations made in this report are in accordance with our present understanding of the project and are provided solely for the use of Marshall Macklin Monaghan and its design team for the design of the bridge foundations and approach fills for the southbound lanes, five span bridge to be constructed over Trout Creek, as part of the Highway 11 Trout Creek By-Pass Project. We request that we be retained to review the design and our recommendations as the design proceeds, to ensure that the final design is in general agreement with our recommendations and that our recommendations have been interpreted as intended.

A subsurface investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those reported at the test locations, we require that we be notified immediately in order to allow reassessment of our recommendations. It may then be necessary to carry out additional field work and analyses.

Contractors bidding on or undertaking the works should, relative to the subsurface conditions, decide on their own investigations, if deemed necessary, as well as the their own interpretations of the factual results provided herein, so they may draw their own conclusions as to how the subsurface conditions may affect them.

The information presented in this report is based on a limited investigation designed to provide information to support an overall assessment of the current geotechnical conditions at the site of the proposed southbound lanes bridge over Trout Creek. The conclusions presented in this report reflect

site conditions existing at the time of the investigation. It is noted that the soil boundaries indicated on the logs are inferred from discontinuous sampling and observations during drilling. These boundaries are intended to reflect transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change.

This report has been prepared by Mr. D.N. Georgiou, P.Eng. and was reviewed by Messrs. S.E. Gonsalves, P.Eng., and E. Gonneau, P.Eng. The field investigation was performed by Messrs. I. Dumpis, C.E.T., R. Moore, C.E.T., and S. McAuliffe, and was supervised by Mr. E.A. Gonneau, P.Eng.

We trust this report is satisfactory for your purposes. Should you have any questions, please do not hesitate to contact us.

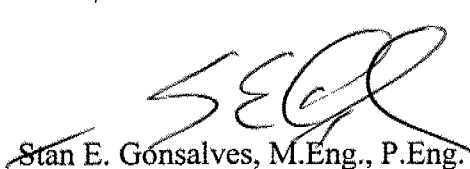
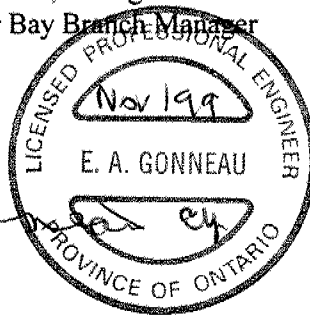
All the foregoing and attachments respectfully submitted,  
**Trow Consulting Engineers Ltd.**



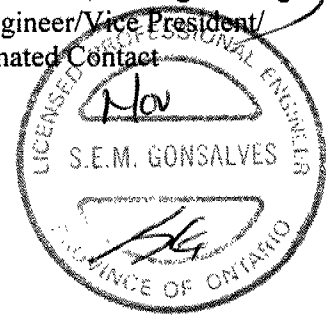
Demetri N. Georgiou, M.A.Sc., P.Eng.  
Principal Engineer/Thunder Bay Branch Manager



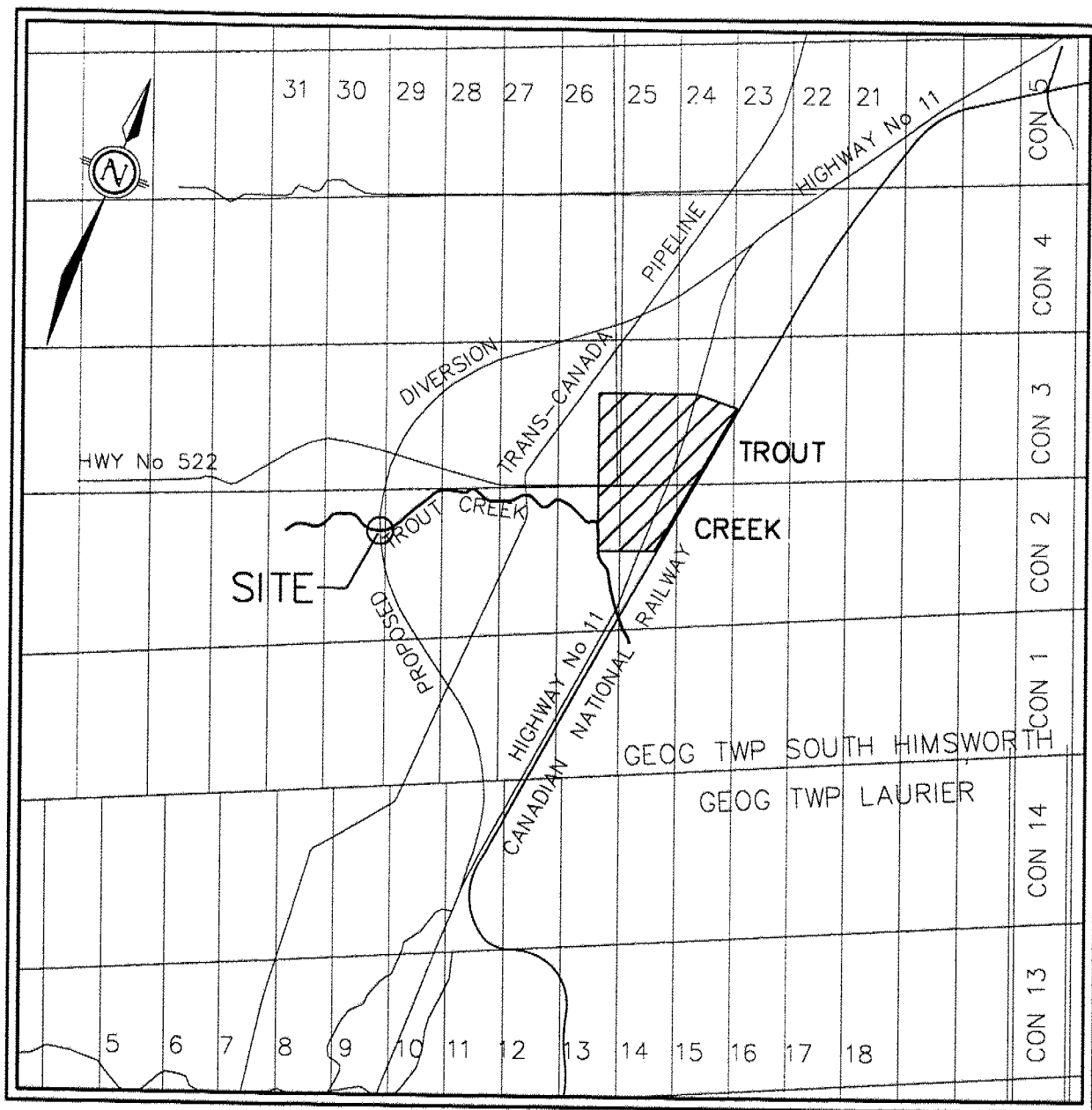
Eric A. Gonneau, P.Eng.  
Project Manager



Stan E. Gonsalves, M.Eng., P.Eng.  
Principal Engineer/Vice President/  
MTO Designated Contact



A



500 0 500 1000  
IN METRES



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Thunder Bay, Ontario

FIGURE  
A1

### KEY PLAN

Trout Creek By Pass  
Trout Creek Bridge-Southbound Lanes

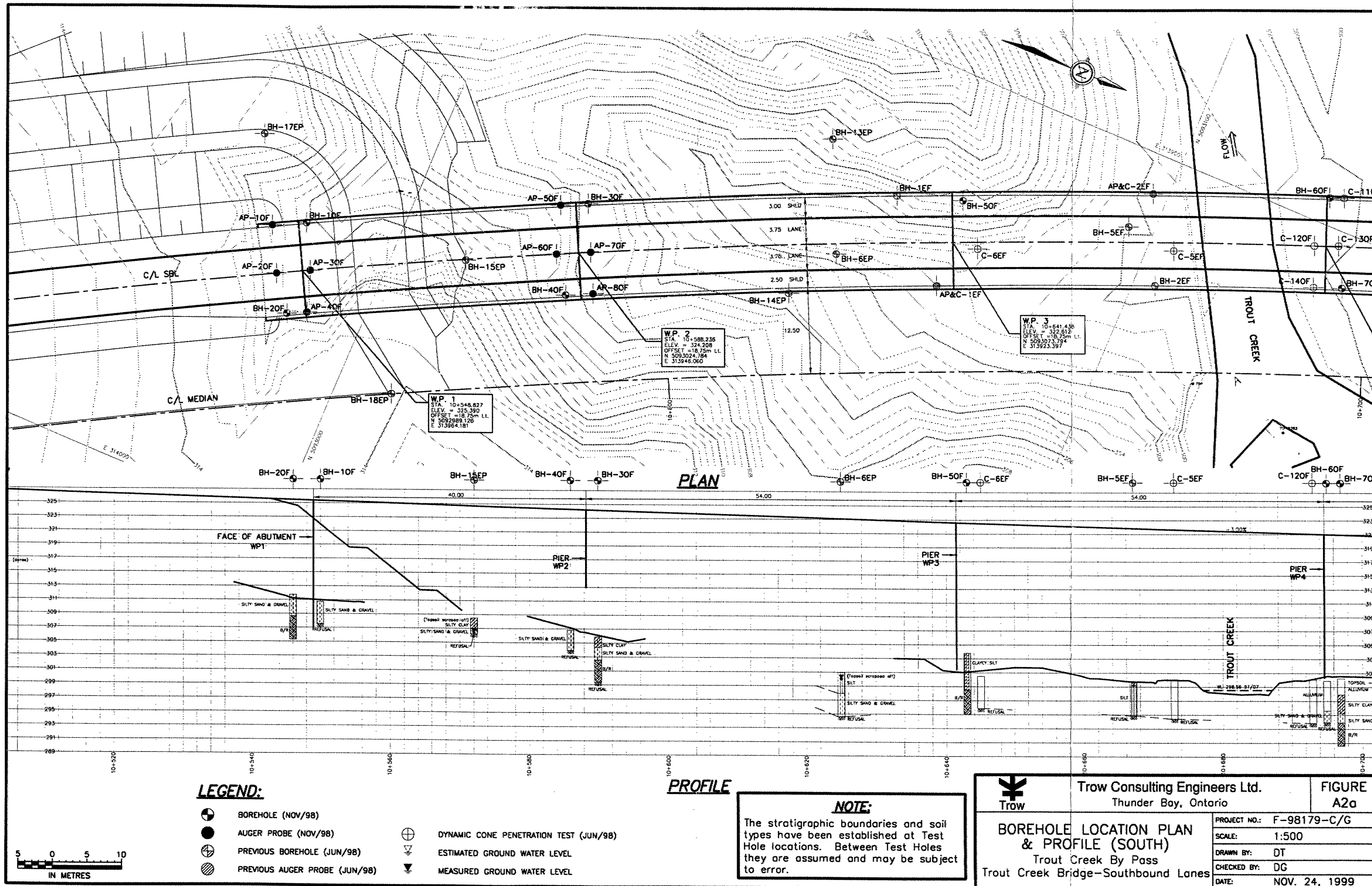
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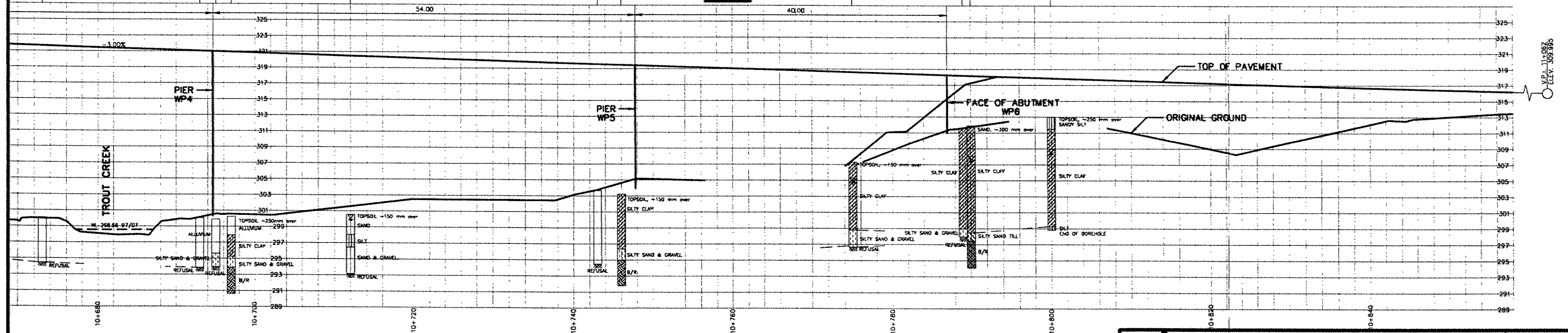
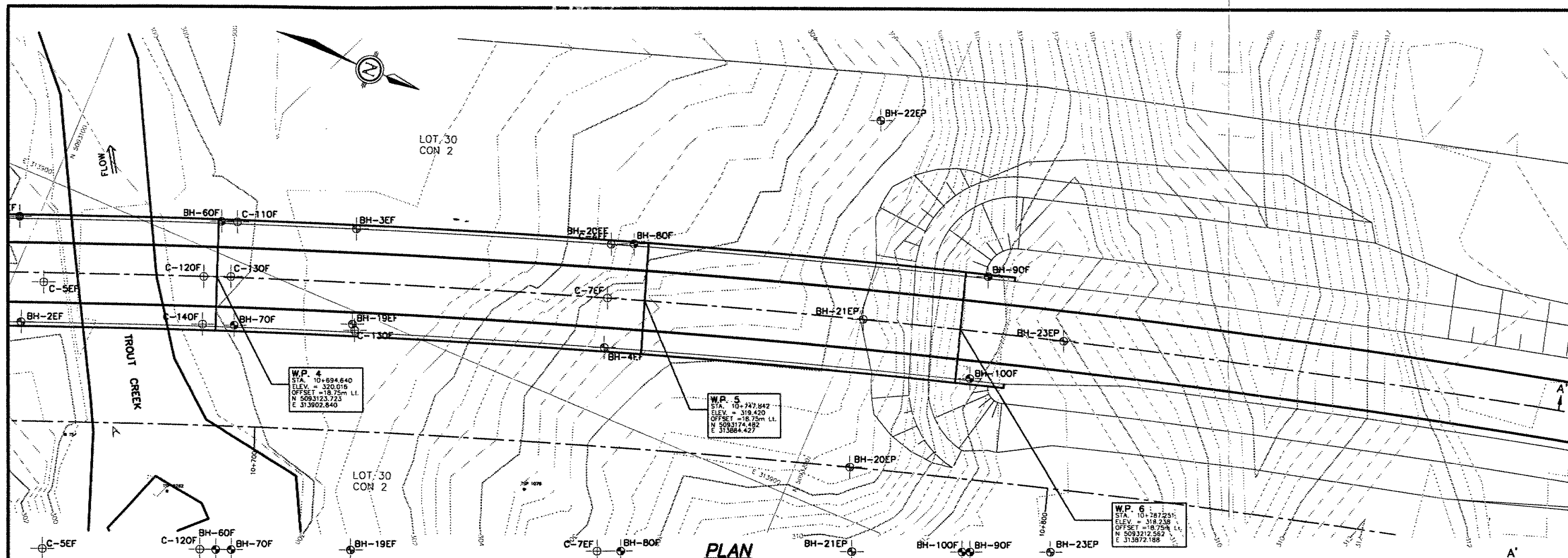
DRAWN BY: DT

CHECKED BY: DG

DATE: MARCH 12, 1999







# LEGEND:

- BOREHOLE (NOV/98)
- AUGER PROBE (NOV/98)
- PREVIOUS BOREHOLE (JUN/98)
- PREVIOUS AUGER PROBE (JUN/98)
- DYNAMIC CONE PENETRATION TEST (JUN/98)
- ESTIMATED GROUND WATER LEVEL
- MEASURED GROUND WATER LEVEL

# PROFILE

# NOTE:

The stratigraphic boundaries and soil types have been established at Test Hole locations. Between Test Holes they are assumed and may be subject to error.



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FIGURE  
A2b

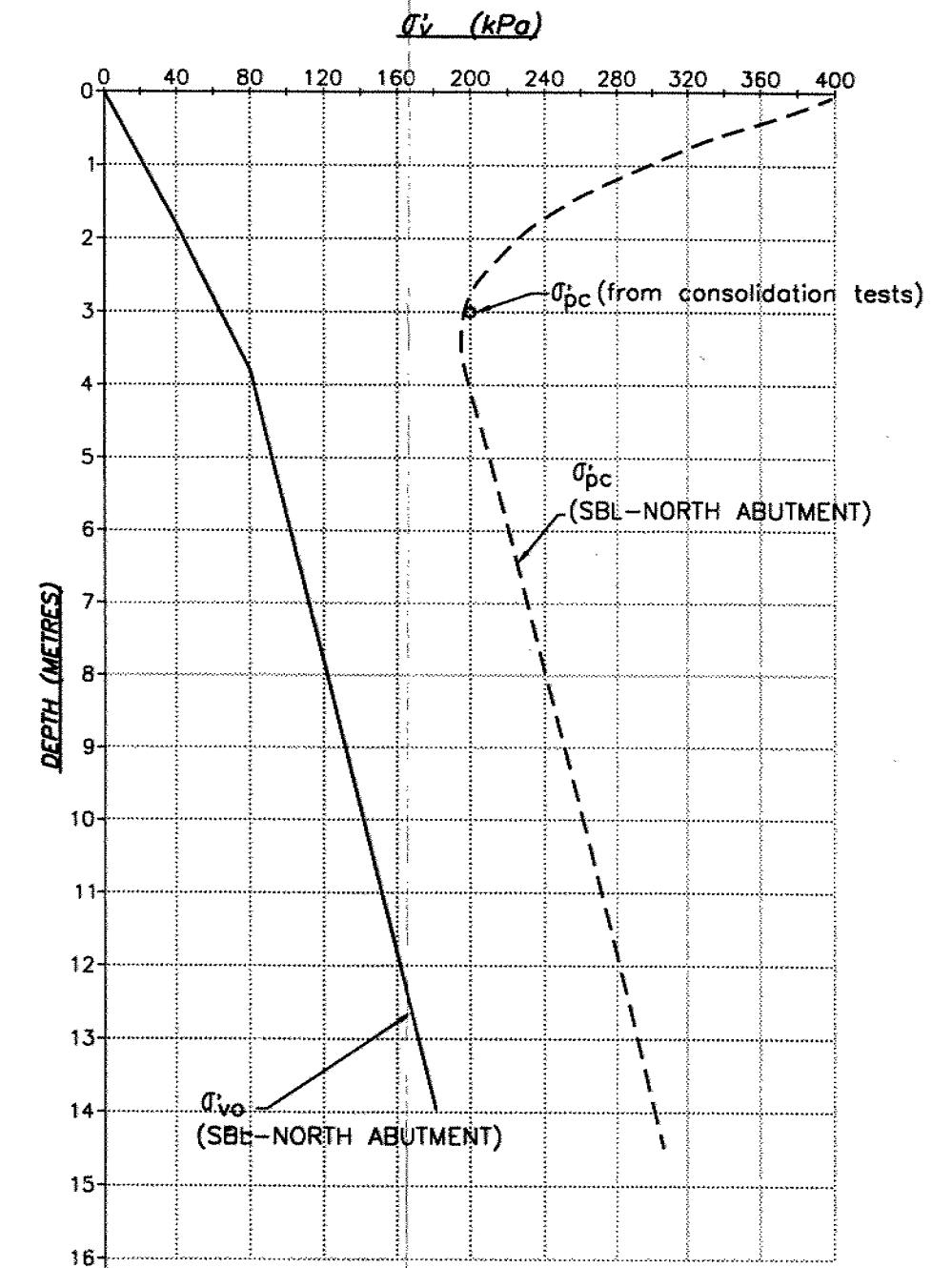
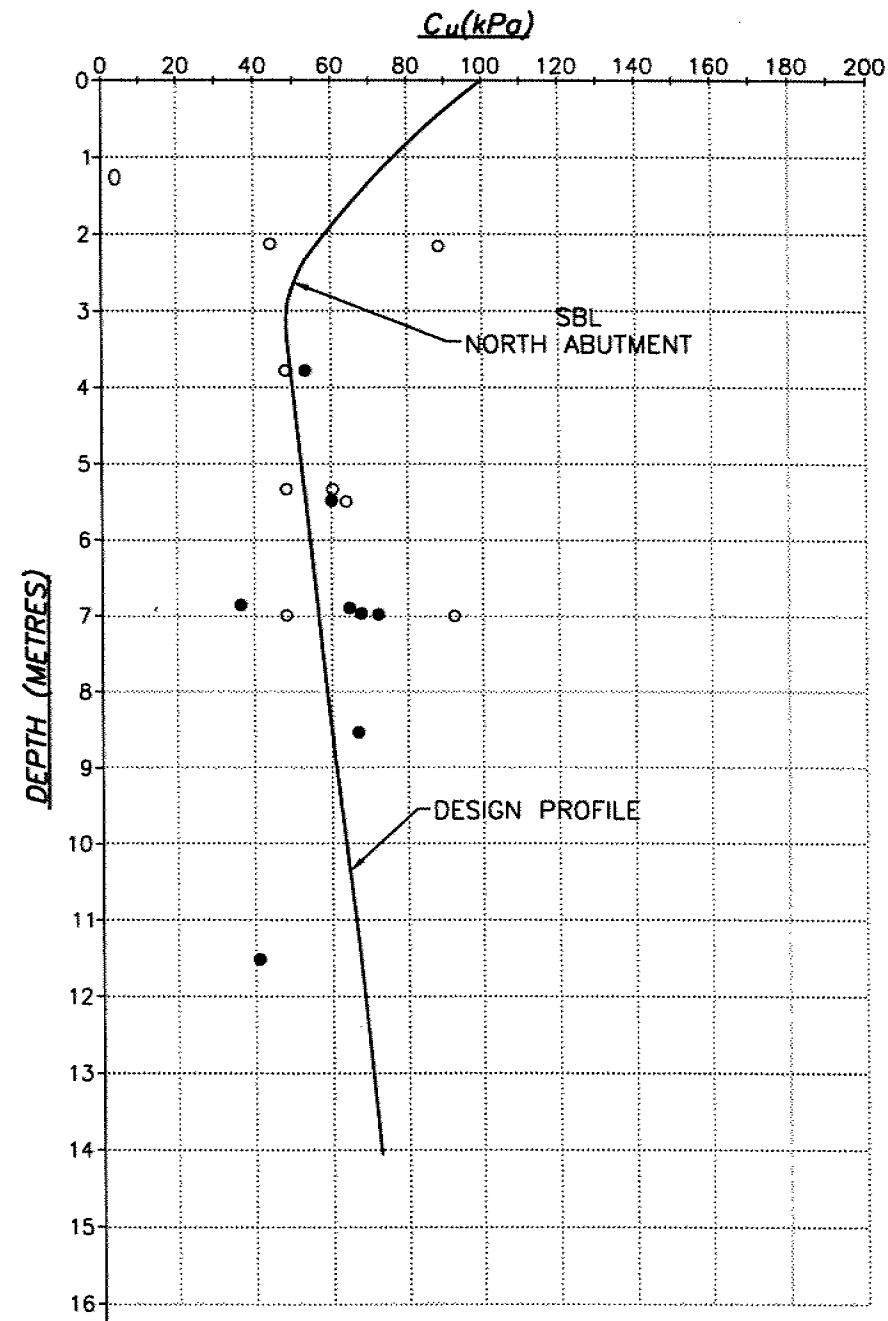
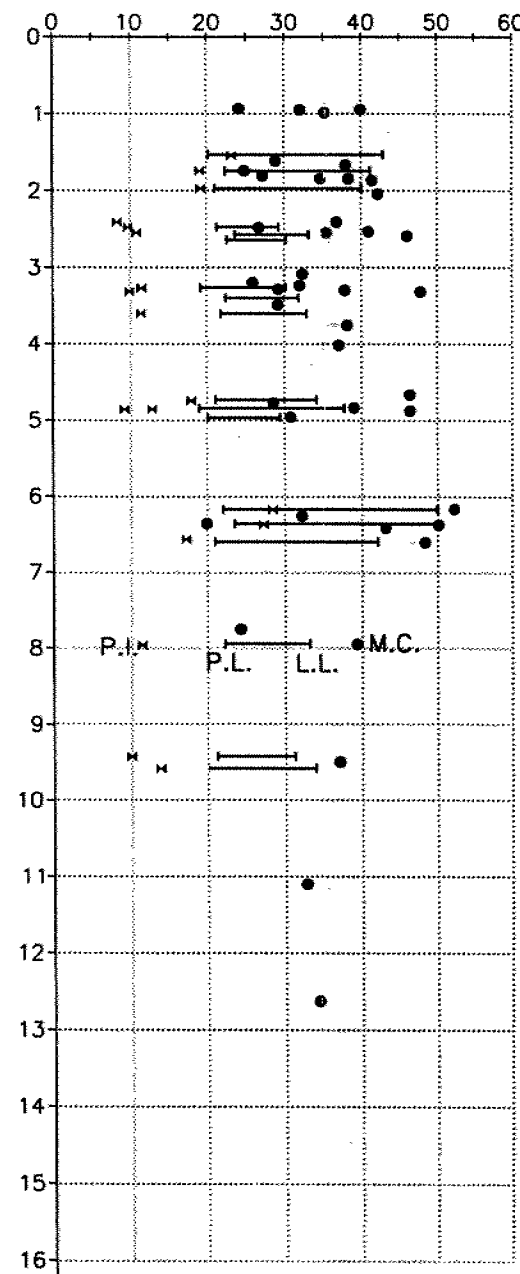
# BOREHOLE LOCATION PLAN & PROFILE (NORTH)

Trout Creek By Pass  
Trout Creek Bridge-Southbound Lanes

PROJECT NO.:	F-98179-C/G
SCALE:	1:500
DRAWN BY:	DT
CHECKED BY:	DG
DATE:	NOV. 24, 1999

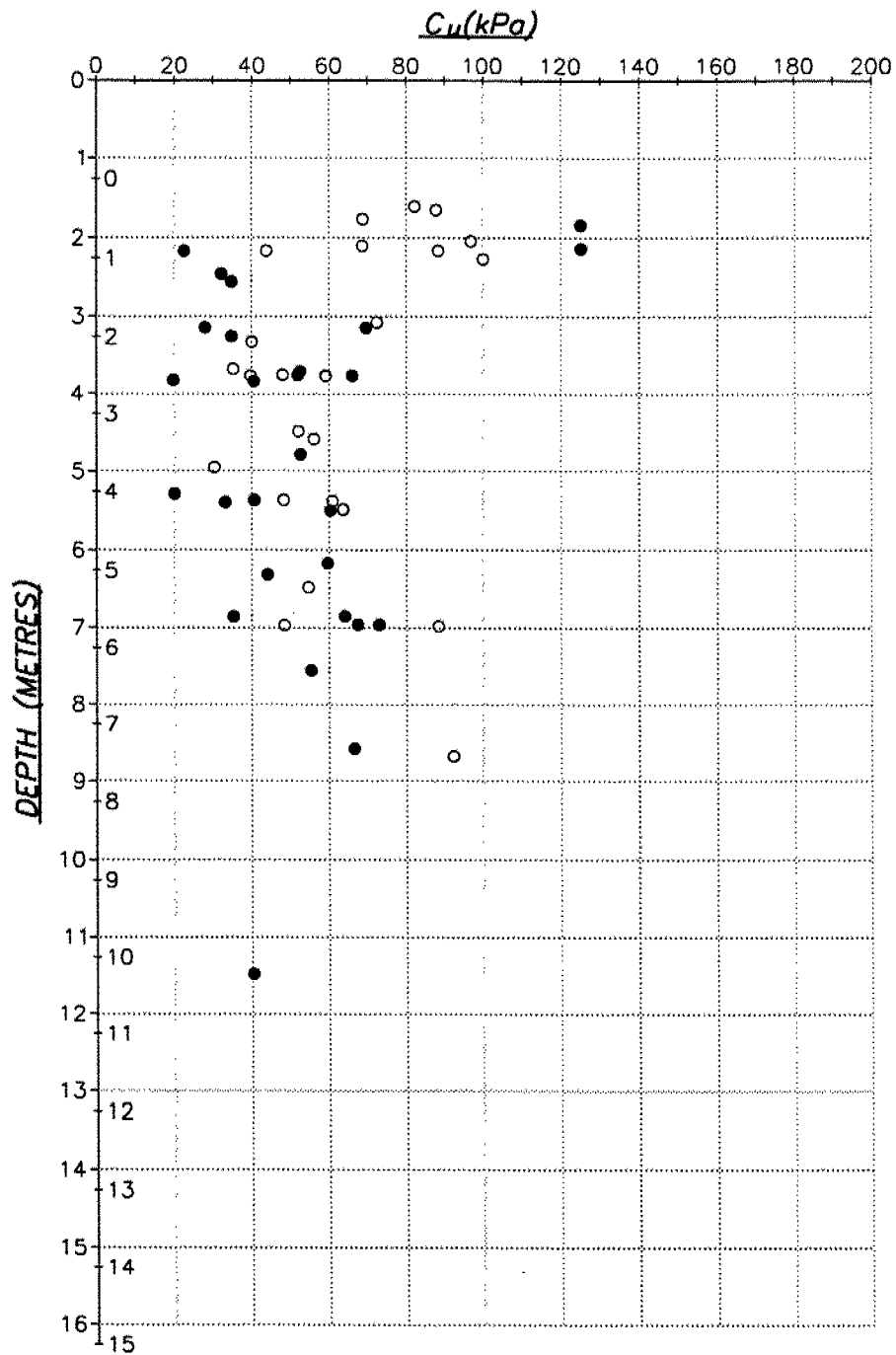


# MOISTURE CONTENT & ATTERBERG LIMITS



 <b>Trow Consulting Engineers Ltd.</b> Thunder Bay, Ontario	<b>FIGURE A3</b>
	PROJECT NO.: F-98179-C/G
	SCALE: AS SHOWN
	DRAWN BY: DT
	CHECKED BY: DG
DATE: MARCH 12, 1999	

UNDRAINED SHEAR STRENGTH,  
ATTERBERG LIMITS &  
EFFECTIVE STRESS PROFILES  
Trout Creek By Pass  
Trout Creek Bridge-Southbound Lanes



**LEGEND:**

- SHEAR STRENGTHS (NBL)
- SHEAR STRENGTHS (SBL)



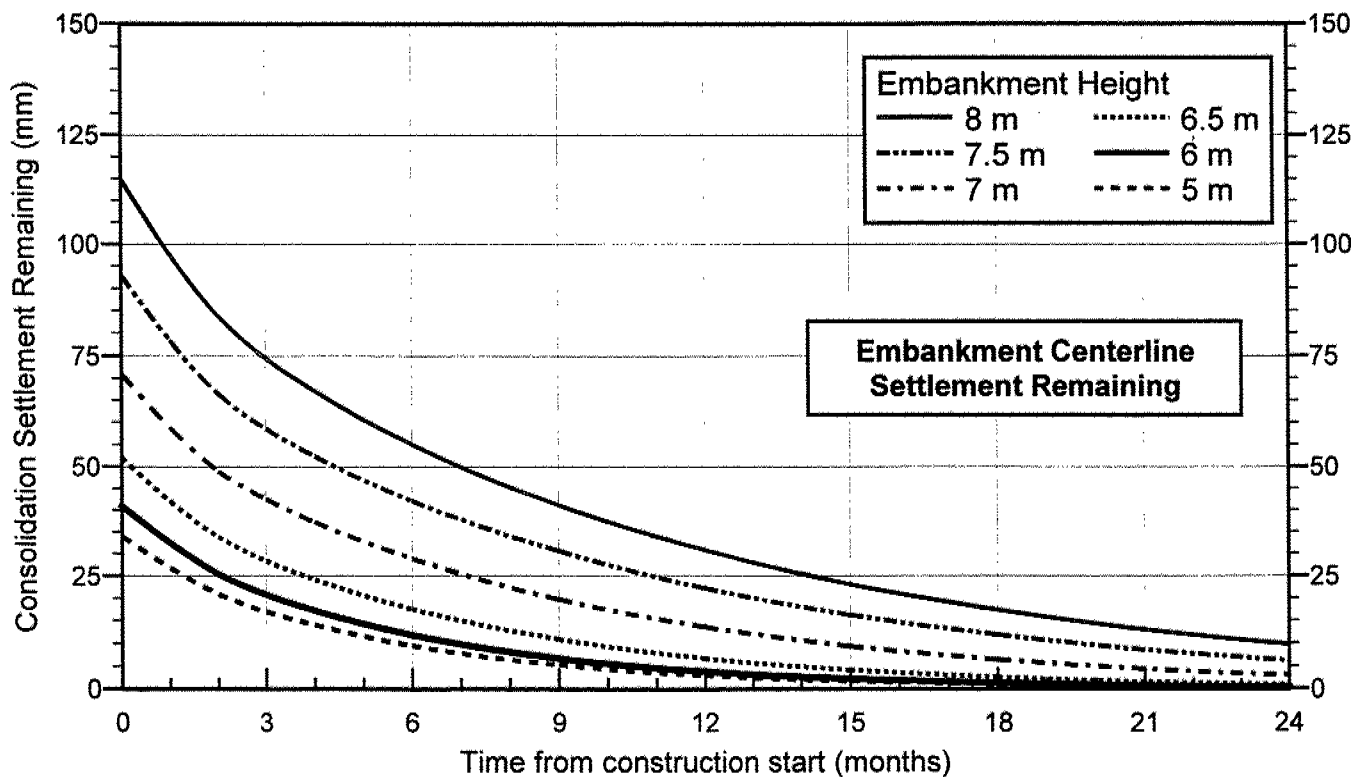
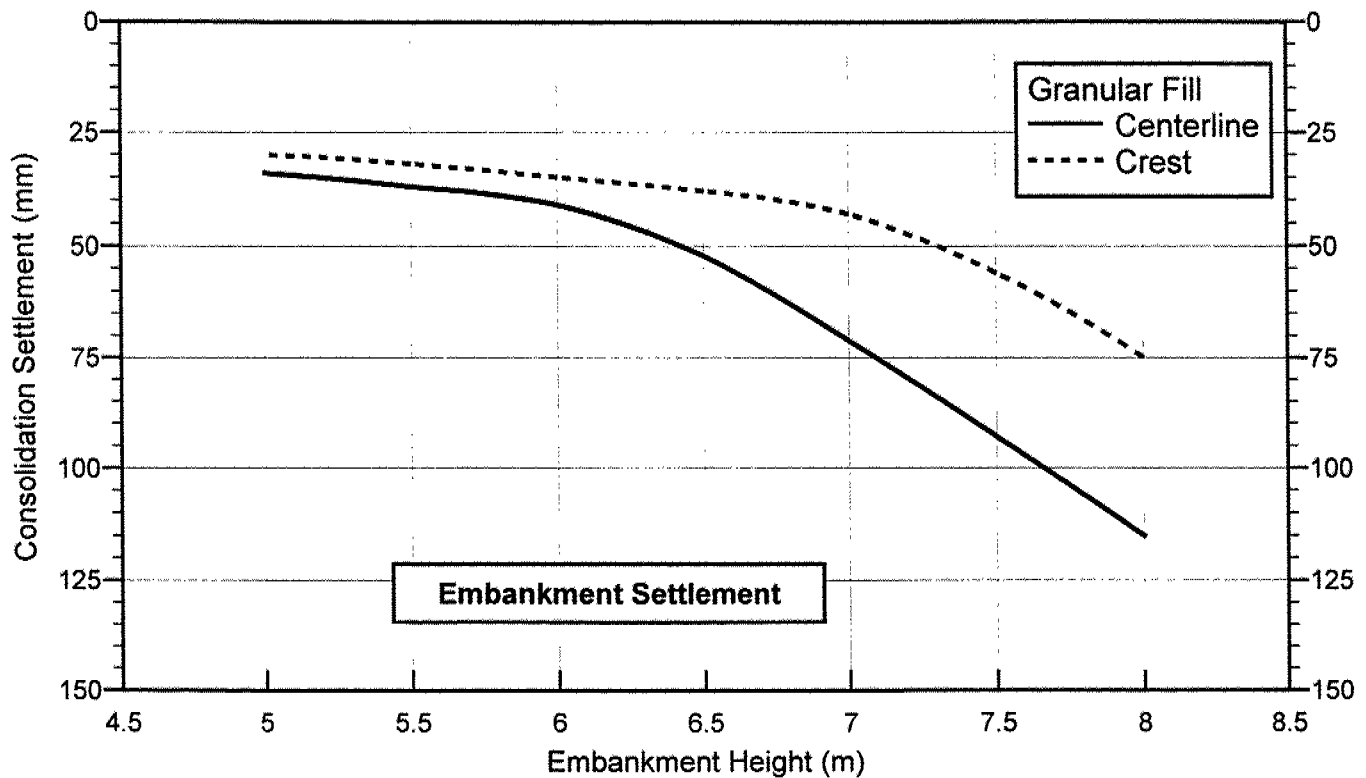
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Thunder Bay, Ontario

**FIGURE**  
**A4**

**UNDRAINED SHEAR STRENGTH  
ALL BOREHOLES**

Trout Creek By Pass  
Trout Creek Bridge—Southbound Lanes

PROJECT NO.:	F-98179-C/G
SCALE:	AS SHOWN
DRAWN BY:	DT
CHECKED BY:	DG
DATE:	MARCH 12, 1999



Trow Consulting Engineers Ltd.  
Thunder Bay, Ontario

### Estimated Consolidation Settlement NORTH APPROACH

F98179-C/G

Mar 3/99

Marshall Macklin Monaghan

Trout Creek Bridge - SOUTHBOUND LANES

Figure A5

B

# RECORD OF BOREHOLE BH-10F 1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station ~10+550, offset ~7 m left of centreline of Southbound Lane

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Hollow stem augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE November 12, 1998

CHECKED BY E.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION		
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	20	40	60	80	wp	w			wl	10
310.76	GROUND SURFACE																
0.00	SILTY SAND & GRAVEL, some cobbles, brown, wet. (compact)		1	SS	6												
			2	SS	24												
			3	SS	21												
307.56	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER		4	SS	60												
3.20	Notes: 1) This borehole forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 092 986.9 N, 313 957.4 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model. 4) Water level was at ~2.6 m & hole was open to ~2.8 m depth on completion. 5) This area was levelled with a bulldozer prior to advancing borehole.																



# RECORD OF BOREHOLE BH-20F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+546, offset ~6 m right of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / B.Q. core COMPILED BY M.D.  
 DATUM Geodetic DATE November 13, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20 40 60 80		wp — w — wl		WATER CONTENT (%)			
								SHEAR STRENGTH: Cu, KPa		UNCONFINED QUICK TRIAXIAL		FIELD VANE LAB SHEAR			10 20 30 40
311.74	GROUND SURFACE														
0.00	SILTY SAND & GRAVEL, some cobbles, brown, wet, till-like structure below ~1.5 m depth. (compact to dense)		1	SS	22		311								
			2	SS	50		310								
			3	WS			309								
308.62	BIOTITE HORNBLende GNEISS		4	SS	60		308							Rec. 100%R.Q.D. 90%	
3.12			5	BQ			307							Rec. 100%R.Q.D. 80%	
			6	BQ			306							Rec. 100%R.Q.D. 80%	
			7	BQ											
305.28	END OF BOREHOLE														
6.46	Notes: 1) This borehole forms part of the Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 092 989.4 N, 313 970.8 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model. 4) Water level was at ~1.1 m & hole was open to ~2.2 m depth on completion. 5) This area was levelled with a bulldozer prior to advancing borehole. 6) Drill encountered auger refusal on a probable boulder at ~1.6 m depth. Drill moved ~0.5 m from BH-20F and advanced borehole to completion.														



# RECORD OF BOREHOLE BH-30F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station +10+590, offset ~7 m left of centreline of Southbound Lane  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / B.Q. core  
 DATUM Geodetic DATE November 12, 1998  
 ORIGINATED BY I.D.  
 COMPILED BY M.D.  
 CHECKED BY E.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m	20	40	60	80	wp	w	wl	WATER CONTENT (%)		
305.90	GROUND SURFACE														
0.00	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (stiff to firm)		1	SS	13										3% 30% 67%
304.38	SILTY SAND & GRAVEL, some cobbles, till-like below ~3 m depth, brown, wet. (compact then dense below ~3 m depth)		2	SS	26										
1.52			3	SS	10										
302.55			4	SS	27										
3.35	BIOTITE HORNBLENDE GNEISS		5	BQ											Rec. 100% R.Q.D. 75%
299.35			6	BQ											Rec. 100% R.Q.D. 97%
6.55	END OF BOREHOLE														
Notes: 1) This borehole forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 023.3 N, 313 939.0 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model. 4) Water level was at ~2.0 m & hole was open to ~2.7 m depth on completion. 5) This area was levelled with a bulldozer prior to advancing borehole.															





# RECORD OF BOREHOLE BH-40F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+586, offset 76 m right of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 12, 1998 CHECKED BY E.G.


SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE		20	40	60	80	wp	w	wl	WATER CONTENT (%)		
306.84	GROUND SURFACE														
0.00	SILTY SAND & GRAVEL, some cobbles, brown, wet. (compact to dense)		1	SS	23										
			2	SS	60										
			3	SS	60										
303.94	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER														
2.90	Notes: 1) This borehole forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 025.4 N, 313 952.4 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model. 4) Borehole caved wet at 2.7 m depth on completion. 5) This area was levelled with a bulldozer prior to advancing borehole.														



# RECORD OF BOREHOLE BH-50F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10 - 643, offset 16 m left of centreline of Southbound Lane  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / B.Q. core  
 DATUM Geodetic DATE November 11, 1998  
 ORIGINATED BY I.D.  
 COMPILED BY M.D.  
 CHECKED BY E.G.

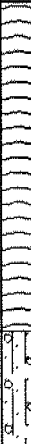
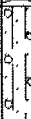
SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20 40 60 80				wp — w — wl					
								SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL FIELD VANE LAB SHEAR				WATER CONTENT (%)					
						20 40 60 80				10 20 30 40				GR SA (SI & CL)			
303.79	GROUND SURFACE																
0.00	CLAYEY SILT, with SILT bands, brown to grey, wet. (loose)		1	SS	8		303									0% 0% 100%	
			2	SS	6		302										
			3	SS	6		301										
			4	SS	7		300										
			5	SS	7		299								0% 0% 100%		
298.52	BIOTITE HORNBLENDE GNEISS		6	BQ			298									Rec. 100% R.Q.D. 60%	
5.27			7	BQ			297									Rec. 100% R.Q.D. 50%	
			8	BQ			296									Rec. 100% R.Q.D. 35%	
295.01	END OF BOREHOLE																
8.78	<b>Notes:</b> 1) This borehole forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 072.8 N, 313 917.3 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model. 4) Water level was at ~1.2 m & hole was open to ~4.6 m depth on completion.																



# RECORD OF BOREHOLE BH-60F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+695, offset 7 m left of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 18, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20      40      60      80				wp ——— w ——— wl				
								SHEAR STRENGTH: Cu, KPa ● UNCONFINED QUICK TRIAXIAL      X FIELD VANE LAB SHEAR				WATER CONTENT (%) 10      20      30      40				
299.89	GROUND SURFACE															
0.00	TOPSOIL, ~300 mm over ALLUVIUM, SILTY SAND, brown to grey, some organics, wood chunks.  (loose)		1	SS	0											
			2	SS	2											
			3	SS	2											
			4	SS	5											
295.62																
4.27	SILTY SAND & GRAVEL, some cobbles, grey. (compact)		5	SS	9											
293.95																
5.94	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER															
<p>Notes:</p> <p>1) This borehole forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation.</p> <p>2) Borehole located at U.T.M. coordinates 5 093 121.5 N, 313 896.2 E.</p> <p>3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model.</p> <p>4) Water level was at ~0.6 m &amp; hole was open to ~0.7 m depth on completion.</p>																



# RECORD OF BOREHOLE BH-70F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+697, offset 6 m right of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / BQ core COMPILED BY M.D.  
 DATUM Geodetic DATE November 18, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20    40    60    80		wp	w			wl
								SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL    FIELD VANE LAB SHEAR						
300.22	GROUND SURFACE													
0.00	TOPSOIL, ~250 mm over ALLUVIUM, SILTY SAND, brown to grey, some organics, wood chunks, (very loose to loose)		1	SS	4									
			2	SS	1									
297.92														
2.30	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		3	SS	4									
			4	SS	6									
295.22														
5.00	SILTY SAND & GRAVEL, brown to grey, wet, (compact)		5	SS	4									
293.90			6	SS	60									
6.32	BIOTITE HORNBLENDE GNEISS		7	BQ										
			8	BQ										
290.62														
9.60	END OF BOREHOLE													
Notes: 1) This borehole forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 128.1 N, 313 907.6 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model. 4) Water level was at ~0.5 m & hole was open to ~0.6 m depth on completion.														



# RECORD OF BOREHOLE BH-80F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+746, offset ~7 m left of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / BQ core COMPILED BY M.D.  
 DATUM Geodetic DATE November 19, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT				UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION		
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)							
								UNCONFINED QUICK TRIAXIAL		FIELD VANE LAB SHEAR									
303.23	GROUND SURFACE							20	40	60	80	wp	w	wl		GR	SA	(SI & CL)	
0.00	TOPSOIL, ~150 mm over SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		1	SS	8														
			2	SS	4														
			3	SS	3														
			4	SS	5														
			5	SS	6														
			6	SS	6														
296.37	SILTY SAND & GRAVEL, some cobbles. (loose to compact)		7	SS	8														
294.94	BIOTITE HORNBLende GNEISS		8	BQ															
			9	BQ															
291.80	END OF BOREHOLE																		
11.43	Notes: 1) This borehole forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 170.4 N, 313 878.4 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan from their terrain model. 4) Water level was at ~3.2 m & hole was open to ~4.2 m depth on completion. 5) This area was levelled with a bulldozer prior to advancing borehole.																		



# RECORD OF BOREHOLE BH-90F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+790, offset ~7 m left of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / BO core COMPILED BY M.D.  
 DATUM Geodetic DATE November 23, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			CONE PENETRATION TEST						
								20	40					
312.07	GROUND SURFACE													
0.00	SAND, brown, fine, ~300 mm over SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (stiff becoming firm)		1	SS	25									
			2	SS	19									
			3	SS	9									
			4	SS	6									
			5	SS	2									
			6	SS	3									
			7	SS	6									
			8	SS	7									
			9	SS	11									
			10	SS	8									
298.66			11	SS	7									
13.41	SILTY SAND & GRAVEL, some cobbles, grey, till-like structure, wet. (compact)		12	BQ										
297.65	BIOTITE HORNBLENDE GNEISS		13	BQ										
14.42	Notes: 1) This borehole forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 213.2 N, 313 864.7 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan from their terrain model.		14	BQ										
294.27	END OF BOREHOLE													
17.80	Notes: (cont'd) 4) Borehole caved wet at ~11.1 m depth on completion. 5) This area was levelled with a bulldozer prior to advancing borehole.													



# RECORD OF BOREHOLE BH-100F 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station ~10+789, offset ~6 m right of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 23, 1998 CHECKED BY E.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE		20	40	60	80					
311.62	GROUND SURFACE													
0.00	SAND, fine, brown, ~600 mm over SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (stiff becoming firm)		1	SS	7									
			2	SS	13									0% 0% 100%
			3	SS	11									
			4	SS	6									
			5	SS	2									
			6	SS	2									0% 0% 100%
			7	SS	4									
			8	SS	6									
			9	SS	8									
			10	SS	8									
298.97	SILTY SAND & GRAVEL, grey, till-like structure, wet. (compact)													
12.65														
297.96	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER													
13.66	Notes: 1) This borehole forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 216.0 N, 313 855.4 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan from their terrain model. 4) Borehole caved wet at ~11.7 m depth on completion. 5) This area was levelled with a bulldozer prior to advancing borehole.													



# RECORD OF BOREHOLE BH-1EF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station +10+634, offset 7 m left of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 10, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80				wp — w — wl					
								SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL X FIELD VANE LAB SHEAR 20 40 60 80				WATER CONTENT (%) 10 20 30 40					
303.95	GROUND SURFACE																
0.00	(Topsoil scraped off) SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (stiff becoming firm)		1	SS	10		303										
			2	SS	8		302			S <sub>u</sub> = 9							
			3	SS	4		301								0% 0% 100%		
			4	SS	7		300			S <sub>u</sub> = 7	X						
			5	SS	5		299			S <sub>u</sub> = 5	X						
			6	SS	7		298			S <sub>u</sub> = 10							
296.13			7	SS	40		297										
7.82	BIOTITE HORNBLENDE GNEISS, good to excellent rock quality, slightly weathered to unweathered.		8	BQ			296			150 mm					Rec. 100% RQD 81%		
			9	BQ			295										
293.13							294								Rec. 100% RQD 92%		
10.82	END OF BOREHOLE																
Notes: 1) This borehole forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 064.0 N, 313 920.0 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Water level was at ~1.5 m depth on completion.																	





# RECORD OF BOREHOLE BH-2EF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+670, offset ~6 m right of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 10, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80								
								SHEAR STRENGTH: Cu, KPa								
								UNCONFINED QUICK TRIAXIAL      FIELD VANE LAB SHEAR								
						20 40 60 80				WATER CONTENT (%)				kN/m <sup>3</sup>	GR   SA   (SI & CL)	
										10 20 30 40						
										wp      w      wl						
299.90	GROUND SURFACE															
0.00	(Topsoil scraped off) SAND, with gravel inclusions & pieces of wood, grey/brown, wet. (very loose)		1	SS	3											
298.40	ALLUVIUM															
1.50	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		2	SS	5											
			3	SS	3											
			4	SS	3											
294.72			5	SS	9											
5.18	BIOTITE HORNBLENDE GNEISS, good rock quality, unweathered.		6	BQ												
			7	BQ												
291.58																
8.32	END OF BOREHOLE															
Notes: 1) This borehole forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 102.8 N, 313 917.6 N. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Water level was at ~1.4 m & hole was open to ~1.6 m depth on completion.																

0% 10% 90%  
 Rec. 100% RQD 83%  
 Rec. 100% RQD 86%



# RECORD OF BOREHOLE BH-3EF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+712, offset 7 m left of centreline of Southbound Lane ORIGINATED BY J.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 25, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION		
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20	40	60	80	wp	w	wl				
SHEAR STRENGTH: Cu, KPa												WATER CONTENT (%)					
UNCONFINED QUICK TRIAXIAL												FIELD VANE LAB SHEAR					
20 40 60 80												10 20 30 40					
300.36	GROUND SURFACE																
0.00	TOPSOIL, ~200 mm over SAND, some gravel sizes & root inclusions, traces of clay & silt, brown, wet. (compact)		1	SS	14										0% 66% 34%		
298.36			2	SS	18												
2.00	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		3	SS	5												
			4	SS	6												
295.36			5	SS	8										0% 16% 84%		
5.00	SAND & GRAVEL, some cobbles, brown, wet. (loose then dense at base)		6	SS	5										14% 69% 17%		
			7	SS	35												
292.23	BIOTITE HORNBLENDE GNEISS, excellent rock quality, unweathered.		8	BQ											Rec. 100% RQD 95%		
			9	BQ											Rec. 100% RQD 95%		
289.29	END OF BOREHOLE																
11.07	Notes: 1) This borehole forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 137.8 N, 313 890.0 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Drill met auger refusal at ~5.5 m depth. Drill moved ~0.6 m south of BH-3EF & carried out borehole to completion. 5) Water level was at ~0.9 m & hole was open to ~6.4 m depth on completion.																



# RECORD OF BOREHOLE BH-4EF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+743, offset ~6 m right of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 30, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			SHEAR STRENGTH: Cu, KPa		WATER CONTENT (%)				
							UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR	wp	w	wl		
							20   40   60   80	20   40   60   80	10   20   30   40			GR   SA   (SI & CL)	
305.36	GROUND SURFACE												
0.00	(Topsoil scraped off) CLAYEY SILT, some layering, brown to grey, moist. (loose)		1	SS	7			X					
			2	SS	5								
			3	SS	4			X					
			4	SS	7								
301.36	4.00 SILT, occasional thin clay seams, grey, wet. (loose)		5	SS	7								
			6	SS	7								
298.36	7.00 SAND & GRAVEL, some cobbles & possible boulders, grey, wet. (compact)		7	SS	9								
			8	SS	12								
295.45	9.91 BIOTITE HORNBLENDE GNEISS, fair to excellent rock quality, slightly weathered to unweathered.		9	BQ									
			10	BQ									
292.38	12.98 END OF BOREHOLE												
Notes: 1) This borehole forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 171.8 N, 313 891.7 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Borehole caved wet at ~8.1 m depth on completion.													



# RECORD OF BOREHOLE BH-5EF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+667, offset 2 m left of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 11, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		UNIT WEIGHT  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20 40 60 80		wp — w — wl		WATER CONTENT (%)			
								SHEAR STRENGTH: Cu, KPa		WATER CONTENT (%)					
							UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR							
							20 40 60 80	20 40 60 80	10 20 30 40		10 20 30 40		GR SA (SI & CL)		
299.76	GROUND SURFACE														
0.00	TOPSOIL, ~150 mm over SILT, some sand, grey, traces of clay, wet below ~600 mm depth. (loose)		1	SS	3										
			2	SS	5										
			3	SS	5										
			4	SS	5										
294.88	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER		5	SS	40										
4.88	Notes: 1) This borehole forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 096.9 N, 313 911.4 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Water level was at ~3.8 m & hole was open to ~4.3 m depth on completion.														



# RECORD OF BOREHOLE BH-6EP 1 OF 1

## METRIC

W.P. 774-93-00      LOCATION Station ~10+625, offset ~2 m right of centreline of Southbound Lane      ORIGINATED BY I.D.  
 DIST 54      HWY 11      BOREHOLE TYPE Hollow stem augers / CME-55      COMPILED BY M.D.  
 DATUM Geodetic      DATE June 15, 1998      CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)		CONE PENETRATION TEST		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR					
300.97	GROUND SURFACE													
0.00	(Topsoil scraped off) SILT, occasional thin clay seams, grey, wet. (very loose to loose)		1	SS	2									
			2	SS	8									
			3	SS	9									
297.97	SILTY SAND & GRAVEL, some cobble sizes, brown, wet. (dense to very dense)		4	SS	36									
3.00			5	SS	105									
			6	SS	60									
294.63	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER													
6.34	Notes: 1) This borehole forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 053.4 N, 313 932.0 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Water level was at ~0.9 m & hole was open to ~2.4 m depth on completion.													



# RECORD OF BOREHOLE BH-12EP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+499, on centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 18, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m	20	40	60	80	wp	w	wl	WATER CONTENT (%)		
316.75	GROUND SURFACE														
0.00	TOPSOIL, 150 mm over														
316.08	SAND & GRAVEL (dense)														
0.67	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER														
Notes: 1) This borehole forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 032 944.9 N, 313 988.7 E 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Borehole was dry & open to full depth on completion. 5) Drill made 4 other attempts to drill adjacent BH-12EF & met auger refusals from 0.4 m to 0.7 m depths.															



# RECORD OF BOREHOLE BH-13EP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+625, offset ~15 m left of centreline of Southbound Lane  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55  
 DATUM Geodetic DATE June 19, 1998

ORIGINATED BY I.D.  
 COMPILED BY M.D.  
 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT		UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20   40   60   80		wp   —   w   —   wl			
								SHEAR STRENGTH: Cu, KPa ● UNCONFINED QUICK TRIAXIAL   × FIELD VANE LAB SHEAR		WATER CONTENT (%) 10   20   30   40			
307.33	GROUND SURFACE												
0.00	(Topsoil scraped off) <b>SILTY CLAY</b> , grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)												
			1	SS	4								
			2	SS	5								
			3	SS	6								
			4	SS	6								
			5	TW									
			6	SS	7								
300.33													
7.00	<b>SILT</b> , traces of fine sand, some gravel sizes at base, wet. (compact)												
299.25			7	SS	9								
8.08	<b>SAND &amp; GRAVEL</b> (dense)												
298.34													
8.99	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER												
	Notes: 1) This borehole forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 052.4 N, 313 916.5 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Borehole caved dry at ~4.8 m depth on completion.												



# RECORD OF BOREHOLE BH-14EP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station +10+618, offset 7 m right of centreline of Southbound Lane  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55  
 DATUM Geodetic DATE June 19, 1998

ORIGINATED BY I.D.  
 COMPILED BY M.D.  
 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE		20	40	60	80	wp	w	wl	WATER CONTENT (%)		
303.60	GROUND SURFACE														
0.00	(Topsoil scraped off) SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		1	SS	4										
			2	SS	5										
301.10			3	SS	20										
2.50	SILTY SAND & GRAVEL, some cobbles, grey, wet. (compact)		4	SS	24										
300.25															
3.35	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER														
Notes: 1) This borehole forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 055.0 N, 313 939.5 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Borehole caved dry at 2.7 m depth on completion.															





# RECORD OF BOREHOLE BH-15EP 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station +10+572, on centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 19, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT				UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m								
309.49	GROUND SURFACE														
0.00	(Topsoil scraped off) <b>SILTY CLAY</b> , grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		1	SS	8										
307.69															
1.80	<b>SILTY SAND &amp; GRAVEL</b> (compact to dense)		2	SS	26										
307.23															
2.26	<b>END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER</b>														
<b>Notes:</b> 1) This borehole forms part of Trout Creek Bridge Southbound Lane Approach Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 010.0 N, 313 953.4 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Borehole caved dry at ~2.2 m depth on completion.															



# RECORD OF BOREHOLE BH-17EP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+545, offset 20 m left of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 19, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80				wp — w — wl				
								SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)				
								●	○	×						
								○	●	×						



## 1 OF 3

METRIC

W.P. 774-93-00

**LOCATION** Station ~10+560, on centreline of Median

ORIGINATED BY I.D.

DIST 54 HWY 11

**BOREHOLE TYPE** Hollow stem augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE June 19, 1998

CHECKED BY I.G.

[illegible]

# RECORD OF BOREHOLE BH-19EF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+712, offset ~5 m right of centreline of Southbound Lane ORIGINATED BY J.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 24, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80				wp — w — wl					
								SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)					
								UNCONFINED QUICK TRIAXIAL X FIELD VANE LAB SHEAR				10 20 30 40					
300.52	GROUND SURFACE																
0.00	TOPSOIL, ~150 mm over SAND, trace of silt, brown, wet below ~600 mm depth. (compact)						300										
298.02			1	SS	11		299										
2.50	SILT, some thin clay seams, grey, wet. (loose)						298										
296.42			2	SS	9		297										
4.10	SAND & GRAVEL, some cobbles & possible boulders. (compact to dense)						296										
							295										
			3	SS	29		294										
			4	SS	31												
293.17	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER																
7.35	Notes: 1) This borehole forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 141.9 N, 313 901.3 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Water level was at ~2.0 m & hole was open to ~2.1 m depth on completion.																



# RECORD OF BOREHOLE BH-20EF 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station 10+742, offset 7 m left of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 26, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)		CONE PENETRATION TEST		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION		
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			SHEAR STRENGTH: Cu, KPa		WATER CONTENT (%)		wp	w	wl							
								UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR												
303.20	GROUND SURFACE																				
0.00	TOPSOIL, ~150 mm over SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		1	SS	9																
			2	SS	8																
			3	SS	6																
			4	SS	5																
			5	TW																	
			6	SS	5																
296.70	SAND & GRAVEL, some cobbles & possible boulders, grey, wet. (loose)		7	SS	5																
6.50			8	SS	7																
293.90	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER																				
9.30	Notes: 1) This borehole forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 166.6 N, 313 879.7 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Water level was at ~4.9 m & hole was open to ~5.94 m depth on completion.																				



# RECORD OF BOREHOLE BH-21EP 1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station ~10+775, on centreline of Southbound Lane

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Hollow stem augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE July 1, 1998

CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		UNIT WEIGHT kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION		
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80		wp — w — wl		10 20 30 40					
								SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL    FIELD VANE LAB SHEAR		WATER CONTENT (%)							
307.28	GROUND SURFACE																
0.00	TOPSOIL, ~150 mm over SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		1	SS	3		307										
			2	SS	5		306										
			3	AS			305		S = 9								
			4	TW			304							18.60			
			5	SS	5		303										
			6	SS	5		302		S = 8								
			7	SS	10		301										
298.78							300										
8.50	SILTY SAND & GRAVEL, grey, wet. (compact)		8	SS	17		299										
							298										
296.67							297										
10.61	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER																
<p>Notes:</p> <p>1) This borehole forms part of Trout Creek Bridge Southbound Lane Approach Foundation Investigation.</p> <p>2) Borehole located at U.T.M. coordinates 5 093 200.7 N, 313 875.9 E.</p> <p>3) Borehole elevation obtained from Marshall Macklin Monaghan survey crew.</p> <p>4) Borehole caved wet at ~8.8 m depth on completion.</p>																	



# RECORD OF BOREHOLE BH-22EP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+775, offset ~25 m left of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE July 1, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	20	40	60	80		
305.85	GROUND SURFACE												
0.00	TOPSOIL, ~150 mm over SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		1	SS	7								
			2	SS	5								
			3	SS	3								
			4	SS	3								
			5	TW									
299.85			6	SS	28								
6.00	SILTY SAND & GRAVEL, till-like structure, some cobbles, grey, wet. (compact to dense)		7	SS	15								
297.65													
8.20	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER												

Notes:  
 1) This borehole forms part of Trout Creek Bridge Southbound Lane Approach Foundation Investigation.  
 2) Borehole located at U.T.M. coordinates 5 093 193.2 N, 313 852.0 E.  
 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew.  
 4) Borehole caved wet at ~5.3 m depth on completion.



# RECORD OF BOREHOLE BH-23EP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+800, on centreline of Southbound Lane ORIGINATED BY S.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE October 24, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT wp	NATURAL MOISTURE CONTENT w	LIQUID LIMIT wl	WATER CONTENT (%)	UNIT WEIGHT kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)		
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60							80	20
313.04	GROUND SURFACE																	
0.00	TOPSOIL, ~250 mm over SANDY SILT, brown, moist. (compact)																	
311.54	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (stiff becoming firm)		1	SS	18													
1.50			2	SS	9													
			3	SS	2													
			4	TW														
			5	SS	3													
			6	SS	9													
			7	SS	9													
			8	SS	12													
299.32	SILT, with SAND seams, grey brown, moist. (compact)		9	SS	16													
13.72	END OF BOREHOLE																	
298.87																		
14.17																		

Notes:  
 1) This borehole forms part of Trout Creek Bridge Southbound Lane Approach Foundation Investigation.  
 2) Borehole located at U.T.M. coordinates 5 093 225.0 N, 313 868.5 E.  
 3) Borehole elevation obtained from Dearden and Stanton Ltd., O.L.S.  
 4) Borehole caved wet at ~10.3 m depth on completion.





# RECORD OF BOREHOLE BH-24EP 1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station ~10+875, on centreline of Southbound Lane

ORIGINATED BY S.M.

DIST 54 HWY 11

BOREHOLE TYPE Hollow stem augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE October 24, 1998

CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION							
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)										
								UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR	wp	w	wl										
						20	40	60	80	20	40	60	80	10	20	30	40	kN/m³	GR	SA	(SI & CL)	
313.54 0.00	GROUND SURFACE																					
	TOPSOIL, ~250 mm over SILTY SAND, brown, moist. (compact)		1	SS	14															1%	95%	4%
311.10 2.44	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		2	SS	5																	
			3	SS	2															0%	0%	100%
			4	SS	1																	
			5	TW																0%	0%	100%
			6	SS	6																	
			7	SS	10																	
301.35 12.19 300.89 12.65	SILT, trace of SAND, grey brown, moist. (compact) END OF BOREHOLE		8	SS	15																	
<p>Notes:</p> <p>1) This borehole forms part of Trout Creek Bridge Southbound Lane Approach Foundation Investigation.</p> <p>2) Borehole located at U.T.M. coordinates 5 093 298.6 N, 313 849.3 E.</p> <p>3) Borehole elevation obtained from Dearden and Stanton Ltd., O.L.S. terrain model.</p> <p>4) Borehole caved wet at ~10.3 m depth on completion.</p>																						



# RECORD OF BOREHOLE AP-10F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+545, offset 7 m left of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 12, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION				
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20	40	60	80	wp	w	wl	10			20	30	40	kN/m³
312.05 0.00	GROUND SURFACE																				
	Probable SAND, GRAVEL & COBBLES																				
309.15 2.90	END OF PROBE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER Notes: 1) This auger probe forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Auger probe located at U.T.M. coordinates 5 092 982.4 N, 313 959.8 E. 3) Probe elevation obtained from Marshall Macklin Monaghan terrain model.																				



# RECORD OF BOREHOLE AP-20F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+545, on centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 12, 1998 CHECKED BY E.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT NUMBER	TYPE			20	40	60	80					
312.05 0.00	GROUND SURFACE		BLOWS/0.3m											
310.37 1.68	Probable SAND, GRAVEL & COBBLES				311									
	END OF PROBE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER Notes: 1) This auger probe forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Auger probe located at U.T.M. coordinates 5 092 985.7 N, 313 966.0 E. 3) Probe elevation obtained from Marshall Macklin Monaghan terrain model.													



# RECORD OF BOREHOLE AP-30F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+550, on centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 12, 1998 CHECKED BY E.G.

SOIL PROFILE				SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT				UNIT WEIGHT  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m	20      40      60      80				wp ——— w ——— wl								
						SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)								
311.24	GROUND SURFACE															GR    SA    (SI & CL)		
0.00																		
	Probable SAND, GRAVEL & COBBLES																	
308.56																		
2.68	END OF PROBE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER																	
	Notes: 1) This auger probe forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 092 990.1 N, 313 963.7 E. 3) Borehole elevation obtained from Dearden and Stanton Ltd., O.L.S. terrain model.																	



# RECORD OF BOREHOLE AP-40F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+549, offset 6 m right of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 12, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60	80	wp	w	wl			
311.83 0.00	GROUND SURFACE														GR	SA	(SI & CL)
	Probable SANDY GRAVEL																
310.00 1.83	Probable SAND, GRAVEL & COBBLES																
309.09 2.74	END OF PROBE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER																
	Notes: 1) This auger probe forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Auger probe located at U.T.M. coordinates 5 092 992.1 N, 313 969.4 E. 3) Probe elevation obtained from Marshall Macklin Monaghan terrain model.																



# RECORD OF BOREHOLE AP-50F 1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station +10+586, offset 7 m left of centreline of Southbound Lane

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Standard augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE November 12, 1998

CHECKED BY E.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m	20	40	60	80					
306.65	GROUND SURFACE													
0.00														
305.13	Probable SILTY SAND & GRAVEL													
1.52														
	Probable SAND, GRAVEL & COBBLES													
302.54														
4.11	END OF PROBE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER													
	Notes: 1) This auger probe forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Auger probe located at U.T.M. coordinates 5 093 019.6 N, 313 940.8 E. 3) Probe elevation obtained from Marshall Macklin Monaghan terrain model.													



# RECORD OF BOREHOLE AP-60F 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station +10+585, on centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 12, 1998 CHECKED BY E.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA	NUMBER	TYPE	BLOWS/0.3m	20	40	60	80	wp	w	wl	WATER CONTENT (%)		
307.39	GROUND SURFACE													kN/m <sup>3</sup>	GR SA (SI & CL)
0.00															
	Probable SILTY SAND & GRAVEL														
305.56															
1.83															
	Probable SAND, GRAVEL & COBBLES														
304.71															
2.68	END OF PROBE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER														
	Notes: 1) This auger probe forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 021.8 N, 313 947.5 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model.														



# RECORD OF BOREHOLE AP-70F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station +10+590, on centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 12, 1998 CHECKED BY E.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m	20	40	60	80					
306.10 0.00	GROUND SURFACE													
	Probable SILTY CLAY													
303.66 2.44														
	Probable SAND, GRAVEL & COBBLES													
302.38 3.72	END OF PROBE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER Notes: 1) This auger probe forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Auger probe located at U.T.M. coordinates 5 093 026.4 N, 313 945.3 E. 3) Probe elevation obtained from Marshall Macklin Monaghan terrain model.													





# RECORD OF BOREHOLE AP-80F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10 + 590, offset 6 m right of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 12, 1998 CHECKED BY E.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N Value)				CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA	NUMBER	TYPE	BLOWS/0.3m	20	40	60	80	20	40	60	80	wp	w	wl	WATER CONTENT (%)		
305.77	GROUND SURFACE																	kN/m <sup>3</sup>	GR SA (SI & CL)
0.00	Probable SILT																		
305.16																			
0.61																			
	Probable SAND, GRAVEL & COBBLES																		
302.72																			
3.05	END OF PROBE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER																		
	Notes: 1) This auger probe forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Auger probe located at U.T.M. coordinates 5 093 029.0 N, 313 950.7 E. 3) Probe elevation obtained from Marshall Macklin Monaghan terrain model.																		



# RECORD OF BOREHOLE AP&C-1EF<sub>1</sub> OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+639, offset 6 m right of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Dynamic cone / Standard auger COMPILED BY M.D.  
 DATUM Geodetic DATE June 10, 1998 CHECKED BY I.G.

SOIL PROFILE				SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)		CONE PENETRATION TEST		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m	SHEAR STRENGTH: Cu, KPa			WATER CONTENT (%)		wp	w	wl							
						UNCONFINED QUICK TRIAXIAL			FIELD VANE LAB SHEAR											
301.02 0.00	GROUND SURFACE (Topsoil scraped off)																			GR SA (SI & CL)
	Probable SILTY CLAY																			
295.84 5.18	END OF CONE TEST DUE TO 'BOUNCING' REFUSAL ON PROBABLE BEDROCK OR POSSIBLE BOULDER																			
	Notes: 1) This cone test forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Auger probe & cone test located at U.T.M. coordinates 5 093 073.9 N, 313 929.9 E. 3) Probe elevation obtained by Marshall Macklin Monaghan survey crew.																			



# RECORD OF BOREHOLE AP&C-2EF<sub>1</sub> OF 1

METRIC

W.P. 774-93-00

LOCATION Station ~ 10+670, offset ~ 7 m left of centreline of Southbound Lane

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Dynamic cone / Standard auger

COMPILED BY M.D.

DATUM Geodetic

DATE June 10, 1998

CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION		
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	20	40	60	80	wp	w			wl	10
299.84 0.00	GROUND SURFACE (Topsoil scraped off)																
	Assumed SILTY CLAY																
294.87 4.97	END OF CONE TEST DUE TO 'BOUNCING' REFUSAL ON PROBABLE BEDROCK OR POSSIBLE BOULDER																
	Notes: 1) This cone test forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Auger probe located at U.T.M. coordinates 5 093 097.8 N, 313 905.6 E. 3) Probe elevation obtained by Marshall Macklin Monaghan survey crew.																



# RECORD OF BOREHOLE C-110F 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station 10+697, offset 7 m left of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Dynamic cone / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 25, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	UNCONFINED QUICK TRIAXIAL	Cu, KPa	FIELD VANE LAB SHEAR	WATER CONTENT (%)	wp	w		
300.01 0.00	GROUND SURFACE Dynamic cone test only.						20 40 60 80			10 20 30 40					GR SA (SI & CL)
293.33 6.68	END OF CONE TEST DUE TO 'BOUNCING' REFUSAL ON PROBABLE BEDROCK OR POSSIBLE BOULDER  <b>Notes:</b> 1) This cone test forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Cone test located at U.T.M. coordinates 5 093 123.4 N, 313 895.4 E. 3) Cone elevation obtained from Marshall Macklin Monaghan terrain model.														



# RECORD OF BOREHOLE C-120F 1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station 10+693, offset on centreline of Southbound Lane

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Dynamic cone / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE November 25, 1998

CHECKED BY E.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (meters)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA	NUMBER			TYPE	BLOWS/0.3m	UNCONFINED QUICK TRIAXIAL	SHEAR STRENGTH: Cu, KPa	FIELD VANE LAB SHEAR	WATER CONTENT (%)	wp	w		
300.15	GROUND SURFACE														
0.00	Dynamic cone test only.														
293.94															
6.22	END OF CONE TEST DUE TO 'BOUNCING' REFUSAL ON PROBABLE BEDROCK OR POSSIBLE BOULDER														
	Notes: 1) This cone test forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 122.2 N, 313 903.4 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model.														



# RECORD OF BOREHOLE C-130F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+697, on centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Dynamic cone / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 25, 1998 CHECKED BY E.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER TYPE			BLOWS/0.3m	UNCONFINED QUICK TRIAXIAL	Cu, KPa FIELD VANE LAB SHEAR	wp			w
300.57 0.00	GROUND SURFACE Dynamic cone test only.					20 40 60 80						
293.96 6.61	END OF CONE TEST DUE TO 'BOUNCING' REFUSAL ON PROBABLE BEDROCK OR POSSIBLE BOULDER  Notes: 1) This cone test forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Cone test located at U.T.M. coordinates 5 093 126.0 3) Cone elevation obtained from Marshall Macklin Monaghan terrain model.											



# RECORD OF BOREHOLE C-140F 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station +10+693, offset +6 m right of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Dynamic cone / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 25, 1998 CHECKED BY E.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT				UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT NUMBER	TYPE			BLOWS/0.3m	20	40	60	80	wp	w	wl		
300.12 0.00	GROUND SURFACE Dynamic cone test only.				300										
					299										
					298										
					297										
					296										
					295										
294.06 6.06	END OF CONE TEST DUE TO 'BOUNCING' REFUSAL ON PROBABLE BEDROCK OR POSSIBLE BOULDER  <b>Notes:</b> 1) This cone test forms part of the Trout Creek Bridge, Southbound Lane Foundation Investigation. 2) Cone test located at U.T.M. coordinates 5 093 124.3 N, 313 909.2 E. 3) Cone elevation obtained from Marshall Macklin Monaghan terrain model.														



# RECORD OF BOREHOLE C-3EF

1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+712, offset ~6 m right of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Dynamic cone / COMPILED BY M.D.  
 DATUM Geodetic DATE June 24, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			CONES/0.3m	CONE PENETRATION TEST							
							20	40	60	80					
300.59	GROUND SURFACE														
0.00	Dynamic cone penetration test only.														
293.40	END OF CONE TEST DUE TO 'BOUNCING' REFUSAL ON PROBABLE BEDROCK OR POSSIBLE BOULDER														
7.19	Notes: 1) This cone test forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Cone test located at U.T.M. coordinates 5 093 142.3 N, 313 902.2 E. 3) Cone elevation obtained by Marshall Macklin Monaghan survey crew.														





# RECORD OF BOREHOLE C-4EF

1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station 10+744, offset 7 m left of centreline of Southbound Lane

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Dynamic cone /

COMPILED BY M.D.

DATUM Geodetic

DATE June 26, 1998

CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m					
303.19	GROUND SURFACE											
0.00	Dynamic cone penetration test only.											
294.00	END OF CONE TEST DUE TO 'BOUNCING' REFUSAL ON PROBABLE BEDROCK OR POSSIBLE BOULDER											
9.19	Notes: 1) This cone test forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Cone test located at U.T.M. coordinates 5 093 168.5 N, 313 879.1 E. 3) Cone elevation obtained by Marshall Macklin Monaghan survey crew.											

5093170.0N

313885.9E



# RECORD OF BOREHOLE C-5EF

1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+673, offset 1 m right of centreline of Southbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Dynamic cone / COMPILED BY M.D.  
 DATUM Geodetic DATE June 10, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	UNCONFINED QUICK TRIAXIAL	Cu, KPa	wp	w	wl	WATER CONTENT (%)		
300.08	GROUND SURFACE														
0.00	Dynamic cone penetration test only.														
294.44	END OF CONE TEST DUE TO 'BOUNCING' REFUSAL ON PROBABLE BEDROCK OR POSSIBLE BOULDER														
5.64	Notes: 1) This cone test forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Cone test located at U.T.M. coordinates 5 093 103.7 N, 313 911.9 E. 3) Cone elevation obtained by Marshall Macklin Monaghan survey crew.														



# RECORD OF BOREHOLE C-6EF

1 OF 1

METRIC

W.P. 774-93-00	LOCATION Station ~10+645, offset ~1 m right of centreline of Southbound Lane	ORIGINATED BY I.D.
DIST 54 HWY 11	BOREHOLE TYPE Dynamic cone /	COMPILED BY M.D.
DATUM Geodetic	DATE June 10, 1998	CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION		
ELEV. DEPTH	DESCRIPTION	STRATA PLOT NUMBER	TYPE			BLOWS/0.3m	20	40	60	80	wp	w	wl			10	20
300.50	GROUND SURFACE																
0.00	Dynamic cone penetration test only.																
295.90																	
4.60	<b>END OF CONE TEST DUE TO 'BOUNCING' REFUSAL ON PROBABLE BEDROCK OR POSSIBLE BOULDER</b>  <b>Notes:</b> 1) This cone test forms part of Trout Creek Bridge Southbound Lane Foundation Investigation. 2) Cone test located at U.T.M. coordinates 5 093 077.5 N, 313 922.9 E. 3) Cone elevation obtained by Marshall Macklin Monaghan survey crew.																



# RECORD OF BOREHOLE C-7EF

1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station 10+743, on centreline of Southbound Lane

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Dynamic cone /

COMPILED BY M.D.

DATUM Geodetic

DATE June 26, 1998

CHECKED BY I.G.

SOIL PROFILE				SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20	40						60
303.70	GROUND SURFACE							SHEAR STRENGTH: Cu, KPa		WATER CONTENT (%)					
0.00	Dynamic cone penetration test only.							UNCONFINED QUICK TRIAXIAL		FIELD VANE LAB SHEAR		10 20 30 40		GR SA (SI & CL)	



SO7524G/O/F						
TABLE 1 ROCK CORE DESCRIPTION						
BH#	Core Recovery				Core Description	
	RC #	Depth (m)	% CR*	% RQD**	Depth (m)	Description
<b>SOUND BOUND LANE</b>						
2-OF	1	3.14 to 4.66	100	78	3.14 to 6.46	<b>Biotite Hornblende Gneiss</b> - light grey to pink, medium to coarse grained, unweathered with sulfide inclusions, very close spacing of fractures dipping 45° to 90° from vertical, planar to slightly undulated, rough
	2	4.66 to 6.46	100	88		
3-OF	1	3.35 to 4.81	100	69	3.35 to 6.55	<b>Biotite Hornblende Gneiss</b> - light grey, medium to coarse grained, unweathered with sulfide inclusions, moderately spaced fractures and joints dipping 45° to 90° from vertical, rough and slightly undulating
	2	4.81 to 6.55	100	97		
5-OF	1	5.27 to 6.67	100	90	5.27 to 8.78	<b>Biotite Hornblende Gneiss</b> , light grey to pink, medium to coarse grained, slightly weathered, very close spacing of cemented fissures, joints and fissures dipping 0° to 90° from vertical, smooth and slightly undulating
	2	6.67 to 8.78	100	90		

S07524G/O/F

**TABLE 1**  
**ROCK CORE DESCRIPTION**

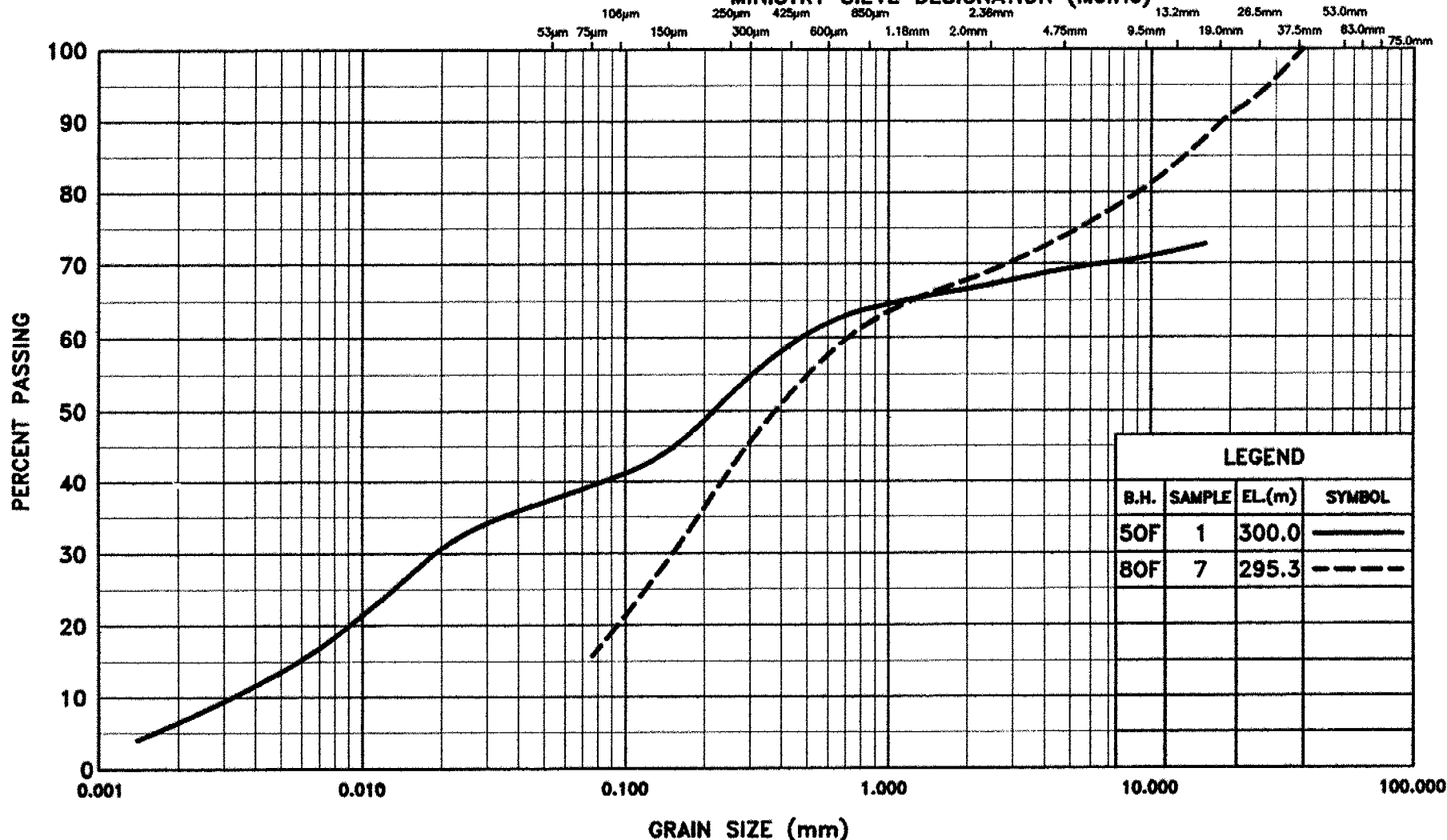
BH#	Core Recovery				Core Description	
	RC #	Depth (m)	% CR*	% RQD**	Depth (m)	Description
7-OF	1	6.37 to 7.90	100	71	6.37 to 9.60	<b>Biotite Hornblende Gneiss (Garnetiferous)</b> , light grey to grey pink, medium to coarse grained, unweathered, moderate spacing of fractures dipping 90° from vertical, slightly undulated, rough
	2	7.90 to 9.60	100	86		
8-OF	1	8.29 to 9.84	100	95	8.29 to 11.43	<b>Biotite Hornblense Gneiss (Garnetiferous)</b> , light grey to grey-pink, medium to coarse grained, unweathered, moderate spacing of joints and fissures, some sulfide inclusions, close spacing of fractures dipping 45° to 90° from vertical, planar and smooth
	2	9.84 to 11.43	100	77		
9-OF	1	14.42 to 15.95	100	81	14.42 to 17.74	<b>Biotite Hornblende Gneiss (Garnetiferous)</b> , light grey to pink, unweathered, very close spacing of cemented fractures, inclusions of sulfides, fractures dipping 0° to 90° from vertical, rough and undulating
	2	15.95 to 17.74	100	100		
<b>*CR</b> = Core Recovery <b>**RQD</b> = Rock Quality Designation						

C

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



LEGEND			
B.H.	SAMPLE	EL.(m)	SYMBOL
50F	1	300.0	————
80F	7	295.3	-----

Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION

SAND & GRAVEL

FIGURE C-1

W.P. 774-93-00



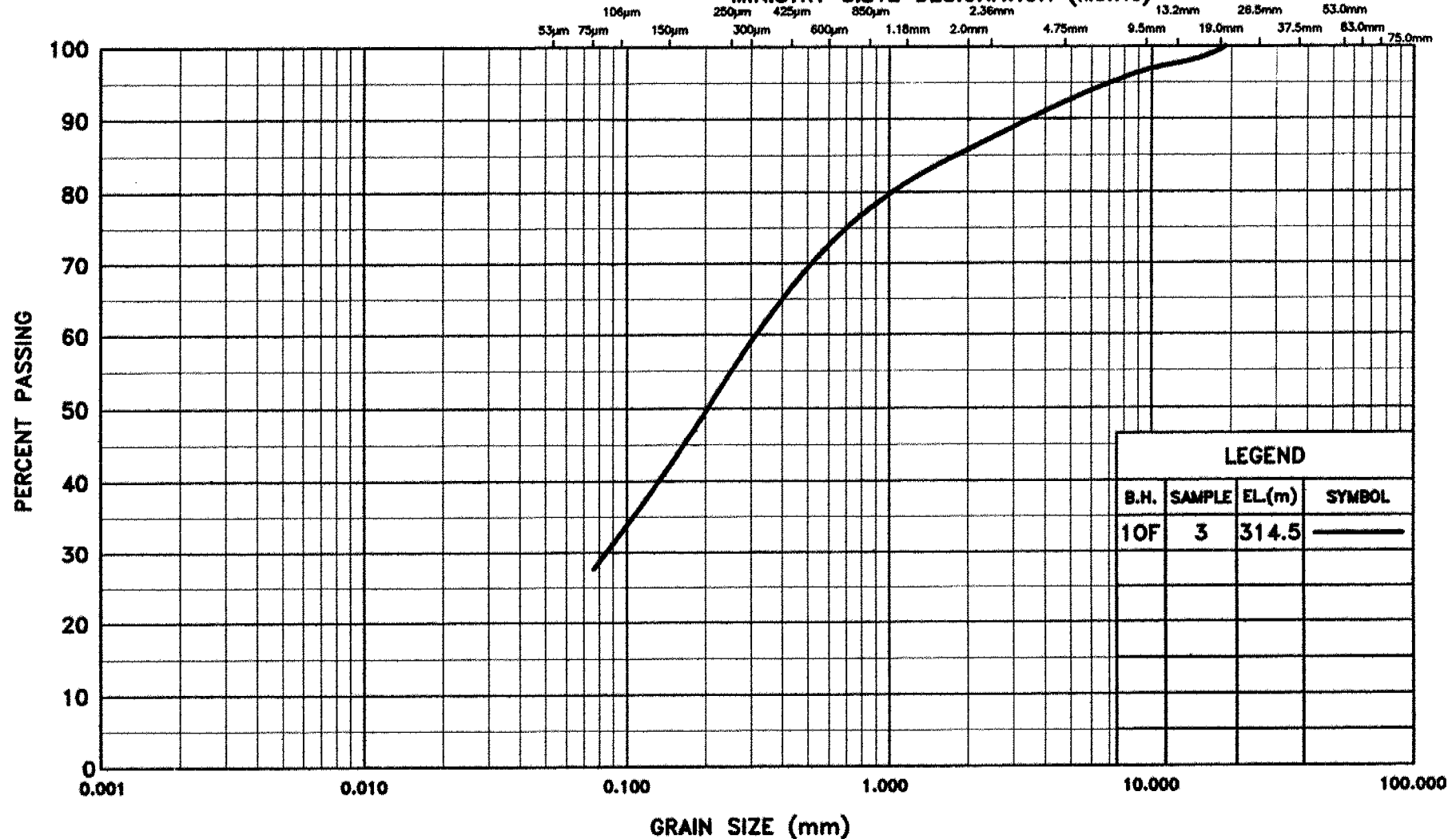
PROJ. No. S07524G0



# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



### LEGEND

B.H.	SAMPLE	EL.(m)	SYMBOL
10F	3	314.5	—

Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION

SAND

FIGURE C-2

W.P. 774-93-00

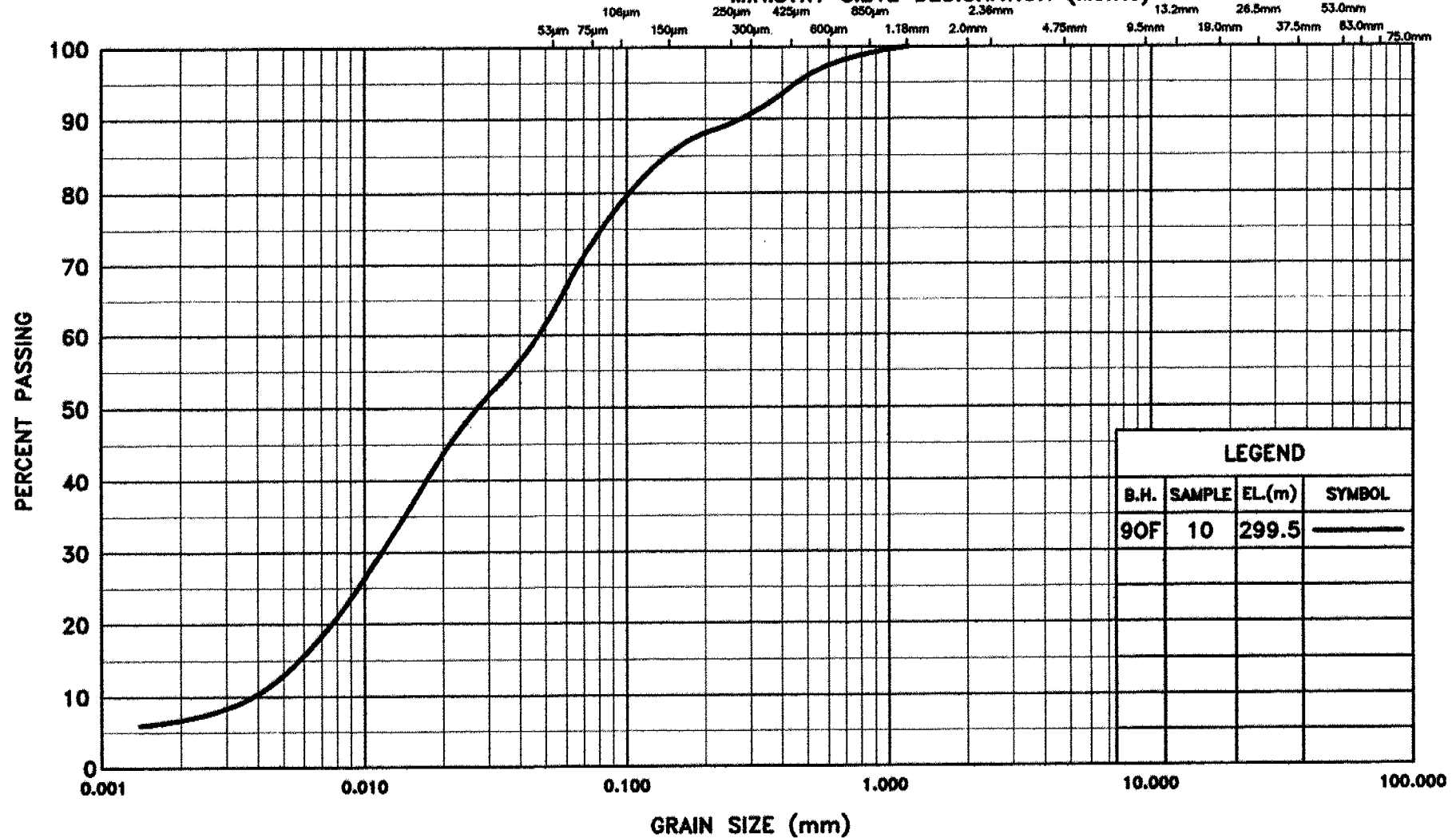


PROJ. No. S07524G0

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION

SANDY SILT

FIGURE C-3

W.P. 774-93-00

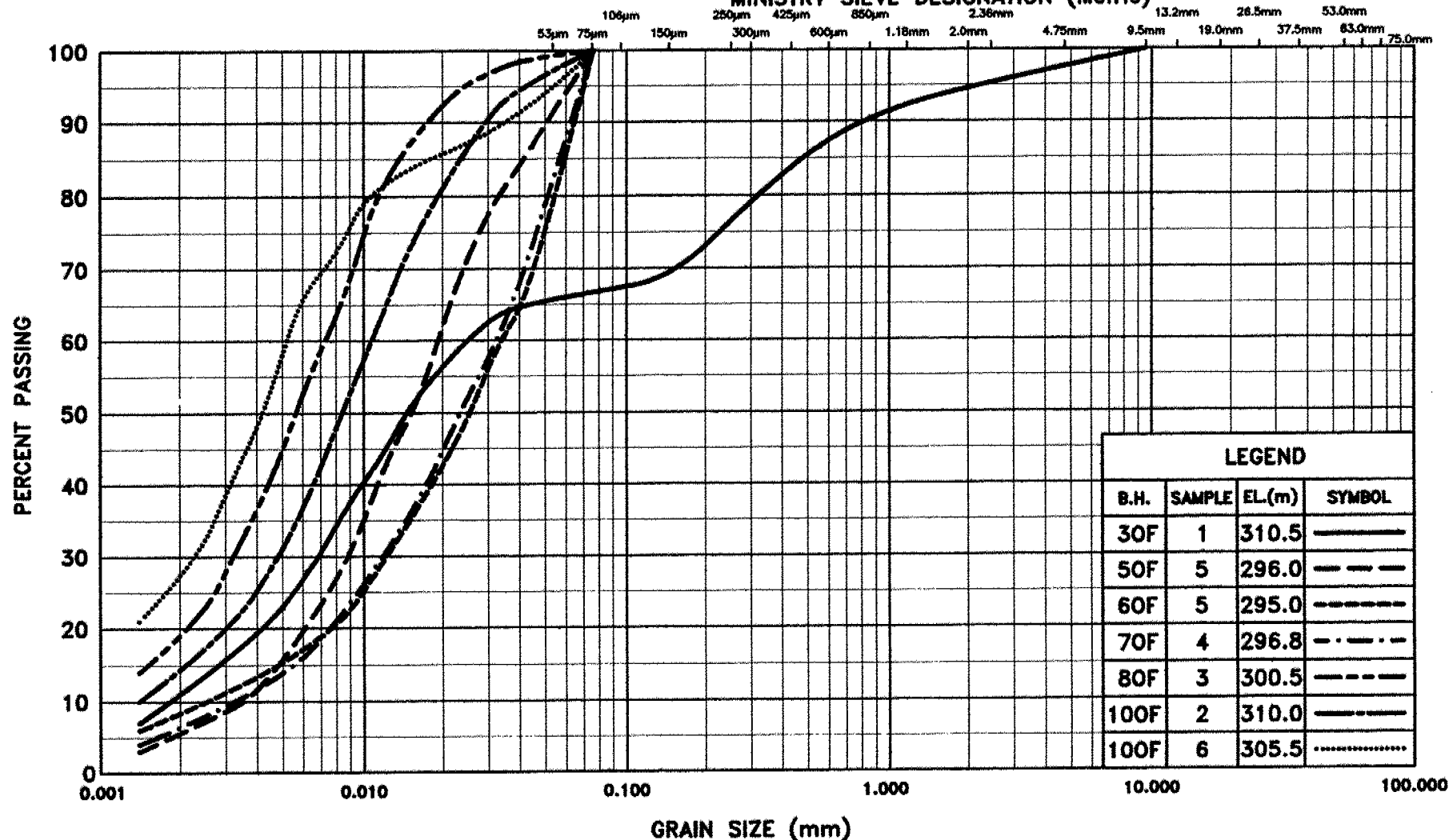


PROJ. No. S07524GO

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION

SILTY CLAY

FIGURE C-4

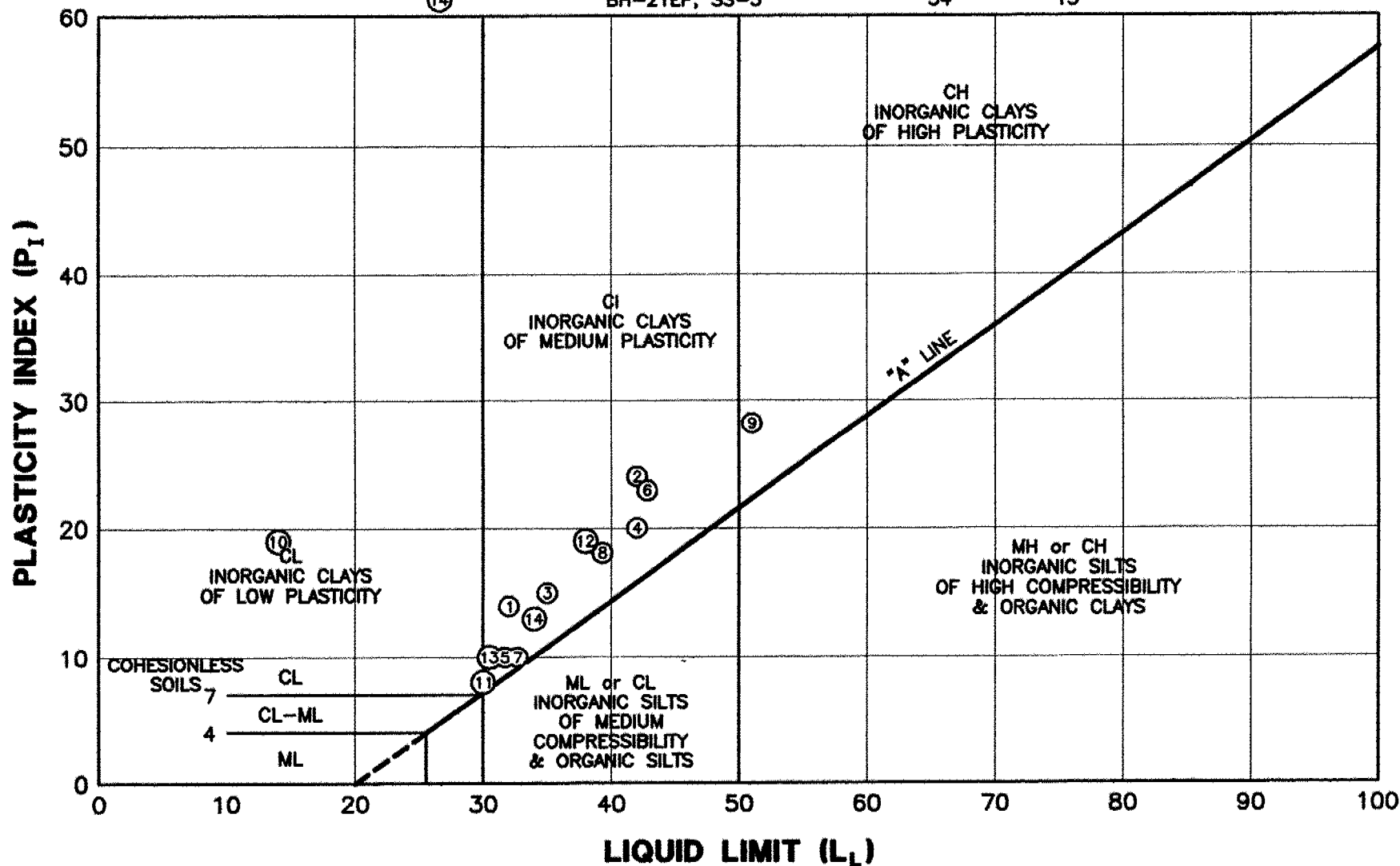
W.P. 774-93-00



PROJ. No. S07524G0

# **ATTERBERG LIMITS - PLASTICITY CHART**

SYMBOL	DESCRIPTION	LL	PI
①	BH-1EF, SS-5	32	14
②	BH-4EF, SS-2	42	24
③	BH-4EF, SS-4	35	15
④	BH-13EP, SS-2	42	20
⑤	BH-17EP, SS-2	32	10
⑥	BH-21EP, SS-2	43	23
⑦	BH-21EP, TW-4	33	10
⑧	BH-80F, SS-3	39	18
⑨	BH-100F, SS-6	51	28
⑩	BH-90F, SS-2	41	19
⑪	BH-100F, SS-3	30	8
⑫	BH-100F, SS-5	38	19
⑬	BH-100F, SS-8	31	10
⑭	BH-21EP, SS-5	34	13

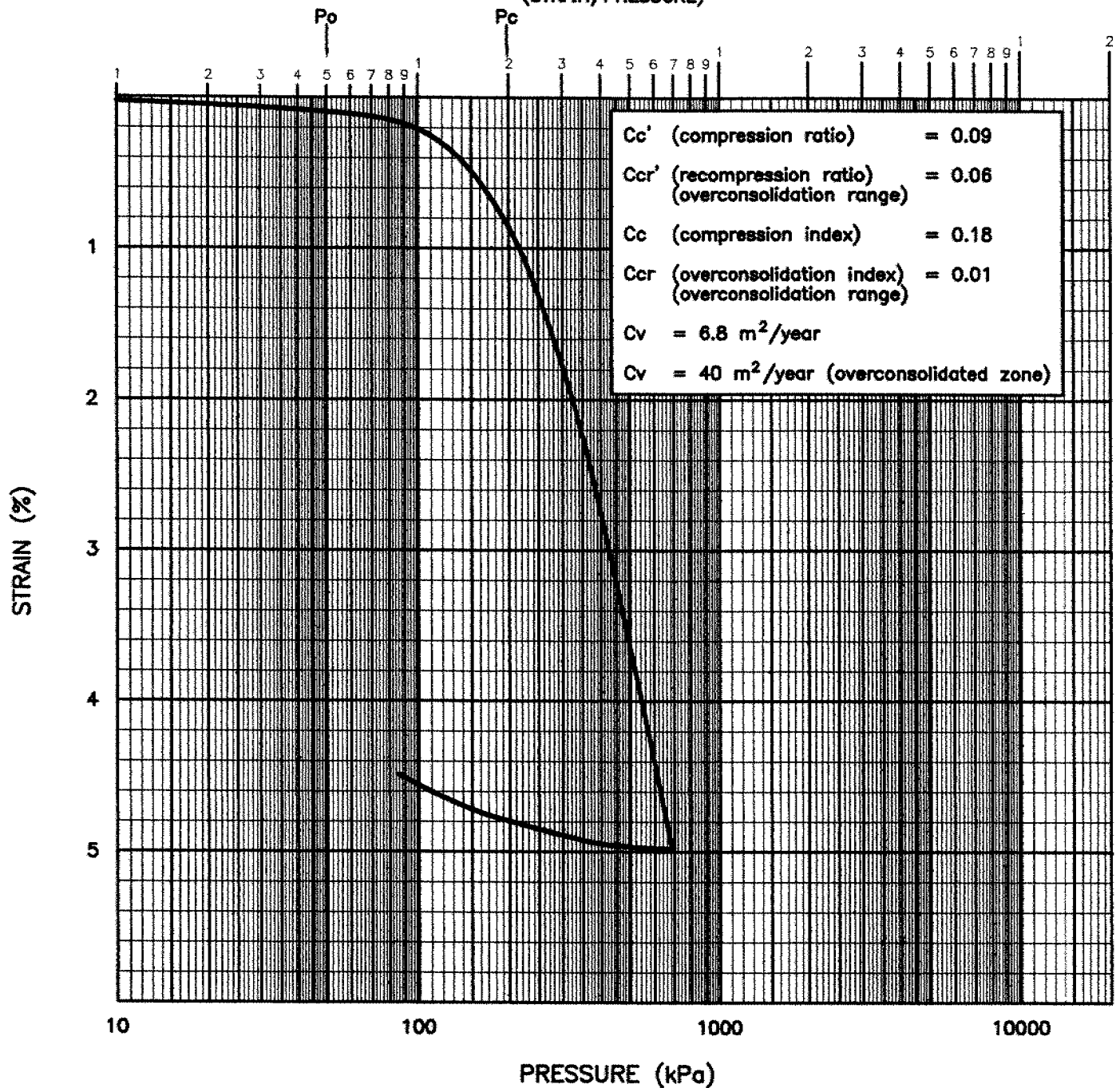


S0752460

FIG. No. C-5



# CONSOLIDATION TEST RESULTS (STRAIN/PRESSURE)



BOREHOLE No. 21EP  
 DEPTH 3 m  
 MOISTURE CONTENT 30.3 %  
 LIQUID LIMIT 32 %  
 PLASTIC LIMIT 22 %  
 $e_o$  1.02  
 UNIT WEIGHT 18.6 kN/m<sup>3</sup>

## SAMPLE DESCRIPTION

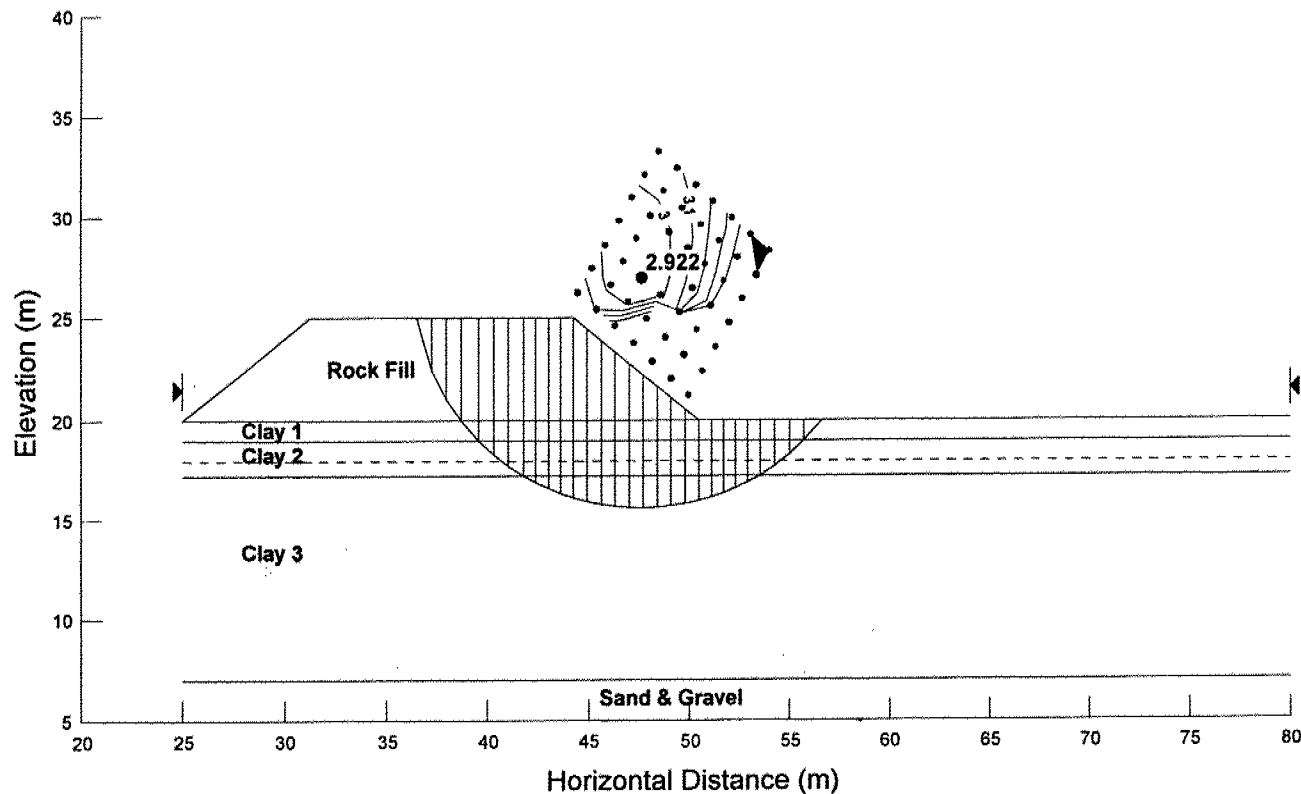
SILTY CLAY with firm to stiff, grey SILT layers

$$M_v = \frac{\Delta \text{ strain}}{\Delta \text{ pressure}} = 0.032 \text{ MPa}^{-1}$$

(COEFFICIENT OF VOLUME COMPRESSIBILITY)

D

Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-C/G)  
 North Abutment - Southbound Lanes  
 5 metre embankment height, 1.25:1 side slopes  
 N\_S5H.SLP



Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 100  
 Rate of Increase -30  
 Cv - Minimum 70  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 70  
 Rate of Increase -28.57  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Clay 3  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 50  
 Rate of Increase 1.95  
 Cv - Maximum 70  
 Ch/Cv Ratio 1

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39

Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-C/G)  
 North Abutment - Southbound Lanes  
 6 metre embankment height, 1.25:1 side slopes  
 N\_S6H.SLP

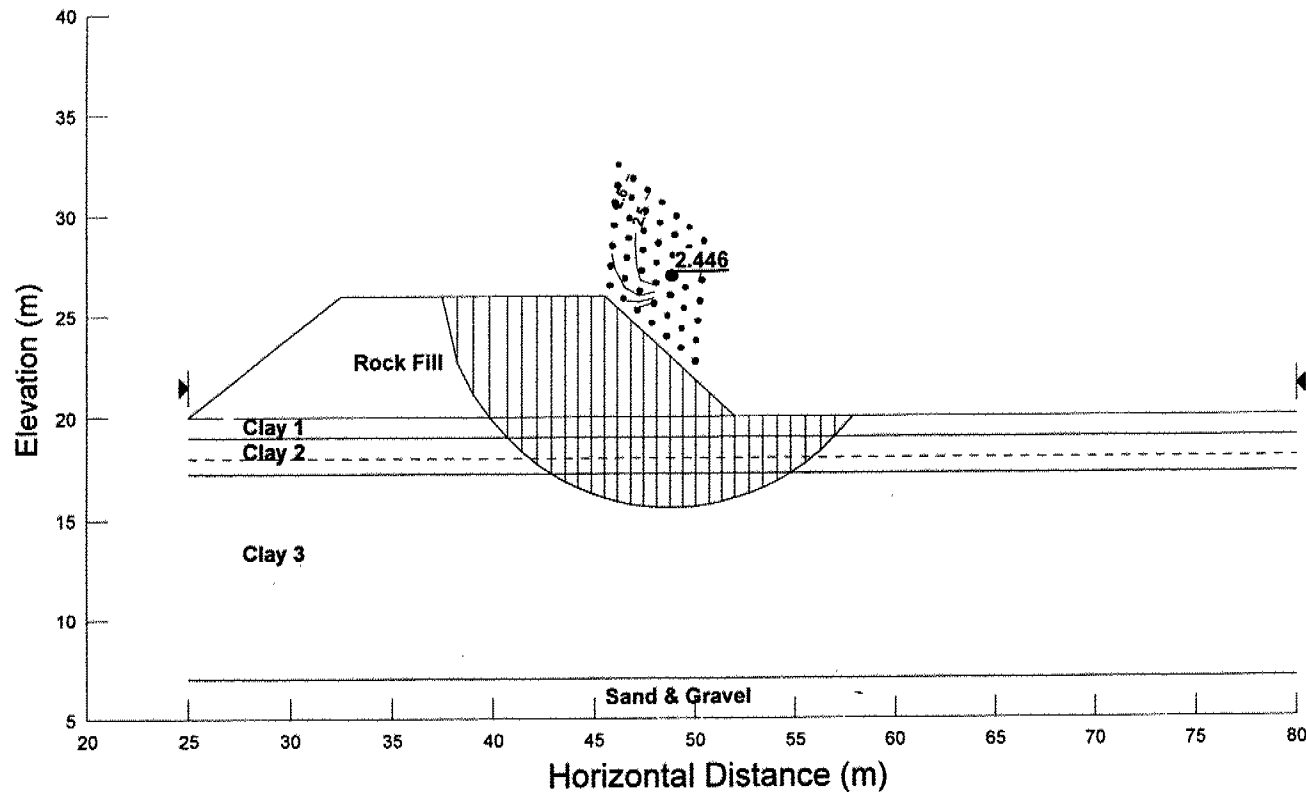
Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 100  
 Rate of Increase -30  
 Cv - Minimum 70  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 70  
 Rate of Increase -28.57  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Clay 3  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 50  
 Rate of Increase 1.95  
 Cv - Maximum 70  
 Ch/Cv Ratio 1

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39





Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-C/G)  
 North Abutment - Southbound Lanes  
 7 metre embankment height, 1.25:1 side slopes  
 4 metre high, 2 metre wide bench  
 N\_S7H.SLP

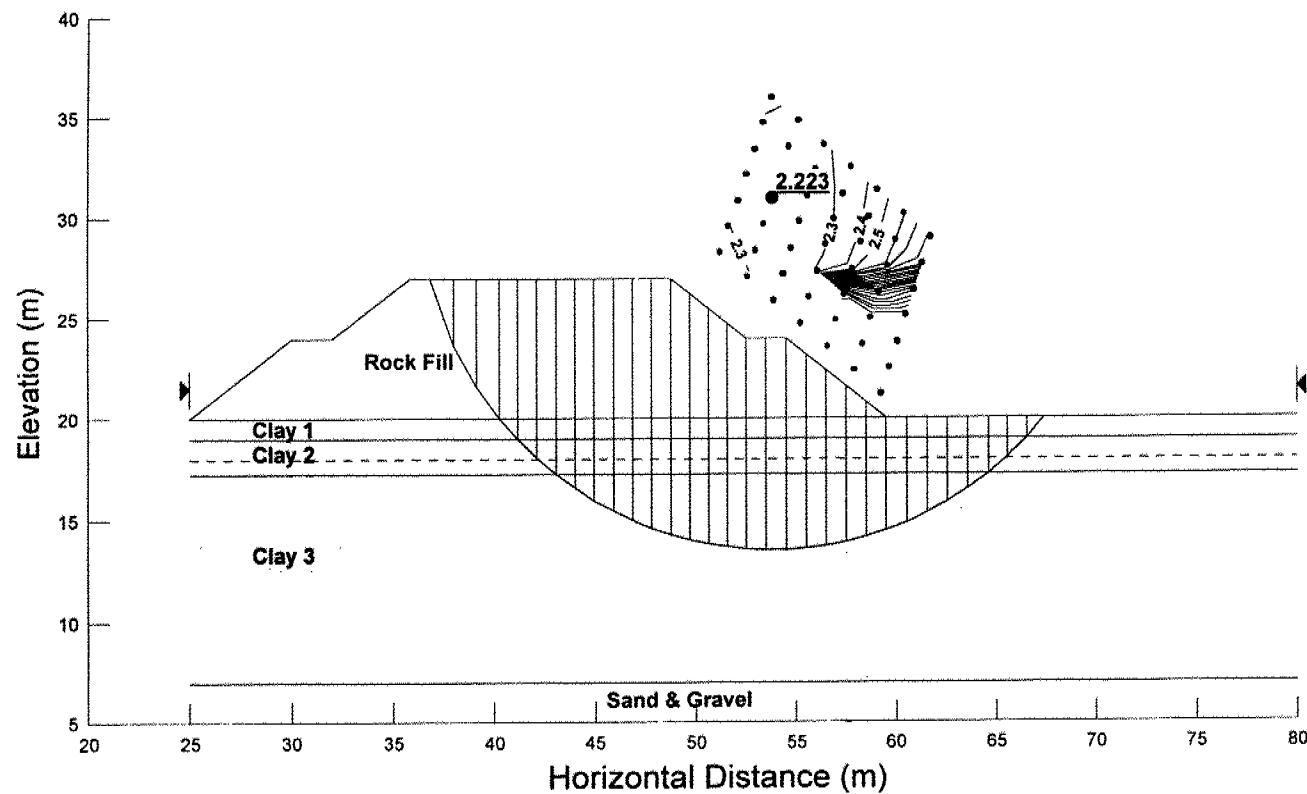
Rock Fill  
 Soil Model Mohr-Coulom  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 100  
 Rate of Increase -30  
 Cv - Minimum 70  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 70  
 Rate of Increase -28.57  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Clay 3  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 50  
 Rate of Increase 1.95  
 Cv - Maximum 70  
 Ch/Cv Ratio 1

Sand & Gravel  
 Soil Model Mohr-Coulom  
 Unit Weight 20  
 Cohesion 0  
 Phi 39



Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-C/G)  
 North Abutment - Southbound Lanes  
 8 metre embankment height, 1.25:1 side slopes  
 5 metre high, 2 metre wide bench  
 N\_S8H.SLP

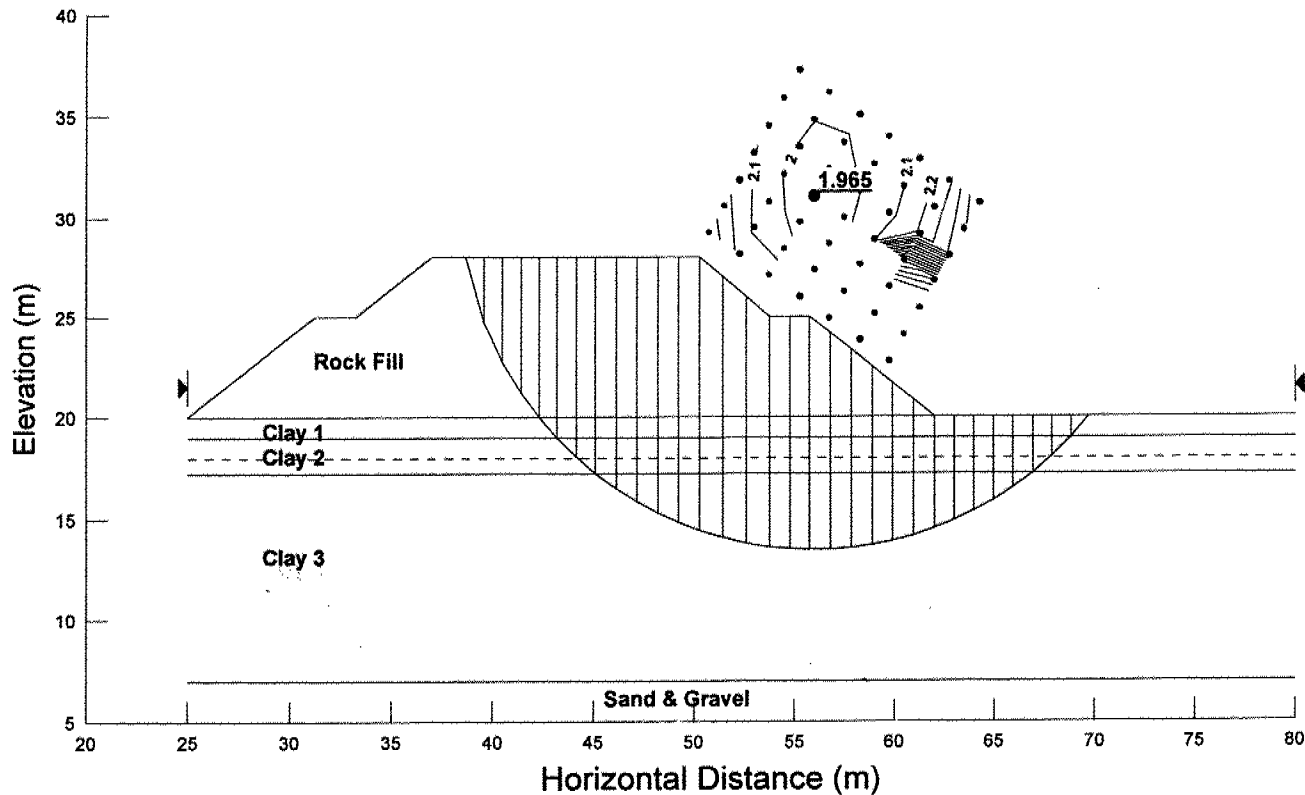
Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 100  
 Rate of Increase -30  
 Cv - Minimum 70  
 Ch/Cv Ratio 1

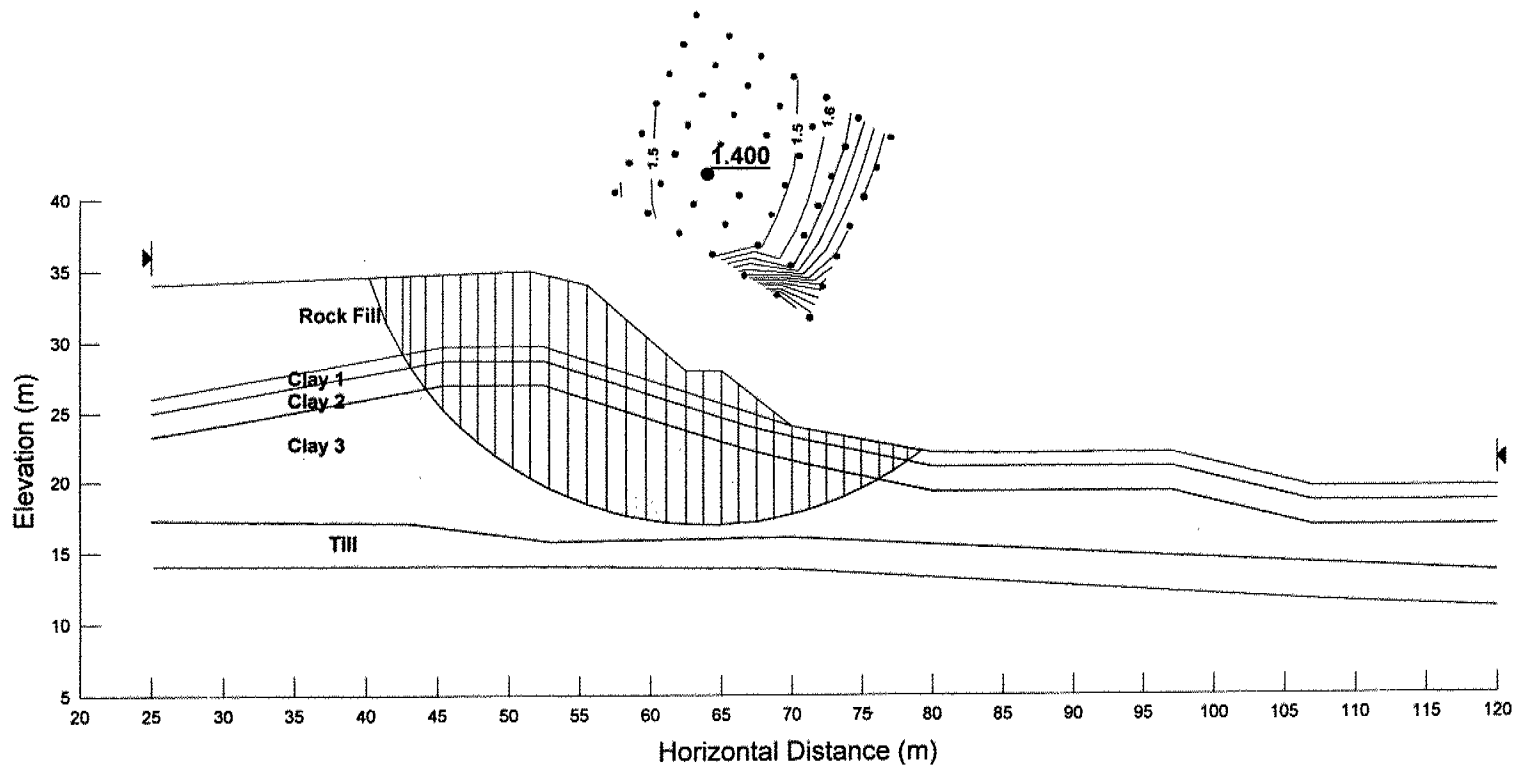
Clay 2  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 70  
 Rate of Increase -28.57  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Clay 3  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 50  
 Rate of Increase 1.95  
 Cv - Maximum 70  
 Ch/Cv Ratio 1

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39



Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-B/G)  
 Southbound Lane, North Abutment  
 With Extended Berm  
 SBL\_NALV.SLP



Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 100  
 Rate of Increase -30  
 Cv - Minimum 70  
 Ch/Cv Ratio 1

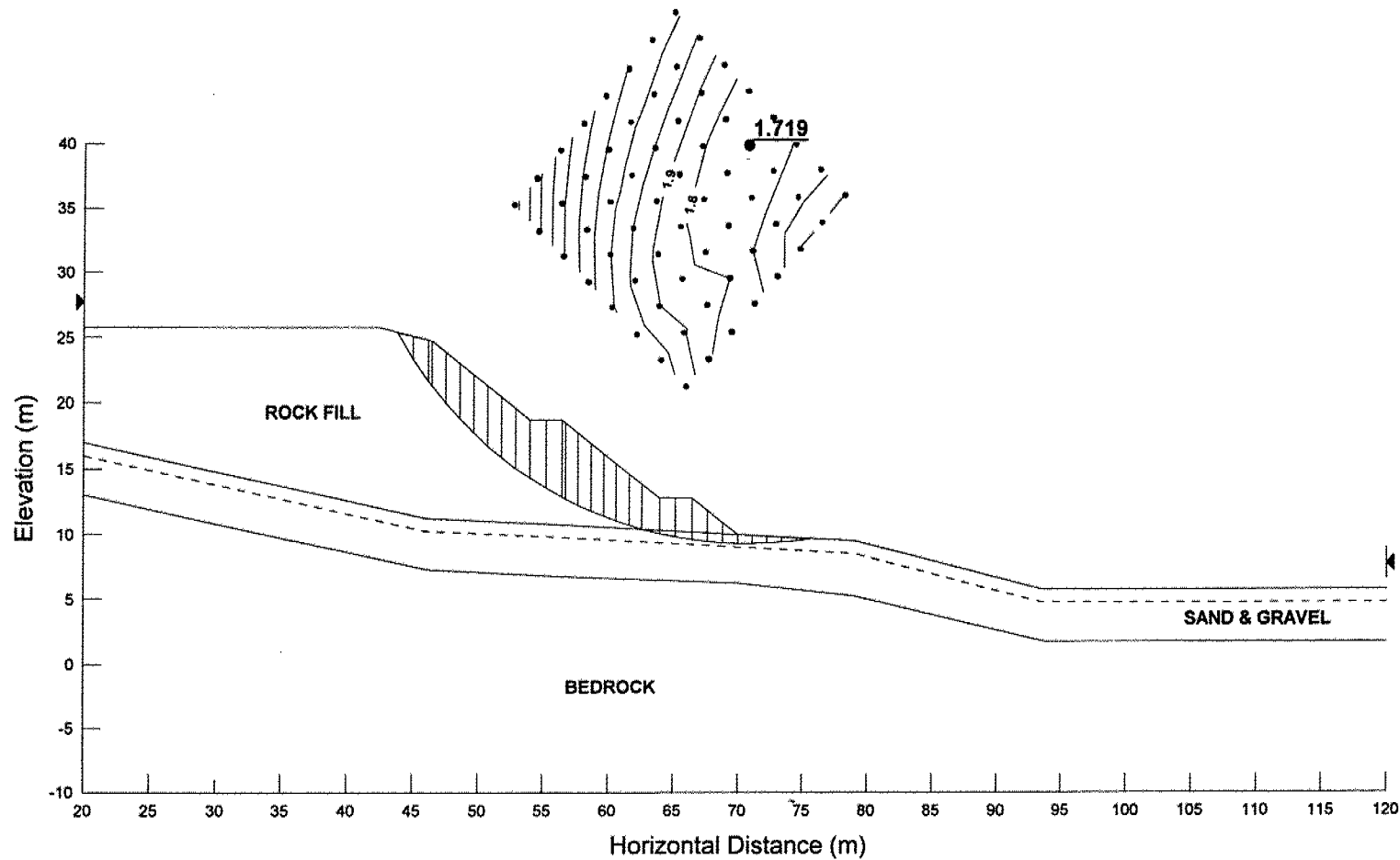
Clay 2  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 70  
 Rate of Increase -28.57  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Clay 3  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 50  
 Rate of Increase 1.95  
 Cv - Maximum 70  
 Ch/Cv Ratio 1

Till  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35  
 Unsaturated Phi B 0

Bedrock  
 Soil Model Bedrock  
 Unit Weight -1  
 Piezometric Line # 0  
 Pore-Air Pressure 0

Slope Stability - Total Stress Analysis  
Trout Creek - Highway 11 (F-98179-C/G)  
Southbound Lane, South Abutment  
With Extended Berm  
SBL\_SA.SLP



Rock Fill  
Soil Model Mohr-Coulomb  
Unit Weight 20  
Cohesion 0  
Phi 42

Sand and Gravel  
Soil Model Mohr-Coulomb  
Unit Weight 21  
Cohesion 0  
Phi 35

Bedrock  
Soil Model Bedrock

**Foundation Investigation & Design Report  
Bridge Structure & Approaches  
Trout Creek (Site 44-371N)  
NORTHBOUND LANES  
Trout Creek By-Pass, King's Highway 11  
District 54, Sudbury, Ontario  
GWP No. 774-93-00**

Prepared For:

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November 24, 1999

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## Preface

Work Project GWP 774-93-00 is one of a series of projects for the four lane expansion of Highway 11. This project involves the four lane design of Highway 11, from 4.0 km south of Highway 522, northerly for 7.9 km. It will result in the construction of a by-pass of the existing Highway 11 to the west of the Town of Trout Creek.

This highway section is located in the Townships of Laurier and South Himsworth, within the geographic District of Parry Sound, as shown on the Key Plan, Figure A1 in Appendix A. The project requires geotechnical input for the following major components:

- New pavement design for the entire length of the four lane by-pass, including associated service roads.
- New structure, Trout Creek South Interchange (underpass), Site 44-372.
- **New structure, Trout Creek Northbound Lanes, Site 44-371N.**
- New structure, Trout Creek Southbound Lanes, Site 44-371N.
- New structure, Highway 522 (underpass), Site 44-370.
- New structure, Trout Creek North Interchange (underpass), Site 44-369.

This report deals with the new bridge structure for the **northbound lanes** at the proposed Trout Creek crossing, Site 44-371N, as well as the approach fills. Separate reports will be submitted for the additional components.

Part 1 of this report, Foundation Investigation, describes the geotechnical investigation methodology and the results derived, as pertaining to the design parameters required for the bridge foundations and related earthworks.

Part 2 of this report, Engineering Discussion and Recommendations, addresses the geotechnical engineering design considerations pertaining to the bridge foundations and the approaches.

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## **Appendix A**

Figure A1.	Key Plan
Figure A2a.	Borehole Location Plan and Profile - South
Figure A2b.	Borehole Location Plan and Profile - North
Figure A3.	Undrained Shear Strength, Atterberg Limits and Effective Stress Profiles
Figure A4.	Undrained Shear Strength - All Boreholes
Figure A5.	Footing SLS Bearing Resistance - Granular Fill on Native Soil - SOUTH APPROACH
Figure A6.	Estimated Consolidation Settlement - SOUTH APPROACH
Figure A7.	Estimated Consolidation Settlement - NORTH APPROACH

## **Appendix B**

Borehole Logs and Rock Descriptions

## **Appendix C**

Results of Laboratory Testing (grain size, Atterberg Limits, consolidation tests)

## **Appendix D**

Stability Analysis Printouts

## **Rear Pockets**

Drawing No. 1. Bridge Site Plan & Profile  
Drawing No. 2. Bridge Site Plan & Sections



## **Part 1 Foundation Investigation**

### **1.1 Introduction**

This report presents the results of a geotechnical foundation investigation by Trow Consulting Engineers Ltd. (Trow) for the proposed bridge structure and approaches for the northbound lanes (NBL) at the Highway 1, Trout Creek crossing at Site 44-371N.

An original investigation was completed at this site in June and July of 1998 for an originally proposed 105 m long ( $\pm$ ) three span structure. However, for technical reasons based on the results of the original investigation, as discussed more fully in Part 2 of this report, a longer (242 m), five span bridge was selected. Accordingly, a supplementary investigation was completed during September and November, 1998 which investigated the subsurface conditions at the revised pier and abutment locations of the new proposed five span structure, where conditions were not previously investigated as part of the original work.

This report contains the results of the supplementary investigation, plus the relevant results of the original investigation, compiled for the five span structure arrangement of the Highway 11, northbound lane crossing of Trout Creek. It is Trow's understanding that the 5 span structure will be located with the central span crossing Trout Creek. The structure will include an approximately 11 metre high south abutment, 22 metre high centre span, and 5 metre high north abutment. The span lengths will be approximately 40, 54, 54, 54, 40 metres in length, creating a bridge with a length of about 242 metres.

### **1.2 Site Description and Geological Setting**

The site is located in Lot 30, Concession 2, Township of South Himsforth, District of Parry Sound, along the banks of Trout Creek, about 2 km west of the Town of Trout Creek, and 750 metres south of Highway 552, as shown on Figure A1, in Appendix A.

Generally, the terrain at the site is sloping towards the creek and is well drained. However several drainage gullies run parallel and perpendicular to the creek, with steep embankments on either side of the gullies, creating a highly variable terrain in the vicinity of the proposed structure. The relief

can vary at slopes steeper than 1H:1V within the site. There are mature trees with heavy underbrush across the site.

According to OGS Maps 2544 and 2556, as well as Ontario Geological Survey Map P.3160 (Quaternary Geology, South River Area), the site is located in what is known as the Central Gneiss Belt, with mainly felsic igneous rocks of the Mesoproterozoic Group.

The overburden soils have been mapped as glaciolacustrine deposits consisting of laminated, rhythmically bedded (also referred to as varved) to massive silts and clays. The southernmost portion of the site edges onto an area mapped as a bedrock drift complex with relatively thin, primarily cohesionless soil cover (till).

### **1.3 Investigative Procedures**

#### **1.3.1 General**

Part 1 of this report describes the investigative procedures used for the geotechnical assessment of the northbound lanes crossing of Trout Creek, within the Trout Creek By-Pass, King's Highway 11. Properties of the overburden soils were obtained by *in situ* and laboratory testing and the procedures employed during the investigation are described below.

#### **1.3.2 Field Investigation**

The initial field investigation, or explorations, for the originally proposed three span arrangement was carried out between June 9 and July 2, 1998. Originally these explorations investigated only the pier and abutment locations, as well as the immediate approach embankments. With the discovery of clay at this site additional explorations were advanced, while on site, to outline the vertical and areal extent of the clay within the approach embankments. A further investigation of the north approach was completed September 23 and 24, 1998. The supplementary investigation of the revised, five span, arrangement occurred between November 10 and 24, 1998. The locations of the boreholes, cones and probes, completed as part of these investigations are shown on Figures A2a and A2b, in Appendix A, as well as on Drawings, No. 1 and No. 2, located in the pockets at the end of this report. These locations, as well as the surface elevations, were established from the terrain model for the project, and/or a survey crew from Marshall Macklin Monaghan, and are referenced to geodetic datum.

The investigation of the piers and abutments of the five span arrangement includes 13 boreholes (BH-1NF to BH-10NF, inclusive, BH-8DP, BH-3DF, BH-18DF), two dynamic cone penetration tests (C-3DF, C-5DF), and four auger probes (AP-1NF to AP-4NF inclusive). All of these explorations were advanced to auger refusal or to refusal of the penetration cone ( $N > 100$ ). Six of the boreholes were advanced into the bedrock to obtain core, as described below.

The investigation of the approaches included six additional boreholes (BH-23DP, BH-24DP, BH-10DP, BH-11DP, BH-12DP, BH-18EP). Boreholes BH-23DP and BH-24DP were advanced within the northern approach and were advanced to beneath the upper silt and clay layer. Boreholes BH-10DP, BH-11DP, BH-12DP and BH-18EP were advanced within the southern approach, until auger refusal.

Other explorations completed in the vicinity of the NBL structure, as part of the original three span investigation, or as part of the subsequent investigation to determine the horizontal extent of the clay layer included 16 boreholes (BH-1DF, BH-2DF, BH-4DF, BH-6DP, BH-7DP, BH-9DP, BH-11 EP, BH-13DP, BH-14DP, BH-15DP, BH-16DP, BH-17DP, BH-19DF, BH-20 DP, BH-21DP, BH-22DP), five dynamic cone penetration tests (C-1DF, C-2DF, C-4DF, C-5DF, C-6DF) and two auger probes (AP-1DF, AP-2DF), plus those explorations completed as part of the SBL investigations (see foundation report for the SBL structure).

The boreholes, cones and probes were advanced through the overburden soils using a Bombardier mounted CME-55 drill and a Dietric 50 drill, equipped with solid and hollow stem augers, and supplied by a soils drilling contractor, Master Soil Investigations Limited. At some of the borehole locations, a bulldozer was used to prepare the site for drill access.

Soil samples were obtained by using a 51 mm OD split-spoon sampler in conjunction with standard penetration tests at approximately 0.75 m and 1.5 m intervals. The standard penetration ( $N$ ) values, together with the blows from the dynamic cone penetration tests, were recorded and used to provide an assessment of the compactness or consistency of the overburden soils. Sampling and testing procedures were in general accordance with ASTM D1586.

Several undisturbed, nominal 50 mm diameter, 'Shelby' tube samples were also obtained in the cohesive deposits. Field vane testing was completed in the boreholes and the auger probes throughout the cohesive soils to measure the *in situ* undrained shear strength of the cohesive soils. The field vane used had dimensions of 152 mm long by 70 mm diameter, not including the pointed 45° point. Torque measurement was by using two calibrated scales on a calibrated lever arm threaded to the drill rods. The testing was in general accordance with ASTM D2573.

At each bridge foundation element, conventional rock coring techniques were used to advance one of the explorations approximately 3 metres into the underlying bedrock. A BQ size core barrel and casing was used and core samples of the bedrock were retrieved for rock quality determination and classification. These six explorations advanced into the underlying bedrock included BH-2NF, BH-3NF, BH-5NF, BH3-DF, BH-7NF and BH-10NF.

Details of the soil and bedrock conditions encountered in the boreholes are included on the logs in the attached Appendix B.

### 1.3.3 Laboratory

The laboratory testing program for selected soil samples consisted of the following:

- natural moisture content
- unit weight
- grain size distribution
- laboratory shear tests (lab vane, penetrometer)
- Atterberg limits
- 1-d consolidation test

The laboratory test results are summarized on the attached borehole logs in Appendix B and are also presented in Appendix C. Many of the results are also discussed in the following descriptive sections.

## 1.4 Subsurface Conditions

The borehole locations are shown on the site plans, Figures A2a and A2b in Appendix A, as well as on Drawings, No. 1 and No. 2. Centerline soil profiles are also shown on the Figures and Drawing No. 1 and the soil cross sections at the locations of the foundation elements are shown on Drawing No. 2.

The borehole, auger probe and dynamic cone penetration (CPT) logs are provided in Appendix B. In general, the following main soil layers were encountered, with increasing depth:

- topsoil
- sand
- silty sand
- silty clay

- silty sand and gravel
- bedrock

A summary of the soil strata encountered in the boreholes, and inferred from the probes and dynamic cone penetration tests, is presented below.

#### **1.4.1 Topsoil**

The majority of boreholes encountered a surficial layer of approximately 150 mm to 200 mm of topsoil. At some of the borehole locations, a bulldozer was used to prepare the site for drill access, thus the topsoil in these areas was removed in the process and the logs do not indicate the presence of topsoil.

#### **1.4.2 Sand**

A layer of loose to compact (N-values from 5 to 25), fine to medium grained brown sand was encountered beneath the topsoil at many of the boreholes advanced at the foundation elements. The thickness ranged from less than 200 mm to over 2 m, and was usually underlain by silty clay, as described below. For the boreholes drilled at the foundation elements, the sand was not encountered in Boreholes, BH-5NF, BH-6NF, BH-8NF, BH-3DF, BH-18DF, BH-5DF, BH-10DF or BH-23DF.

#### **1.4.3 Silty Sand**

Brown to grey, very loose to compact silty sand with local gravel was encountered beneath the topsoil in Boreholes, BH-3DF and BH-18DF at the location of Pier WP4, adjacent to the north bank of Trout Creek. The standard penetration indices ranged from about 1 to 17. The soil contained organics such as roots and pieces of wood and may be a recent stream alluvium. This soil was underlain by silty sand and gravel at a depth of about 4 m.

#### **1.4.4 Silty Clay**

Beneath the upper sand or topsoil, a stratum of silty clay was encountered as the principal soil in all boreholes at the locations of the foundation elements except BH-2NF (WP1 - south abutment) and BH-3DF and BH-18DF (WP4 - pier). In BH-2NF, silty sand and gravel, as described in the next section was encountered. In BH-3DF and BH-18DF, silty sand, as described in the previous section was encountered.

The silty clay is relatively thin (3 m to 4 m) at the location of the south abutment and increases in thickness to between about 12 m to 14 m on the north side of Trout Creek, in the areas of Pier WP5 and the north abutment and approach. Generally, the silty clay is thinly laminated with silt (varved). The individual layers, or laminations, varied in thickness from a few millimetres to a few centimetres, but in general were less than about one centimetre. The proportions of clay to silt and clayey silt varied also, but in general the clay portion dominated.

The natural moisture content of the clay varies from about 20% to over 50% (depending on the silt content) and consistency. Atterberg limit determinations provided the following approximate ranges: Plastic Limit, 18 to 23; Liquid Limit, 28 to 50; Plasticity Index, 8 to 30. These data indicate that, in general, the clay can be described as a low plasticity silty clay (CL) to medium plasticity silty clay (CI). Typically, the silty clay is drier near the top and bottom, but there does not appear to be a depth relationship relative to the Atterberg Limits. The laboratory test data are shown on the borehole logs, on Figure A3 in Appendix A, and in Appendix C.

Standard penetration test (SPT) values ranging from about 1 to over 20 were obtained in the silty clay. The higher values were generally obtained within the upper metre, or so. *In situ* field vanes and laboratory shear vane testing, as well as the SPT values, indicate that the silty clay has a stiff to very stiff crust with undrained shear strengths exceeding 100 kPa. The silty clay consistency, however, reduces with depth to soft to firm to stiff, with undrained shear strengths of about 20 kPa to 70 kPa, at depths of about 3 m to 5 m. Sensitivity, the measure of peak shear strength to remoulded shear strength, ranged from about 2 to 16, with an average of about 7, indicating the clay is moderately sensitive. The undrained shear strength profile which includes shear strength data from all boreholes is shown on Figure A4. The strength profile shown on Figure A3 is based on the strength data from boreholes at the locations of the abutments and the design profile assumed is based on the actual vane test results and the SPT values.

Based on all the above, and with reference to Figure A3, it is evident that the clay is heavily overconsolidated in the upper 3 m to 4 m, becoming moderately to lightly overconsolidated with increasing depth. The preconsolidation pressure near the top of the stratum is estimated at about 400 kPa, on average. The overconsolidation ratio (OCR) is estimated as ranging from over 30 near the top to about 3 at a depth of about 3 m. Thereafter with increasing depth, the OCR decreases gradually to about 1.6 at a depth of 14 m.

A one-dimensional consolidation test was performed on a sample of the silty clay extruded from a thin walled Shelby tube, obtained from BH-21EP. The results are presented graphically and numerically

in Appendix C. The data are also summarized below in Table 1-1, along with the value ranges used in our subsequent analyses.

<b>Table 1-1. Consolidation Parameters for Silty Clay</b>		
	<b>BH-21EP, 3 m</b>	<b>Values for Analyses</b>
Compression ratio, $C_c' (= C_c/(1+e_0))$	0.08	0.08 - 0.20
Recompression ratio, $C_r' (= C_r/(1+e_0))$	0.006	0.008 - 0.02
Coefficient of consolidation (recompression), $C_{vr}$	40	25 - 60 (avg 40)
Coefficient of consolidation (virgin), $C_v$	7	5 - 12 (avg 8)
Coefficient of secondary compression, $C_{\alpha\epsilon}^*$	0.003 - 0.004	0.003 - 0.005
Notes: Coefficients of consolidation in units of $m^2/year$ * $C_{\alpha\epsilon} = \Delta H/H_0$ (= secondary compression ratio, in some literature)		

#### 1.4.5 Silty Sand and Gravel

A basal deposit of loose to dense, brown silty sand and gravel was encountered above the bedrock (or refusal) surface in most of the boreholes. Standard penetration indices (N-values) ranged from about 6 to 58. Its thickness varied from less than about 0.5 m to over 4 m.

#### 1.4.6 Bedrock

The presence of bedrock was established by retrieving "BQ" size cores in one sampled borehole at each of the six foundation element locations, for depths of between about 3.1 m and 3.4 m. Detailed descriptions of the rock are presented in Table 1-1 in Appendix B, following the borehole logs. Generally, the bedrock can be described as a pink and light grey, biotite-hornblende gneiss. The rock is strong and unweathered for the most part.

Rock core recovery was 100% for all runs and the Rock Quality Designation (RQD) values for individual core runs ranged from about 10% to 96%. The average RQD for the rock core recovered was about 70%, based on the 20 core runs. In accordance with the MTO classification system, the rock quality can be described as very poor to excellent, with an average of fair. It is noted that the RQD values are likely conservative; it is expected that higher values would be obtained using NQ core.

Table 1-2, below, lists the bedrock depths and elevations as well as those of refusal, at the locations of each of the six foundation elements. It can be seen that the bedrock and refusal depths and elevations are quite variable, even within short distances at the individual element locations. Refusal (to augering or dynamic cone penetration testing (CPT)) is inferred to be due to probable bedrock, but it is noted that refusal may be due to cobbles, boulders, or very dense soil. The bedrock depths and elevations have been positively established only at the locations where the bedrock has been cored.

<b>Table 1-2. Bedrock and Refusal Depths and Elevations at Boreholes</b>					
<b>Location</b>	<b>Borehole</b>	<b>Orig. Gr. Elev. (m)</b>	<b>Bedrock or Refusal Elev (m)</b>	<b>Overburden Thickness (m)</b>	<b>Comment RQD (by run)</b>
<b>WP1 South Abutment</b>	BH-1NF	313.8	307.46	6.34	auger refusal
	BH-2NF	314.3	310.43	3.87	B/R cored 87%, 96%
	AP-1NF	313.82	308.58	5.24	auger refusal
	AP-2NF	313.91	309.76	4.15	auger refusal
	AP-3NF	313.9	309.72	4.18	auger refusal
	AP-4NF	314.21	310.6	3.61	auger refusal
<b>WP2 Pier</b>	BH-3NF	312.54	302.42	10.12	B/R cored 78%, 90%
	BH-4NF	313.51	304.52	8.99	auger refusal
	BH-8DP	313.26	303.51	9.75	auger refusal
<b>WP3 Pier</b>	BH-5NF	307.04	300.12	6.92	B/R cored 10%, 20%, 75%
	BH-6NF	304.88	296.04	8.84	auger refusal
	C-5DF	305.9	299.42	6.48	CPT refusal
<b>WP4 Pier</b>	BH-3DF	301.11	294.34	6.77	B/R cored 98%, 100%
	BH-18DF	300.76	295.09	5.67	auger refusal
	C-3DF	300.64	294.95	5.69	CPT refusal



<b>Table 1-2. Bedrock and Refusal Depths and Elevations at Boreholes</b>					
<b>Location</b>	<b>Borehole</b>	<b>Orig. Gr. Elev. (m)</b>	<b>Bedrock or Refusal Elev (m)</b>	<b>Overburden Thickness (m)</b>	<b>Comment RQD (by run)</b>
<b>WP5 Pier</b>	BH-7NF	310.51	299.14	11.37	B/R cored 65%, 65%, 78%
	BH-8NF	312.21	298.4	13.81	auger refusal
<b>WP6 North Abutment</b>	BH-9NF	312.8	298.11	14.69	auger refusal
	BH-10NF	312.93	298.18	14.75	B/R cored 82%, 93%

## 1.5 Groundwater Conditions

Information regarding the groundwater levels at the site was obtained by measuring the water levels in the open boreholes after completion of drilling and by tactile examination of the recovered samples. The measured or inferred groundwater levels are shown on the borehole logs and the stratigraphic sections. In general, the groundwater table at the times of the field work was between about 1 m and 4 m in depth. It appears to follow the topography and this suggests that local subsurface drainage would be towards Trout Creek.

## Part 2 Engineering Discussion and Recommendations

### 2.1 Introduction

The following subsection addresses the geotechnical design considerations pertaining to the proposed five span bridge for the Northbound Lanes crossing of Trout Creek, as well as the approaches.

An original investigation was completed at this site in June and July of 1998 for an originally proposed 105 m long ( $\pm$ ) three span structure and revealed significant clay deposits on either side of Trout Creek which caused concern for the stability of the approach embankments, which were about 20 m high. Various design alternatives were considered including removal of the clay to the underlying bedrock, extensive berming of the embankments, the use of lightweight fill within the embankments, retaining walls, and a lengthening of the structure to limit the impact of the clay deposits. A subsequent cost benefit analysis of the design alternatives proposed by Trow, performed by Marshall Macklin Monaghan, indicated the preferred alternative was to lengthen the structure.

It was considered that the lengthened structure would effectively span the clay deposit, so that the approach embankments could either be located on a thinner or absent clay layer, or alternatively the lengthened structure would extend to the point where the height of the approach embankments could be reduced, and thus reduce or eliminate the complications encountered by placing high approach embankments on thick clay deposits.

Upon MTO acceptance of the lengthened structure alternative, a supplementary investigation was completed during September and November, 1998 as described in Part 1 of this report. The supplementary investigation examined the soil conditions at the revised pier and abutment locations of the new proposed five span structure, where conditions were not previously investigated as part of the original work.

The five span bridge is proposed to carry northbound Highway 11 traffic over Trout Creek and its valley. It is Trow's understanding that the bridge will be located with the central span crossing Trout Creek. The structure will include an approximately 11 metre high south abutment, 22 metre high centre span, and 5 metre high north abutment. The span lengths will be approximately 40, 54, 54, 54, 40 metres in length, creating a bridge with a length of about 242 metres.

It is understood that the project is scheduled for completion within a two year time frame. Accordingly, some of the discussions and design recommendations presented herein are based on this time frame.

## **2.2 Foundations**

In general, because of the presence of loose to compact sand and silt and relatively weak and compressible clay at the locations of the foundation elements, it is not considered feasible to support the proposed bridge foundations on spread footings constructed on the native mineral soils. For all foundation elements, driven steel H-piles are considered to be the preferred alternative. Alternate types of foundations may, however, be considered for the support of the bridge piers and abutments. The alternate types that are considered applicable to the site and proposed layout include large diameter reinforced concrete caissons and spread footings on rock or structural fill. Not all of these foundation types are applicable to all six foundation elements.

The following sections present the foundation design recommendations for the six foundation elements of the proposed bridge.

### **2.2.1 Steel H-Piles (all locations)**

All abutments and piers are recommended to be supported on steel H-piles driven to the bedrock surface, using the ULS capacities for HP310x110 and HP310x132 sections, as given in Table 2-1, below.

Current design practice requires that consideration be given to downdrag loads that may be generated as a result of soil consolidation due to fills, groundwater table fluctuations and the soil stresses induced by pile installation. We consider that, even after primary consolidation of the clay soils due to the fill placement is complete, the potential exists for downdrag loads due to events such as significant groundwater lowering and the long term process of secondary compression. At this site, secondary compression is the likely mechanism. Secondary compression of the clay foundation soils is discussed further in a subsequent section of this report. Accordingly, we have considered the effects of downdrag loads on the pile capacities given in the following sections, where applicable.

<b>Table 2-1. H-Pile Design Pile Capacities (kN)</b>						
	<b>HP 310x110</b>			<b>HP 310x132</b>		
Factored Structural Capacity (OHBDC)	3285			3890		
Factored Axial Resistance (MTO*)	2000			2300 (est)		
Pile Location ----->	South Abutment W1	Piers W2, W3, W4, W5	North Abutment W6	South Abutment W1	Piers W2, W3, W4, W5	North Abutment W6
Factored Downdrag Load	1000	-	1000	1150	-	1150
<b>Factored Axial Capacity at ULS**</b>	<b>2000</b>	<b>2000</b>	<b>2000</b>	<b>2300</b>	<b>2300</b>	<b>2300</b>
Notes: MTO* = Structural Office Policy Memo 98-01, April 15, 1998 ** Factored axial capacity at ULS is the lesser of: (a) factored structural capacity minus factored downdrag load, and (b) factored axial resistance SLS capacity not applicable to piles driven to bedrock						

The axial capacity at SLS is not applicable to steel piles driven to bedrock because very high loads, in excess of the ULS capacity, are required to produce unacceptable deformations. The pile structural capacity will govern.

The capacities given are axial capacities, for both vertical and inclined piles. No reduction is required for inclined piles. Notwithstanding the above, the horizontal and vertical loads acting on all piles must be resolved axially, such that the vector sum of these loads does not exceed the ULS capacity provided above.

For design purposes, the modulus of horizontal subgrade reaction for steel H-piles can be taken as 8,000 kN/m<sup>3</sup> for the cohesive soils (silty clays) and 40,000 kN/m<sup>3</sup> for the cohesionless soils. It is expected, however, that inclined piles will be required to accommodate the lateral loads. These can be designed using the same axial capacities given in Table 2-1.

A minimum soil embedment depth of 3 m below the pile cap is recommended. Pile caps should be provided with at least 2 m of soil cover for frost protection. Local grade raises may be required in order to provide this cover.

If the underside of the pile caps cannot be provided with a minimum of 2 m earth cover, insulation will be required. Insulation should consist of rigid board extruded polystyrene, meeting

CAN/CGSB-51.20-M87 (Type 4), such as *DOW SM™*. The insulation is recommended to be placed beneath the pile caps, prior to placement of concrete. Since the insulation will not carry any significant loads, high strength/low compressibility insulation (such as *DOW HI40™*, etc.) is not required. Products other than those made by *DOW CORNING* may be used, provided they meet the above noted specification.

The insulation thickness and lateral extension beyond the edges of the pile caps will depend on the depth of placement (i.e., underside of pile cap), in accordance with Table 2-2, below. A minimum soil cover of 300 mm is recommended over the top of the insulation.

Table 2-2. Pile Cap Insulation Dimensions		
Depth (mm)	Thickness (mm)	Lateral Extension (mm)
500	90	1500
1000	50	1000
1500	25	500

As discussed in following subsections of this report, substantial settlements of the approach fills will occur as will some associated lateral strains. Accordingly, piles should not be installed until after the majority of the fill settlement has occurred, based on monitoring during construction. This is particularly important for inclined piles, that would be subjected to bending stresses.

All piles should be driven to bedrock. Pile tip elevations can be estimated from Table 1-2 which provides the bedrock or refusal elevations encountered at the boreholes drilled at the various foundation elements. The boreholes indicate that the bedrock elevations are quite erratic and the potential for irregular steeply sloping bedrock at the foundation locations is considered to be high at most locations. As such, problems may arise during pile seating. At some locations, the piles may have a tendency to skip over the bedrock surface resulting in alignment problems and deeper penetration. In the event that this problem occurs, somewhat longer piles may be required and, in some cases, piles may have to be added or replaced.

To minimize seating difficulties, rock injector points should be used to facilitate proper seating. All piles must be fitted with reinforcing plates welded to the flanges as per OPSD 3301 to minimize pile damage. It is recommended that, during pile driving and upon initial contact with the bedrock, the

pile driving energy should be reduced and subsequently increased incrementally until the piles have been sufficiently seated.

### 2.2.2 Concrete Caissons (WP1 - South Abutment)

As an alternative foundation system, concrete caissons installed on or into the bedrock can be considered for all locations. However, they will likely only be practical for the foundation at the south abutment (WP1), where the bedrock surface is generally within about 3.5 m to 6.5 m below original grade. The load capacity will be derived by end bearing, in accordance with the values given in Table 2-3. As for steel H-piles, the effects of downdrag loads must be considered.

Table 2-3. Concrete Caisson Design Capacities	
Factored Downdrag Load (WP1 abutment)*	2800 kN/m pile diameter
Factored Axial Capacity at ULS	8000 kPa
Notes: SLS capacity not applicable to caissons on bedrock * Factored Downdrag Load to be applied to the factored dead loads	

In order to provide an adequate socket, the caisson should be installed at least one pile diameter into the bedrock, or be heavily dowelled. While these units can provide high capacities, because of the irregular and potentially steeply sloping bedrock surface expected at this site, caisson installation may prove difficult, thus expensive. This is complicated by the fact that in most cases the bedrock is overlain by silty sand and gravel that may create dewatering and stability problems during work at the base of the caissons.

### 2.2.3 Spread Footings (WP1 - South Abutment)

An alternate foundation for the south abutment is a conventional abutment on a spread footing. This alternative is not considered practical for the other locations because of the low capacities that will result from SLS considerations when constructed on soil or the need to excavate greater than 6 m for construction on bedrock. At the south abutment, a spread footing abutment can be constructed either on the bedrock or on structural fill placed over the bedrock or over the native soil.

### 2.2.3.1 Spread Footing on Bedrock

The elevations of a footing on bedrock can be estimated from Table 1-2. The factored bearing resistance at ULS for footings on unweathered bedrock is **8,000 kPa**. The bearing resistance at SLS does not apply because of the much higher pressures required to produce unacceptable deformations.

In order to evaluate the sliding resistance of spread footings on bedrock, the unfactored coefficients of friction for mass concrete on clean bedrock can be taken as 0.7. If the factored resistance against sliding failure is inadequate based on friction only, steel dowels will be required for footings on bedrock.

The ULS capacity of spread footings must be reduced for the effects of inclined loads. The reduction factors given in Table 2-4, below, can be used for footings on bedrock. Interpolation is possible. These factors must be applied to the ULS bearing resistance given previously.

<b>Table 2-4. ULS Reduction Factors for Inclined Loads on Spread Footings</b>	
<b>Ratio of Horizontal to Vertical Load</b>	<b>Footings on Rock</b>
0.1	0.86
0.2	0.76
0.3	0.66
0.4	0.58
The ULS reduction factors for inclined loads have been taken from Figure 6-8.4.2 of the OHBDC	

### 2.2.3.2 Spread Footing on Structural Fill

Spread footings can be designed for construction on structural fill. Structural fill can be constructed after removal of the overburden soils, where shallow, or it can consist of the granular (or rockfill) approach fill placed on the stripped native soils. For the abutment support, it is recommended that

the structural fill consist of OPSS Granular A, placed in small lifts and adequately compacted (100% standard Proctor). Alternatively, a relatively fine well graded rockfill, with a maximum size of 300 mm can be used. This finer graded rockfill should be placed in lifts limited to about 500 mm and adequately compacted with heavy vibratory rollers (minimum 6 passes, 10 tonnes).

At the south abutment, a spread footing abutment foundation, if considered, should be designed for construction in the approach fill, with a depth of about 2 m below the slope face (to provide adequate frost cover). This would place the base of the footing at a distance of about 6 m above original ground, at an elevation of about 320 m.

If the native soils are left in place, the structural fill supporting the foundation should have a thickness at least equal to the width of the footing. In addition, the structural fill should be constructed to occupy a zone, down and out from the footing edges at a slope of no steeper than 1H:2V, in order to accommodate the footing stresses.

For a foundation constructed on structural fill, as discussed above, the ULS bearing resistance given in Table 2-5, below, can be used.

<b>Table 2-5. Spread Footing ULS and SLS Bearing Resistance</b>	
Factored Bearing Resistance at ULS on Unweathered Bedrock <sup>1</sup>	8000 kPa
Factored Bearing Resistance at ULS on Structural Fill <sup>2</sup>	1000 kPa
Bearing Resistance at SLS - for 25 mm settlement - Fill on native soil <sup>3</sup>	See Figure A5
Bearing Resistance at SLS - for 25 mm settlement - Fill on bedrock <sup>4</sup>	> 1000 kPa
Notes: 1. SLS resistance not applicable to footings on bedrock 2. Thickness of fill greater than footing width. 3. 25 mm settlement is combined immediate granular settlement and primary consolidation settlement of clay soil. Structural fill placed on native soils after removal of organics/topsoil, and bottom of footing about 6 m above original ground. 4. Structural fill placed on bedrock after removal of all native soil, and bottom of footing about 6 m above original ground.	

The bearing resistance at SLS, however, will depend on whether the fill is placed on the bedrock after the removal of the native soil, or if it is placed over the native soil after stripping. If the native soil



is removed and replaced with granular structural fill, the SLS bearing resistance for 25 mm immediate elastic settlement is greater than at ULS. Accordingly, the ULS resistance governs the design.

If the native soil is left in place, the resulting settlements will be due to a combination immediate elastic settlement of the fill as well as consolidation settlement of the underlying cohesive soil. Figure A5 shows the footing bearing resistance at SLS for various footing sizes, based on 25 mm combined settlement.

Because of the consolidation settlement that will occur due to the loading by the embankment itself, footing construction should be delayed until the majority of settlement is complete, as discussed in a subsequent section of this report.

It is important to note that the stratigraphic cross section at the south abutment (WP1) shows silty clay as not having been encountered in BH-2NF or AP-4NF (the east end of the abutment). Accordingly, differential consolidation settlement of a large spread footing constructed over this zone will likely result in rotation or tilting. The magnitudes will likely be in the same order as the total deflections on which the SLS bearing resistance curves shown on Figure A5 are based.

Secondary compression of the silty clay soils will also occur, primarily due to the embankment fill loading, as discussed later in Section 2.5.4. Up to about 20 mm of associated settlement may occur within 10 years following construction, and this magnitude may be differential as discussed in the previous paragraph. Additional minor related settlements will occur beyond the 10 year period.

For the determination of the sliding resistance of spread footings, the unfactored coefficient of friction for mass concrete on granular structural fill can be taken as 0.6. If the factored resistance against sliding failure is inadequate based on friction only, a soil key can be considered for footings on structural fill, making use of the passive soil resistance. Passive earth pressure coefficients are provided in Section 2.3.

The ULS resistance of spread footings must be reduced for the effects of inclined loads. For footings on granular structural fill, Figure 6-8.4.2 of the OHBDC may be used for the applicable footing depth to effective width ratio. These factors must be applied to the ULS resistance given previously in Table 2-5.

## 2.3 Backfill

Backfill for abutments or retaining walls should consist of free draining granular materials such as Granular A, Granular B, or rock fill. Computation of earth pressures shall be in accordance with Section 6.7.4 of the Ontario Highway Bridge Design Code. Unfactored properties for backfill materials are provided in the following Table 2-6.

<b>Table 2-6. Fill Types and Unfactored Geotechnical Properties</b>					
<b>Material</b>	<b>Friction Angle, <math>\phi'</math></b>	<b><math>\gamma</math> (kN/m<sup>3</sup>)</b>	<b><math>K_A</math></b>	<b><math>K_p</math></b>	<b><math>K_0</math></b>
Granular A	35 degrees	22	0.27	3.7	0.43
Granular B	30 degrees	21	0.33	3	0.5
Rock Fill	42 degrees	20	0.2	5	0.33
Note: Values given for $K_A$ and $K_p$ are for horizontal backfill, and will vary for other configurations. $K_A$ is the earth pressure coefficient corresponding to the active state. $K_p$ is the earth pressure coefficient corresponding to the passive state. $K_0$ is the earth pressure coefficient at rest.					

The mobilization of full active or passive resistance requires a measurable and perhaps significant wall movement or rotation, in general accordance with that indicated in Figure C6-7.1 of the OHBDC. Therefore, unless the structural element can tolerate these deflections, the at rest earth pressure should be used in design. Alternatively, partially mobilized active and passive earth pressure coefficients may be used, in accordance with the deflections given in the Figure.

The effects of compaction surcharge should be taken into account in the calculation of active and at rest earth pressures. In accordance with Section 6-7.4.3 of the OHBDC, the lateral pressure due to compaction should be taken as at least 16 kPa at the surface, and its magnitude should be assumed to diminish linearly with depth to zero at the depth where the active (or at rest) pressure is equal to 16 kPa. This pressure distribution should be added to the calculated active (or at rest) pressure. Notwithstanding, lighter compaction equipment and smaller lifts should be used adjacent to walls to prevent overstressing.

If rock fill is used as backfill behind abutments, the particle size should be limited to no greater than 300 mm and the backfill must be placed carefully in a manner that does not cause damage to the abutments or other structural components of the bridge.

Weep holes should be provided in the abutments and a filter wrapped, 150 mm diameter perforated PVC drainage pipe should be installed behind the abutments, at or slightly above the base.

## **2.4 Excavations and Dewatering**

All work associated with design and construction relative to excavations shall be in accordance with Part III of Ont. Reg. 213/91 of the Occupational Health and Safety Act. Where the width of the base of the excavation is less than twice its depth, conformance with this regulation is required.

The existing organic zone (topsoil) with a depth ranging to about 200 mm will have to be removed from beneath the approach fill footprints. For pile caps at the pier locations, excavations of at least 2 m depth will likely be required, in order to provide the recommended frost cover. Since the ground surface is uneven at the pier locations, deeper excavations will be required to place the pile cap at a common elevation, unless the finished site grades are raised with fill or the caps are provided with insulation. Based on the borehole information, excavations would generally proceed through the loose to compact upper sand and into the stiff to firm silty clay at most locations.

At the locations of Piers WP2, WP3 and WP5, excavations should be relatively straight forward, since they will likely terminate above the prevailing groundwater level. The upper sand and silty clay is considered a Type 3 soil and excavations should be cut back to at least 1H:1V. If minor groundwater seepage occurs and loosens/softens the soil, flatter slopes will be required. Dewatering of the excavations should be possible by pumping from sumps within the excavations.

At the location of Pier WP4, on the north side of Trout Creek, the excavation will likely be carried out entirely within the loose to compact silty sand and would terminate close to or below the prevailing water level of Trout Creek. These soils would then be classified as Type 4 soils and excavations should be sloped back at 3H:1V. Pumping from sumps within the excavation should suffice here as well, in order to maintain a safe and workable area, although more aggressive effort will likely be required. In order to stabilize the base of the excavation if it becomes loosened due to groundwater infiltration, a 300 mm layer of crushed clear stone may be required to improve working conditions.

If the native soils are to be removed and replaced with structural fill at the south abutment, or to place foundations on the bedrock, excavations as deep as about 6.5 m will be required, based on the results of the investigation. The soils to be excavated will consist of sand, silty clay and silty sand and gravel. The water table would be encountered well above the 6.5 m depth. Accordingly, excavations in this area should be sloped back at 3H:1V, or flatter. Aggressive pumping from sumps will be required. Alternatively, sheeted and braced excavations could be considered, but this may prove problematic because of the variable bedrock surface.

Excavations carried out within granular structural fill in the approaches can likely be completed using a 1H:1V cut since it will be above the water table.

It is recommended that a non-standard special provision (NSSP) for dewatering be provided in the contract documents.

## **2.5 Bridge Approach Fills**

The construction of the bridge approaches will require embankment fills of up to about 11 m height at the south abutment and about 5 m height at the north abutment over areas with varying thicknesses of predominantly firm, compressible silty clay soils. This creates two principal design and construction considerations: embankment stability and consolidation settlement. These two issues are discussed in the following sections.

In all of the following discussions, it is assumed that all organic material (topsoil) is removed from beneath the embankments and the embankments are constructed on the native mineral soils. Fill heights should be measured from the top of the native mineral soil.

### **2.5.1 Embankment Stability**

Highway embankments can be constructed using structural fill of various acceptable soil materials. Typically, however, in this part of the province they are constructed using rockfill as the principal component. A typical MTO section will consist of a minimum (steepest) slope of 1.25H to 1V to a maximum height of 6 m. For embankment sections greater than 6 m height, a 2 m wide horizontal bench is required before the next 6 m, 1.25H to 1V lift is constructed. A 14 m crest width has also been assumed, based on the drawings provided.

Slope stability analyses have been performed using SLOPE/W (ver. 3.x), based on Bishop's Method using total stress parameters. This analysis would apply to rapid construction (short term stability) and factors of safety can be expected to increase with time. The undrained shear strength profile shown on Figure A3 was used to provide the shear strength parameters for the clay soils. Table 2-7, below, lists the parameters used.

<b>Table 2-7. Geotechnical Parameters for Slope Stability Analyses</b>			
	$\gamma_{\text{total}}$ (kN/m <sup>3</sup> )	$c_u$ (kPa)	$\phi'$
Rockfill	20	0	42°
Sand	20	0	32°
Silty clay	19.5	variable (see Fig. A3)	0
Sand and Gravel	21.5	0	35°
Notes: Embankment crest width 14 m.			

Many of the SLOPE/W graphical printouts, for the various analyses performed and discussed below, are included in Appendix D.

### 2.5.1.1 South Approach Stability

The results of the total stress analyses performed on the embankment cross-sections, for heights of 10 m to 13 m, are shown in Table 2-8, below. The short term factors of safety for the embankments assessed are marginally below the recommended value of 1.3. It is our opinion, however, that the factors of safety are likely greater than calculated, since some excess pore water pressure dissipation will occur during the assumed one month construction period. At this location, the silty clay layer is relatively thin (3 m to 4 m) and the results of the consolidation analyses, discussed later in Section 2.5.5.2, support this.

Notwithstanding, we have performed calculations for embankments with benches wider than the standard 2 m width at 6 m height. These required widths to provide a calculated factor of safety of at least 1.3 are shown in Table 2-8, also. Accordingly, it is considered prudent to provide the indicated berm widths to provide a calculated factor of safety of 1.3.

Since the recommended short term factors of safety are considered adequate, with only minor modifications to the geometry, effective stress analyses are not required and have not been performed.

**Table 2-8. Safety Factors for Total Stress Stability Analyses - South Approach**

Embankment Height	Factor of Safety*	Bench Width**
10 m	1.28	+ 0.5 m (2.5 m)
11 m	1.23	+ 2 m (4 m)
12 m	1.18	+ 3 m (5 m)
13 m	1.21	+ 4 m (6 m)

Notes: \* Factor of safety for embankment with standard 2 m wide bench at 6 m height  
\*\* Bench width required to provide safety factor >1.3. First number is width to add to standard 2 m wide lower bench.

An analysis was also performed on the longitudinal design section (front slope) as shown on Figure A2b and Drawing No. 2. The calculated factor of safety for this slope is 1.58, which is considered adequate.

### 2.5.1.2 North Approach Stability

The results of the total stress analyses performed on the embankment cross-sections, for heights of 4 m to 7 m, resulted in safety factors ranging from about 3.6 to 2.2, which are considered more than adequate.

An analysis was also performed on the longitudinal design section (front slope) as shown on Figure A2b and Drawing No. 2. The calculated factor of safety for this slope is 3.0, which is also considered adequate.

## 2.5.2 Consolidation Settlement of Embankments

### 2.5.2.1 Magnitudes of Consolidation Settlement

Consolidation settlement calculations have been performed using the effective stress profiles shown on Figure A3 and compression ratios ( $C_c' = C_c / [1 + e_0]$ ) ranging from 0.08 to 0.20. The values used were established from the consolidation test data, previous experience at the north and south interchanges of this project, as well as from geotechnical literature. Recompression indices ( $C_r' = C_r / [1 + e_0]$ ) ranging from about 0.008 to 0.02 were used. The stress increases in the foundation soils due to the embankment loadings have been calculated based on the Boussinesq elastic solutions. Recompression and virgin consolidation has been calculated based on the assumed existing overburden pressure and preconsolidation stress profiles shown on Figure A3.

#### South Approach

The results of the calculations for embankment centerline and crest settlement using rockfill to construct the embankments are provided in Table 2-9, below and are shown graphically on Figure A6. The thickness of the compressible silty clay soil ranges up to about 4 m. Because of the unevenness of the original ground surface at the approaches, the height of fill may vary slightly. Accordingly, the analyses have been performed taking this into account by using a varying fill height.

Table 2-9. Estimated Embankment Consolidation Settlement - South Approach		
Embankment Ht	Centerline Settlement (mm)	Crest Settlement (mm)
10 m	170	155
11 m	185	175
12 m	200	190
13 m	215	205
Notes: Embankment crest width 16 m, average side slopes = 1.4H:1V Values rounded to nearest 5 mm		

In addition, the loadings imposed by the fill will exceed the indicated preconsolidation pressure, such that the silty clay soils will be normally consolidated following the consolidation process. This will

result in greater settlements due to future additional loadings, such as footings as discussed in a previous section, or due to grade changes, for example. For these reasons, it may be prudent to overbuild the embankment by 1 or 2 m, allow consolidation to take place and subsequently remove the excess fill height (*i.e.* preloading). This will result in a slight overconsolidation of the foundation silty clay soils.

It is noted that, the compressible silty clay was not encountered everywhere beneath the embankment, as was discussed in Section 2.2.3 of this report. Accordingly, there may be greater differential settlement between crest and centerline, as well as along the length of the approach. The differential settlement may approach the total settlements indicated in the Table above.

Notwithstanding the potential for relatively large differential settlement, the time to facilitate this settlement should be relatively rapid (within about nine months), as discussed in Section 2.5.2.2, below. Therefore, minor regrading prior to providing the final granulars and pavement should be possible within the two year construction period that has been inferred.

### North Approach

The results of the calculations for embankment centerline and crest settlement using rockfill to construct the embankments are provided in Table 2-10, below and are shown graphically in the top panel of Figure A7. The thickness of the compressible silty clay soil ranges up to about 12 m. Because of the unevenness of the original ground surface at the approaches, the height of fill may vary slightly. Accordingly, the analyses have been performed taking this into account by using a varying fill height.

Table 2-10. Estimated Embankment Consolidation Settlement - North Approach		
Embankment Ht	Centerline Settlement (mm)	Crest Settlement (mm)
4 m	30	25
5 m	35	30
6 m	40	35
7 m	70	45
Notes: Embankment crest width 16 m, average side slopes = 1.4H:1V Values rounded to nearest 5 mm		



The loadings imposed by the fill will approach, and may exceed, the indicated preconsolidation pressure, such that the silty clay soils will be normally consolidated following the consolidation process due to the fill. Examination of the top panel of Figure A7 indicates that the settlement is expected to increase significantly as the embankment height approaches 6 m, where the settlement curve steepens. This is due to the preconsolidation pressure of the foundation soil being exceeded.

This will result in greater settlements due to any future additional loadings, such as grade changes, for example. Accordingly, it may be prudent to overbuild the embankment by 1 or 2 m, allow consolidation to take place and subsequently remove the excess fill height (*i.e.* preloading). The preload should be left in place for about a year, unless monitoring indicates that it can be removed sooner. This will result in a slight overconsolidation of the foundation silty clay soils.

If consideration is to be given to the use of light weight fill, for preliminary design purposes, the total consolidation settlements can be reduced **approximately** by proportion of rockfill replaced by lightweight polystyrene (eg. 30% replacement of fill height with polystyrene will result in about 70% of the indicated settlement).

#### 2.5.2.2 Time Rate of Consolidation Settlement

The time rate of consolidation settlement has been calculated for the various cases of embankment height for vertical drainage only. It is also assumed for the purposes of calculation, all embankments are constructed to full height in about a one month construction period. Coefficients of consolidation (virgin),  $C_v$ , of 8 m<sup>2</sup>/year, and (recompression),  $C_{vr}$ , of 40 m<sup>2</sup>/year, have been used in the analyses, based on the results of the consolidation tests and the geotechnical literature.

The bottom panels of Figures A6 and A7 show the calculated consolidation rate for the various fill heights at the south and north approaches, respectively.

At the south approach, virtually all of the primary consolidation settlement should be complete within about 9 months. At the north approach, because of the thicker silty clay deposit, the time rate of consolidation will be slower, and should be complete anywhere between about 15 months to over 24 months, depending on the fill height.

#### 2.5.3 Secondary Compression of Clays

Calculations have been performed to estimate the magnitudes of secondary compression of the foundation clays, following the completion of primary consolidation. For purposes of analysis and

discussion, the primary consolidation is assumed to be essentially complete within one to two years from the start of construction of the embankments. The calculations are based on use of a coefficient of secondary compression,  $C_{\alpha}$ , of 0.004, based on the results of the consolidation tests, previous experience, and the geotechnical literature.

At the south approach, calculations indicate that 15 mm to 20 mm may occur in the first 10 years, while at the north approach, the secondary settlement may be about 30 mm to 50 mm in the first 10 years.

### **2.5.5 Rockfill and Rockfill Settlement**

Rockfill used in the embankments and approaches should consist of hard, durable, angular quarried rock. Rockfill should be tested for acid drainage potential and checked against the appropriate regulatory requirements for acceptability.

For the bridge abutments that are to be supported by driven steel H-piles, as discussed in a previous section, these will have to be installed after all the embankment fill has been placed. The driving of steel piles through rockfill will be difficult, if not impossible, unless care is taken to ensure that the rockfill meets a tight size range, generally smaller than 75 mm. For the majority of the embankments and approach fills, however, larger rockfill can be used.

The use of large rockfill, loosely placed or end dumped and compacted by conventional construction traffic only may result in long term settlements of up to about 2% of the fill height. Accordingly, a 10 m high rockfill may settle up to 200 mm following completion of construction. This is primarily due to the rockfill "adjusting" under highway service conditions.

In order to minimize the long term settlements associated with the rockfill itself, consideration can be given to specifying the use of a finer graded rockfill, with a maximum size of 400 mm. This finer graded rockfill should be placed in lifts limited to about 800 mm and compacted with heavy vibratory rollers. While the production of the finer graded rockfill and its placement, as described above, carries with it a cost premium, the long term performance of the finished highway pavement will be improved.

It is recommended that an NSSP for rockfill material and placement requirements be included in the contract documents.

## 2.6 Instrumentation and Construction Monitoring

Construction of embankments should be monitored during construction using a properly designed and effective system of instrumentation, consisting primarily of settlement points. Lateral slope movements should also be monitored both during and following construction of the embankments. This will provide indications of the rate of settlement, such that construction timing of the foundations can be modified, if required.

## 2.7 Closing Comments

The recommendations made in this report are in accordance with our present understanding of the project and are provided solely for the use of Marshall Macklin Monaghan and its design team for the design of the bridge foundations and approach fills for the northbound lanes, five span bridge to be constructed over Trout Creek, as part of the Highway 11 Trout Creek By-Pass Project. We request that we be retained to review the design and our recommendations as the design proceeds, to ensure that the final design is in general agreement with our recommendations and that our recommendations have been interpreted as intended.

A subsurface investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those reported at the test locations, we require that we be notified immediately in order to allow reassessment of our recommendations. It may then be necessary to carry out additional field work and analyses.

Contractors bidding on or undertaking the works should, relative to the subsurface conditions, decide on their own investigations, if deemed necessary, as well as their own interpretations of the factual results provided herein, so they may draw their own conclusions as to how the subsurface conditions may affect them.

The information presented in this report is based on a limited investigation designed to provide information to support an overall assessment of the current geotechnical conditions at the site of the proposed northbound lanes bridge over Trout Creek. The conclusions presented in this report reflect site conditions existing at the time of the investigation. It is noted that the soil boundaries indicated on the logs are inferred from discontinuous sampling and observations during drilling. These boundaries are intended to reflect transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change.

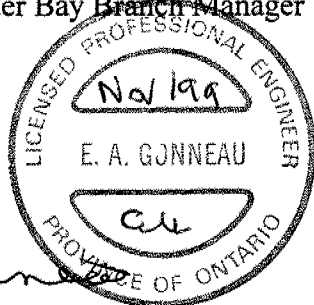

This report has been prepared by Mr. D.N. Georgiou, P.Eng. and was reviewed by Messrs. S.E. Gonsalves, P.Eng., and E. Gonneau, P.Eng. The field investigation was performed by Messrs. I. Dumpis, C.E.T., R. Moore, C.E.T., and S. McAuliffe, and was supervised by Mr. E.A. Gonneau, P.Eng.

We trust this report is satisfactory for your purposes. Should you have any questions, please do not hesitate to contact us.

All the foregoing and attachments respectfully submitted,  
**Trow Consulting Engineers Ltd.**



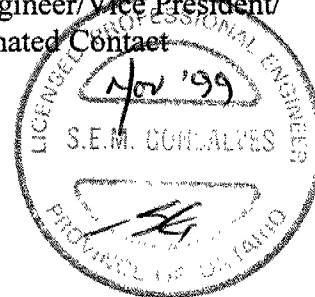

Demetri N. Georgiou, M.A.Sc., P.Eng.  
Principal Engineer/Thunder Bay Branch Manager

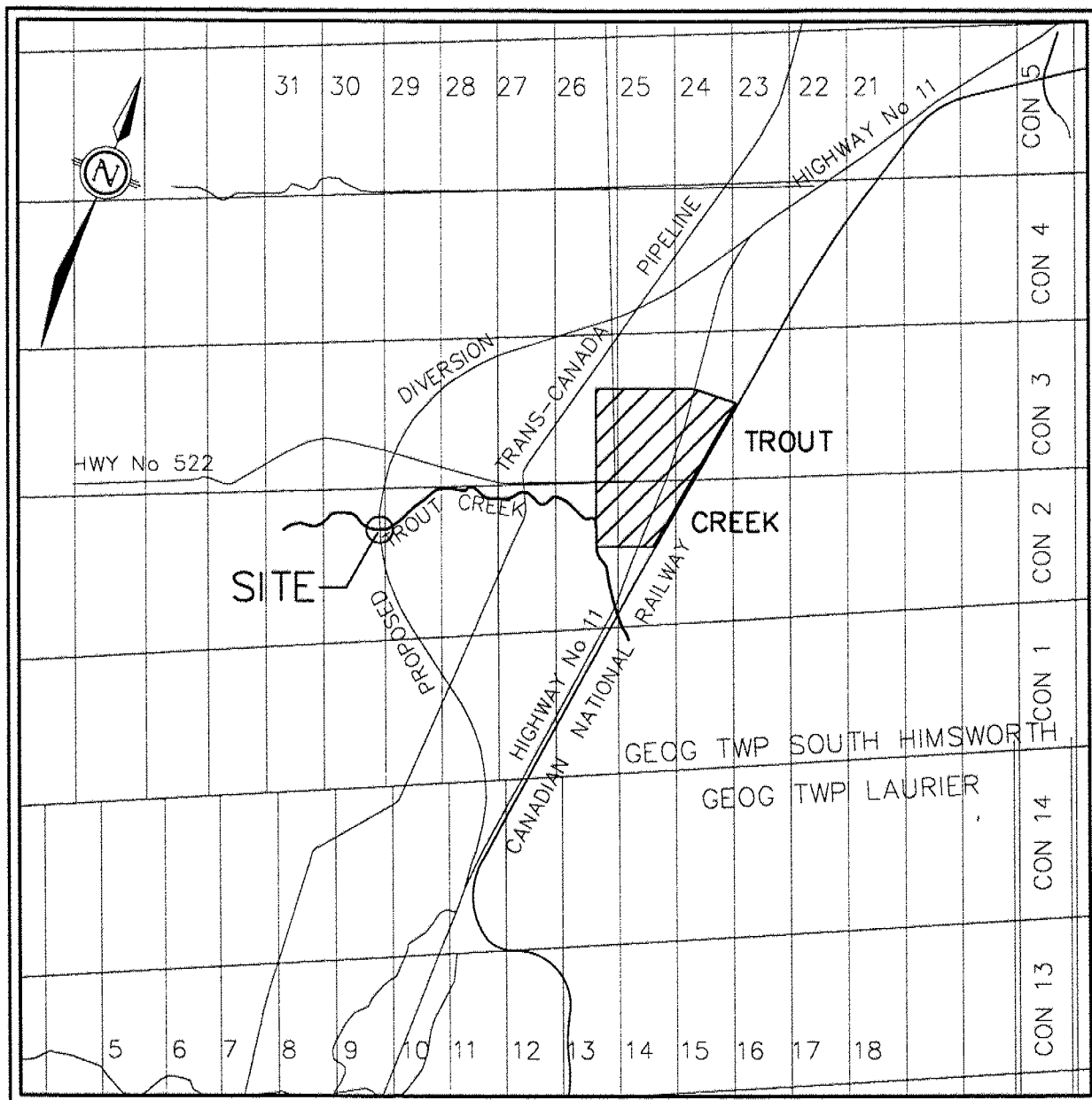
Eric A. Gonneau, P.Eng.  
Project Manager



S.E. Gonsalves, M.Eng., P.Eng.  
Principal Engineer/Vice President/  
MTO Designated Contact



A



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IN METRES



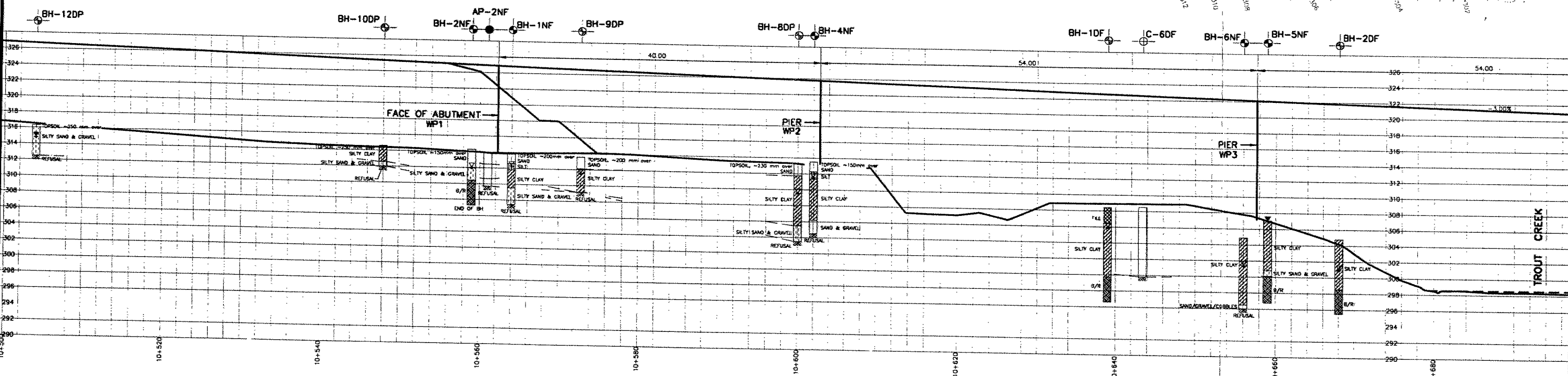
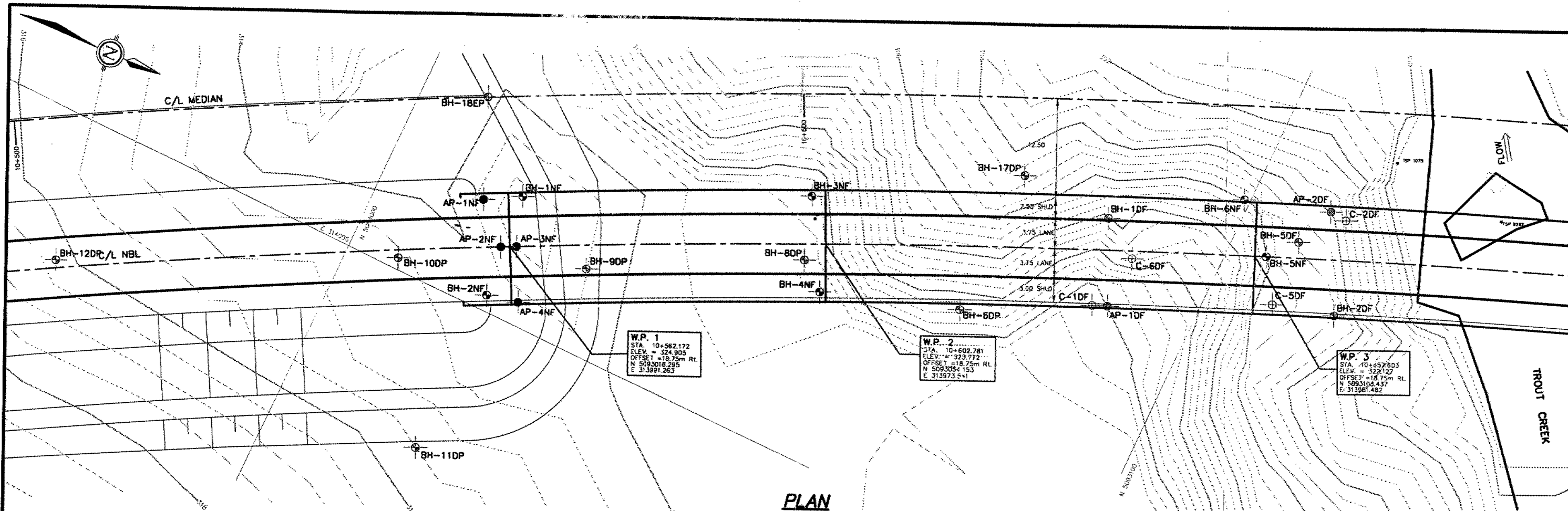
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Thunder Bay, Ontario

FIGURE  
A1

### KEY PLAN

Trout Creek By Pass  
Trout Creek Bridge—Northbound Lanes

PROJECT NO.:	F-98179-B/G
SCALE:	1:50000
DRAWN BY:	DT
CHECKED BY:	DG
DATE:	MARCH 12, 1999



**LEGEND:**

- BOREHOLE (NOV/98)
- AUGER PROBE (NOV/98)
- PREVIOUS BOREHOLE (JUN/98)
- PREVIOUS AUGER PROBE (JUN/98)
- ⊕ DYNAMIC CONE PENETRATION TEST (JUN/98)
- ▽ ESTIMATED GROUND WATER LEVEL
- ▽ MEASURED GROUND WATER LEVEL

**NOTE:**

The stratigraphic boundaries and soil types have been established at Test Hole locations. Between Test Holes they are assumed and may be subject to error.



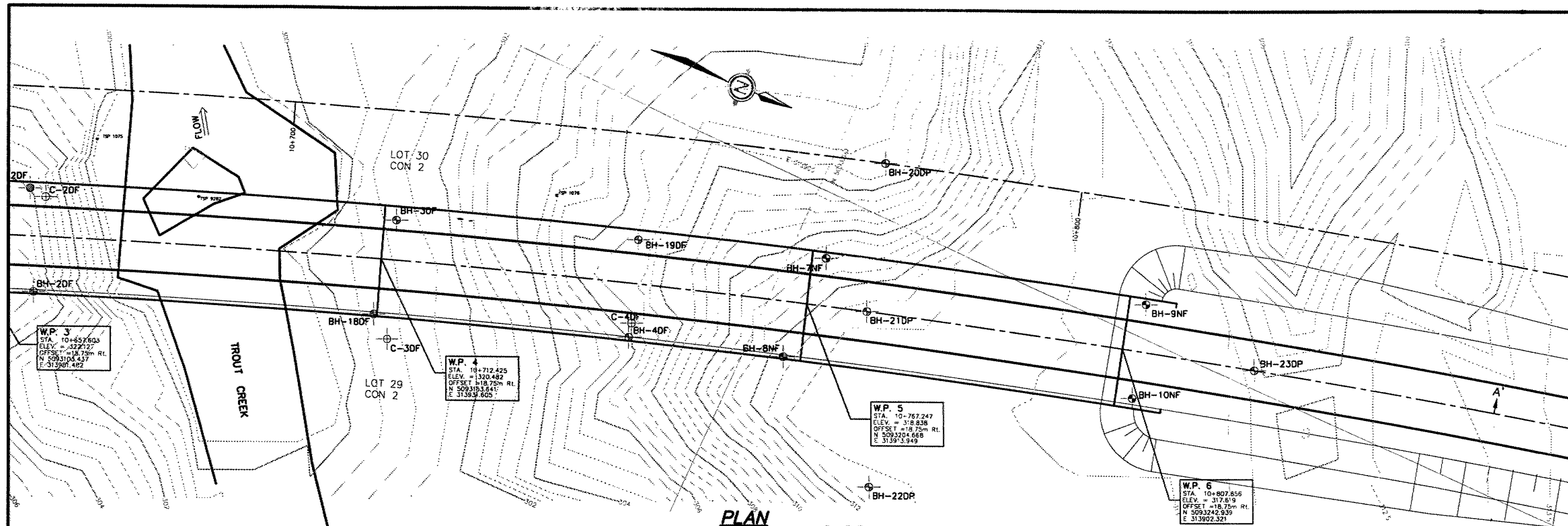
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FIGURE  
A2a

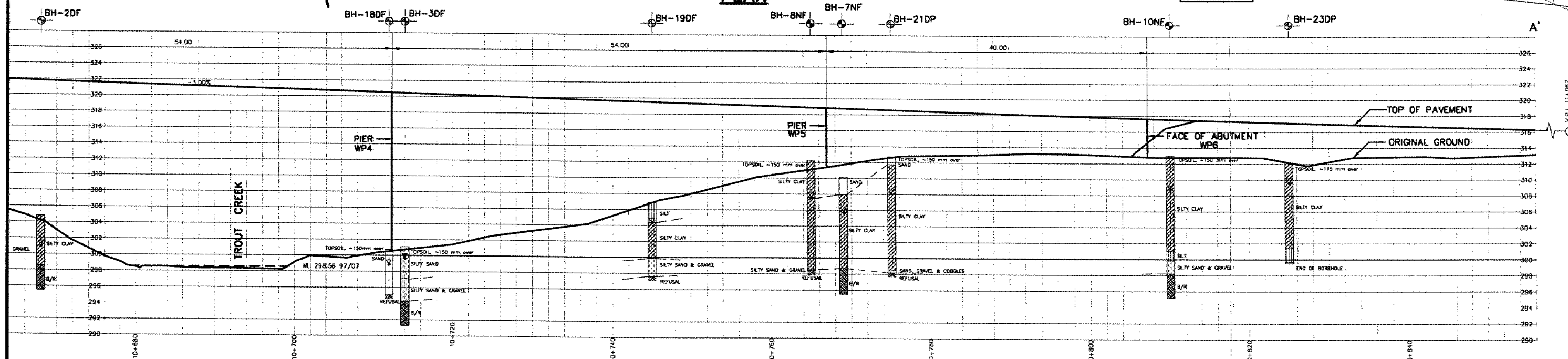
**BOREHOLE LOCATION PLAN  
& PROFILE (SOUTH)**

Trout Creek By Pass  
Trout Creek Bridge-Northbound Lanes

PROJECT NO.:	F-98179-B/G
SCALE:	1:500
DRAWN BY:	DT
CHECKED BY:	DG
DATE:	NOV. 25, 1999



PLAN



PROFILE

**LEGEND:**

- BOREHOLE (NOV/98)
- AUGER PROBE (NOV/98)
- PREVIOUS BOREHOLE (JUN/98)
- PREVIOUS AUGER PROBE (JUN/98)
- ⊕ DYNAMIC CONE PENETRATION TEST (JUN/98)
- ▽ ESTIMATED GROUND WATER LEVEL
- ▽ MEASURED GROUND WATER LEVEL

5 0 5 10  
IN METRES

**NOTE:**

The stratigraphic boundaries and soil types have been established at Test Hole locations. Between Test Holes they are assumed and may be subject to error.



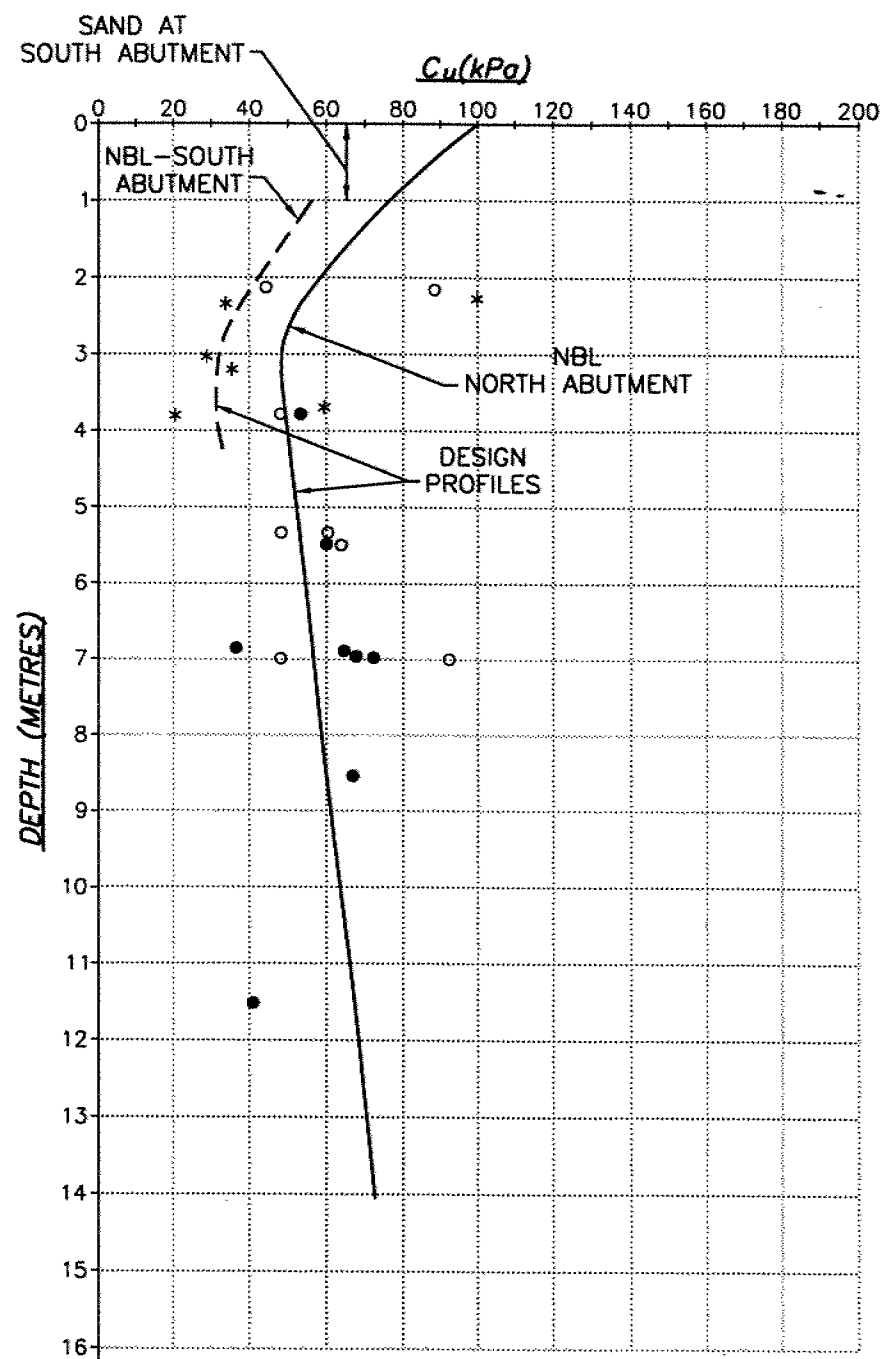
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FIGURE  
A2b

**BOREHOLE LOCATION PLAN  
& PROFILE (NORTH)**  
Trout Creek By Pass  
Trout Creek Bridge-Northbound Lanes

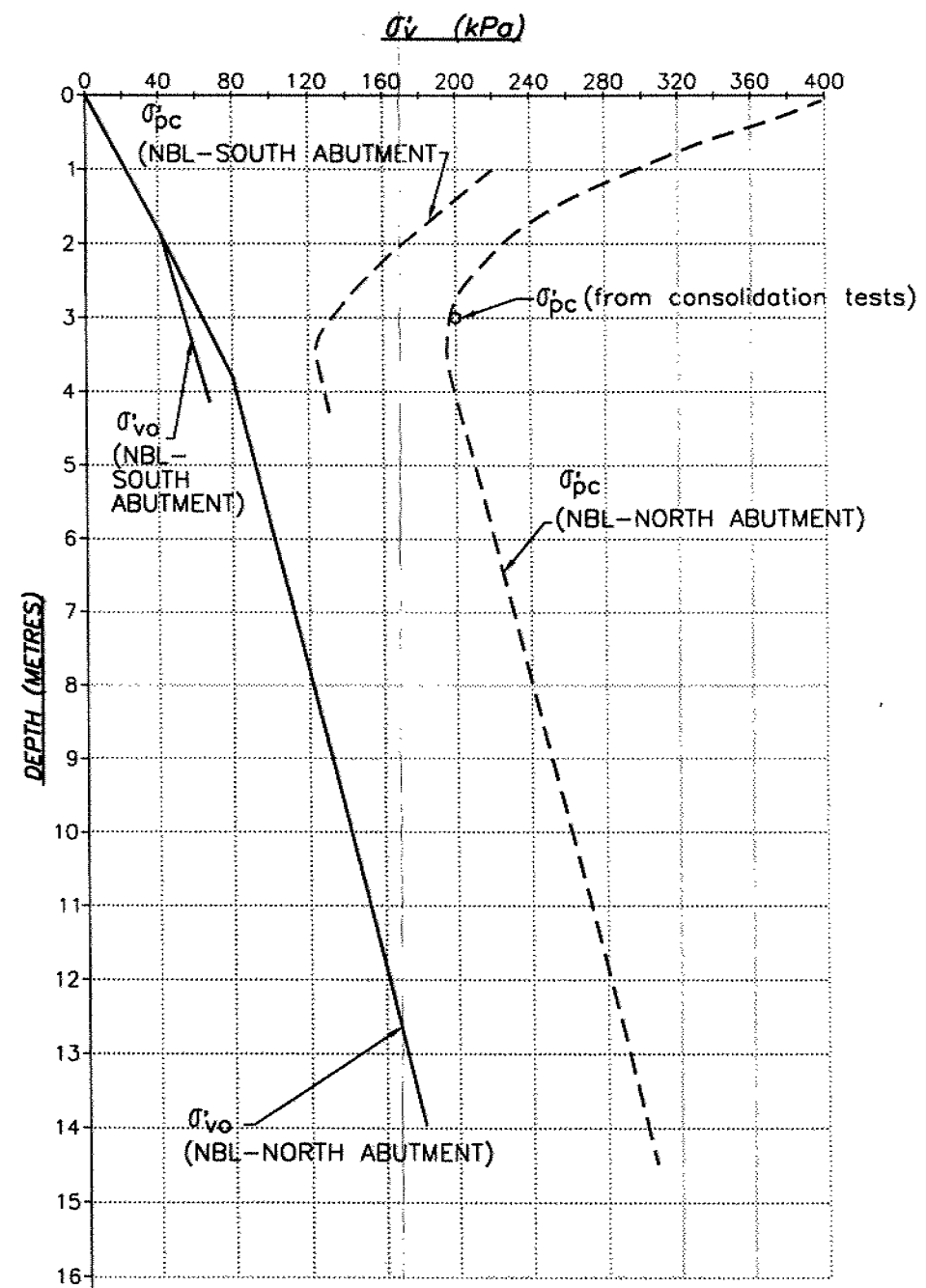
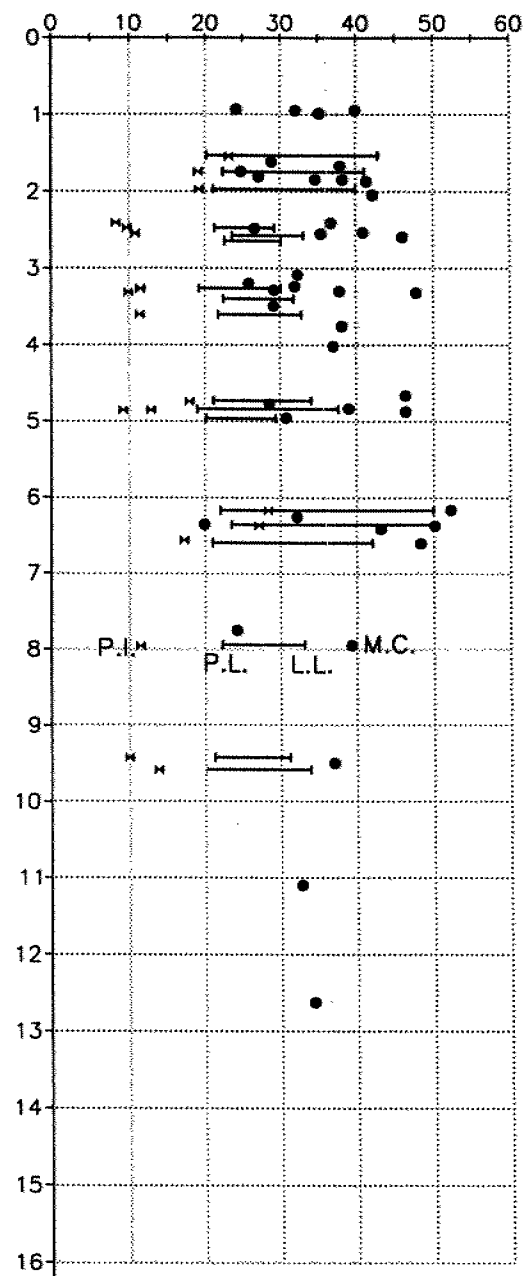
PROJECT NO.: F-98179-B/G  
SCALE: 1:500  
DRAWN BY: DT  
CHECKED BY: DG  
DATE: NOV. 25, 1999





\*NBL - SOUTH ABUTMENT  
 •NBL - NORTH ABUTMENT  
 ○SBL - NORTH ABUTMENT

### MOISTURE CONTENT & ATTERBERG LIMITS

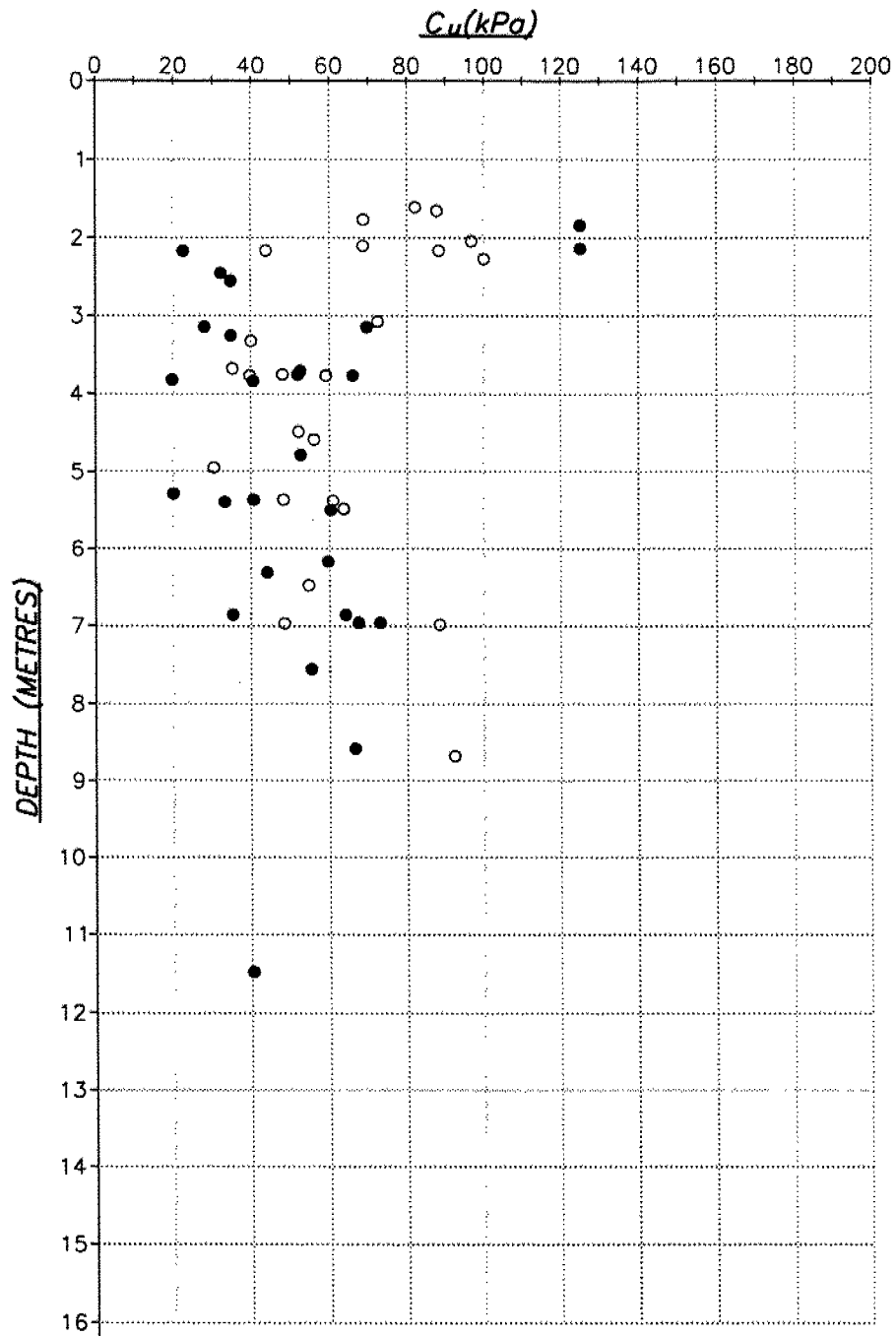


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FIGURE  
 A3

UNDRAINED SHEAR STRENGTH,  
 ATTERBERG LIMITS &  
 EFFECTIVE STRESS PROFILES  
 Trout Creek By Pass  
 Trout Creek Bridge-Northbound Lanes

PROJECT NO.: F-98179-B/G  
 SCALE: AS SHOWN  
 DRAWN BY: DT  
 CHECKED BY: DG  
 DATE: MARCH 12, 1999



LEGEND:

- SHEAR STRENGTHS (NBL)
- SHEAR STRENGTHS (SBL)



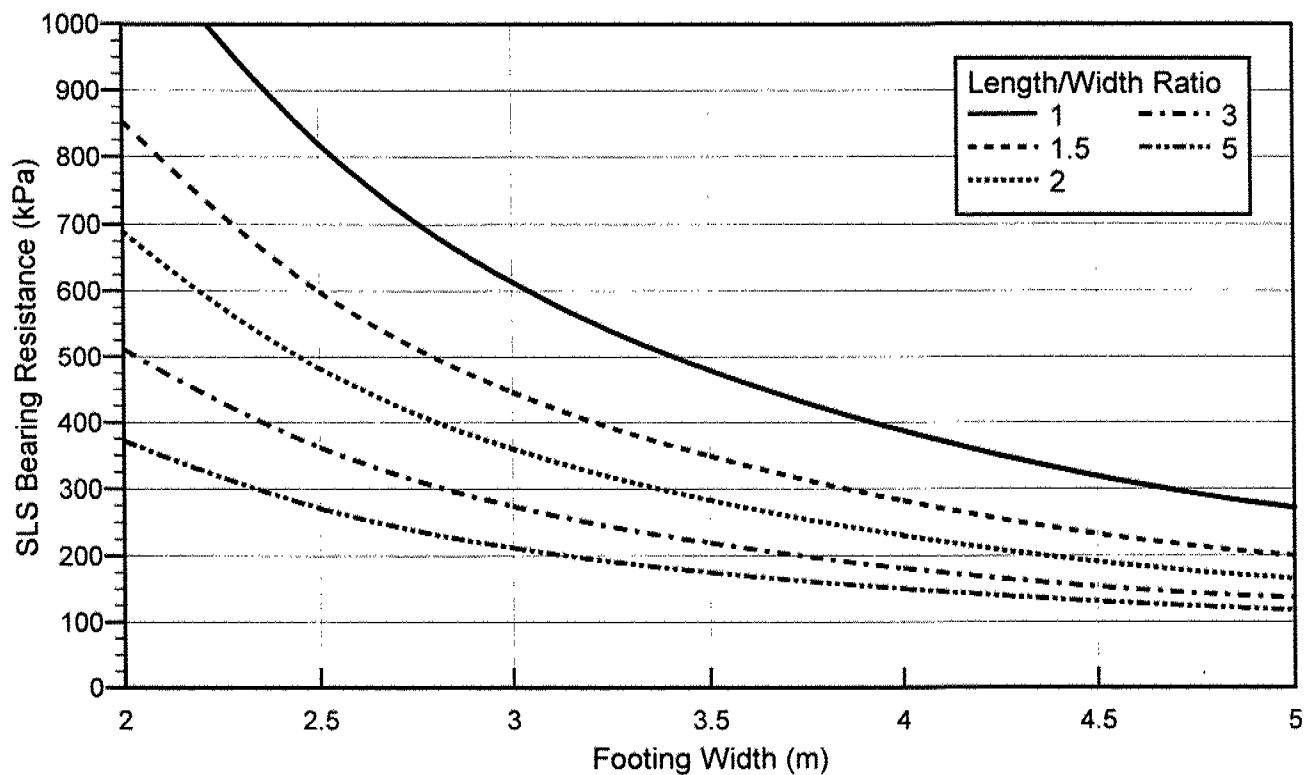
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FIGURE  
A4

UNDRAINED SHEAR STRENGTH  
ALL BOREHOLES

Trout Creek By Pass  
Trout Creek Bridge-Northbound Lanes

PROJECT NO.:	F-98179-B/G
SCALE:	AS SHOWN
DRAWN BY:	DT
CHECKED BY:	DG
DATE:	MARCH 12, 1999



Combined Immediate and  
Consolidation Settlement = 25 mm  
Bottom of footing at 6 m above O/G



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Marshall Macklin Monaghan

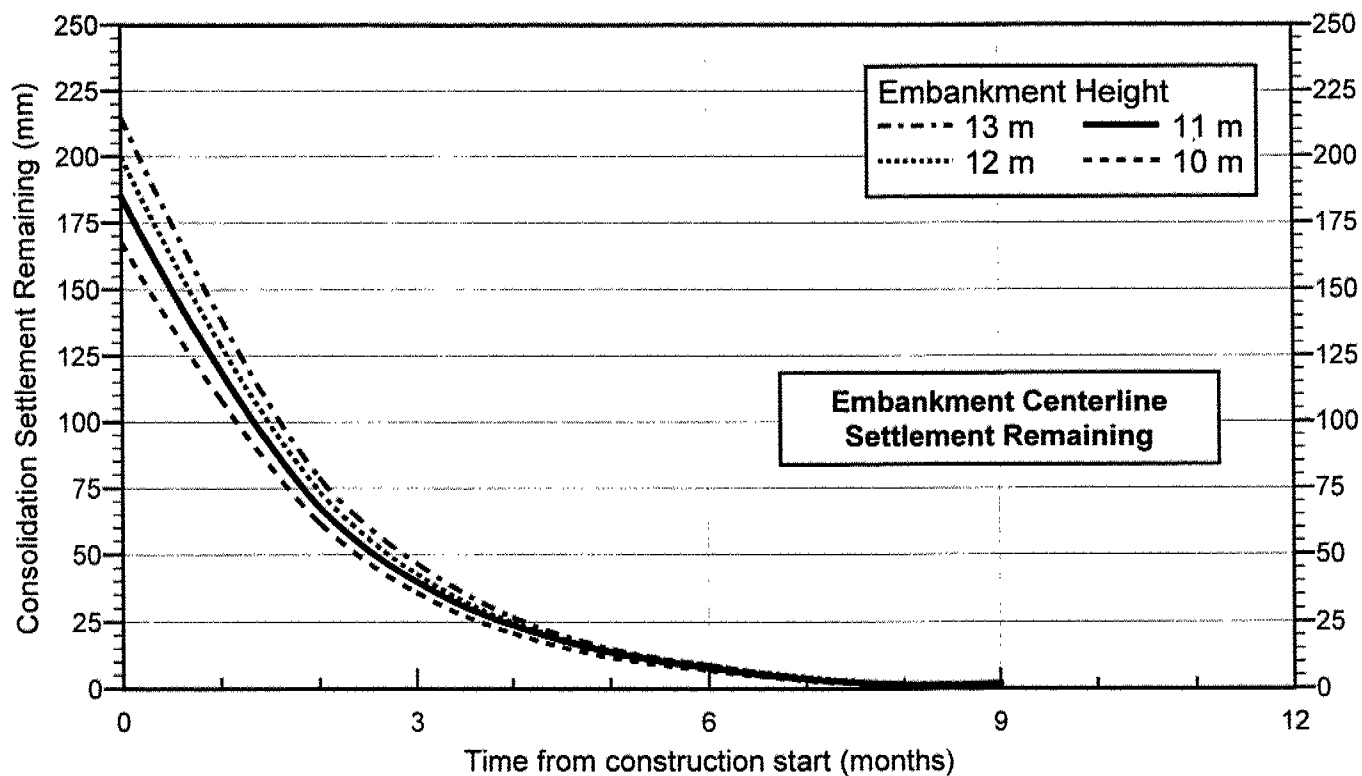
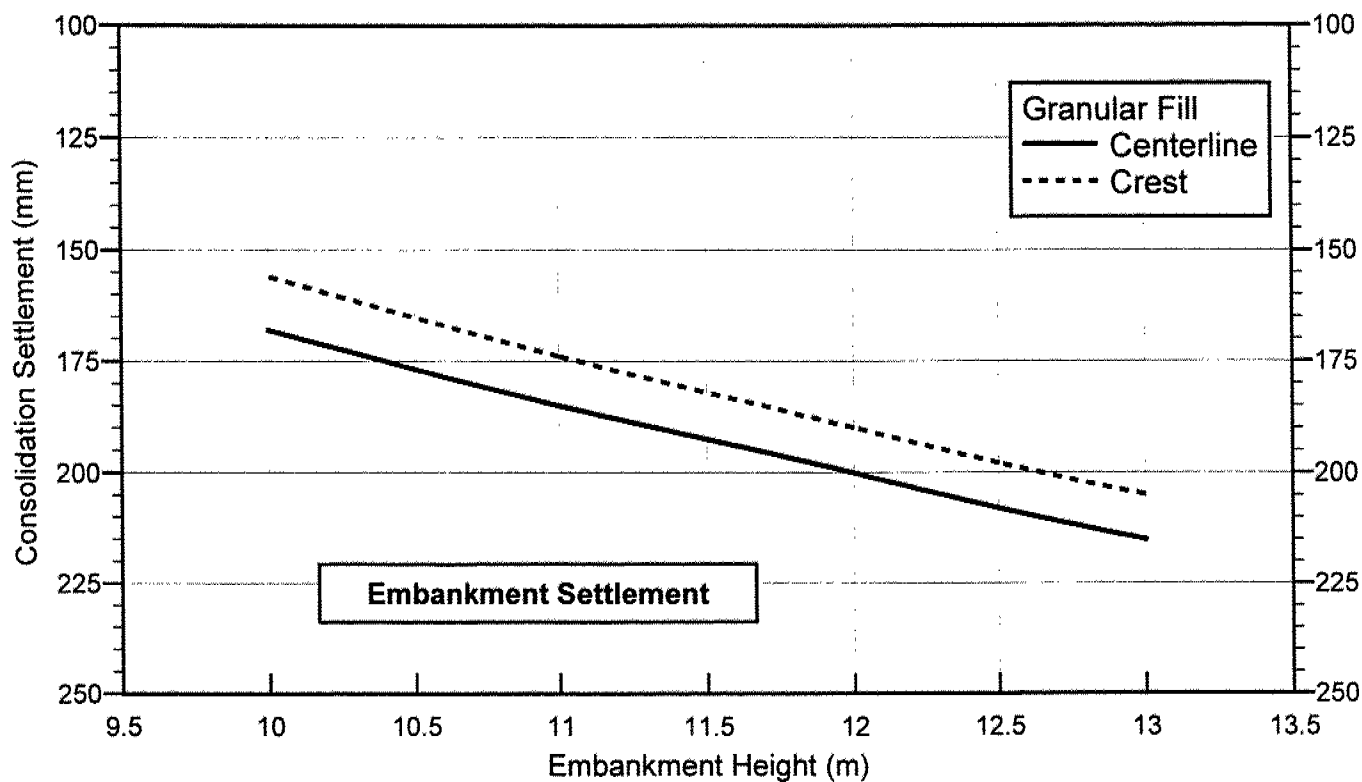
Footing SLS Bearing Resistance  
Granular Fill on Native Soil - SOUTH APPROACH

Trout Creek Bridge - NORTHBOUND LANES

F98179-B/G

Nov 23/99

Figure A5



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**Estimated Consolidation Settlement  
SOUTH APPROACH**

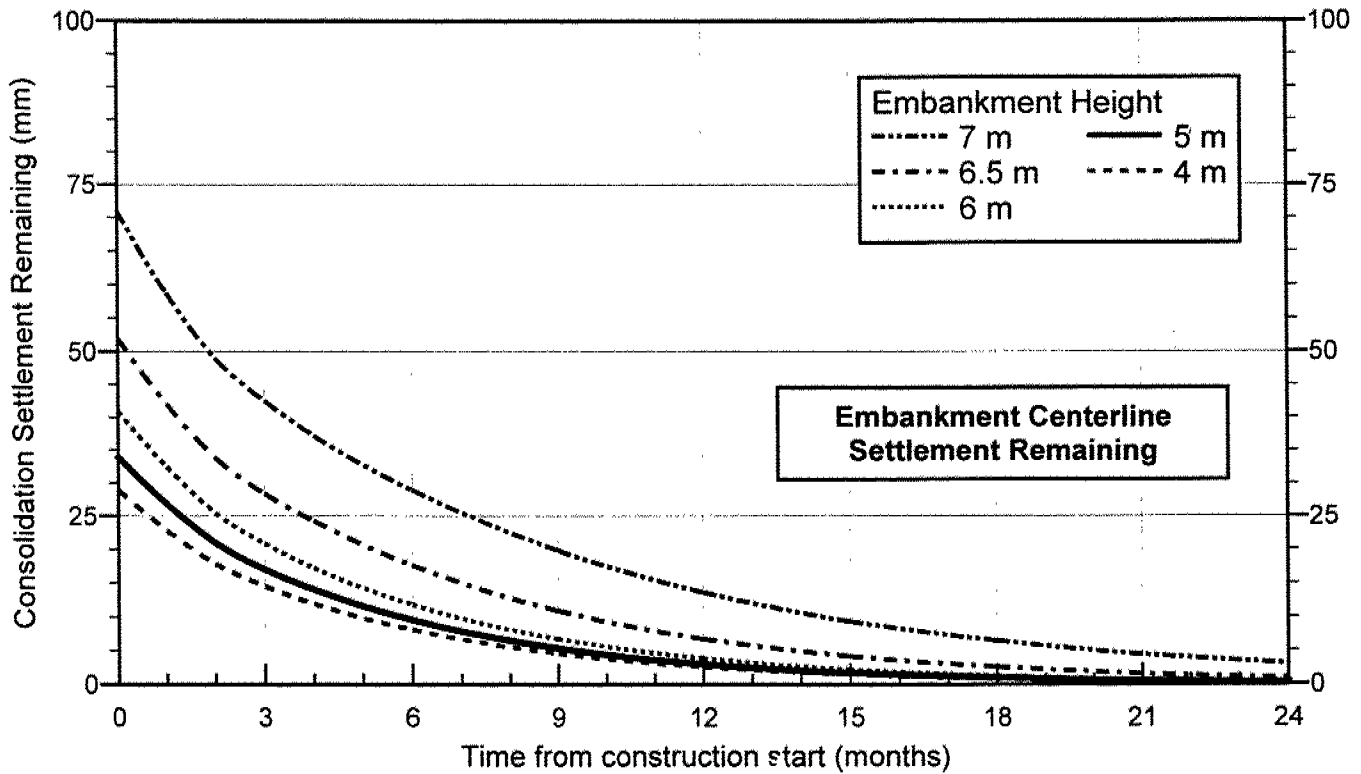
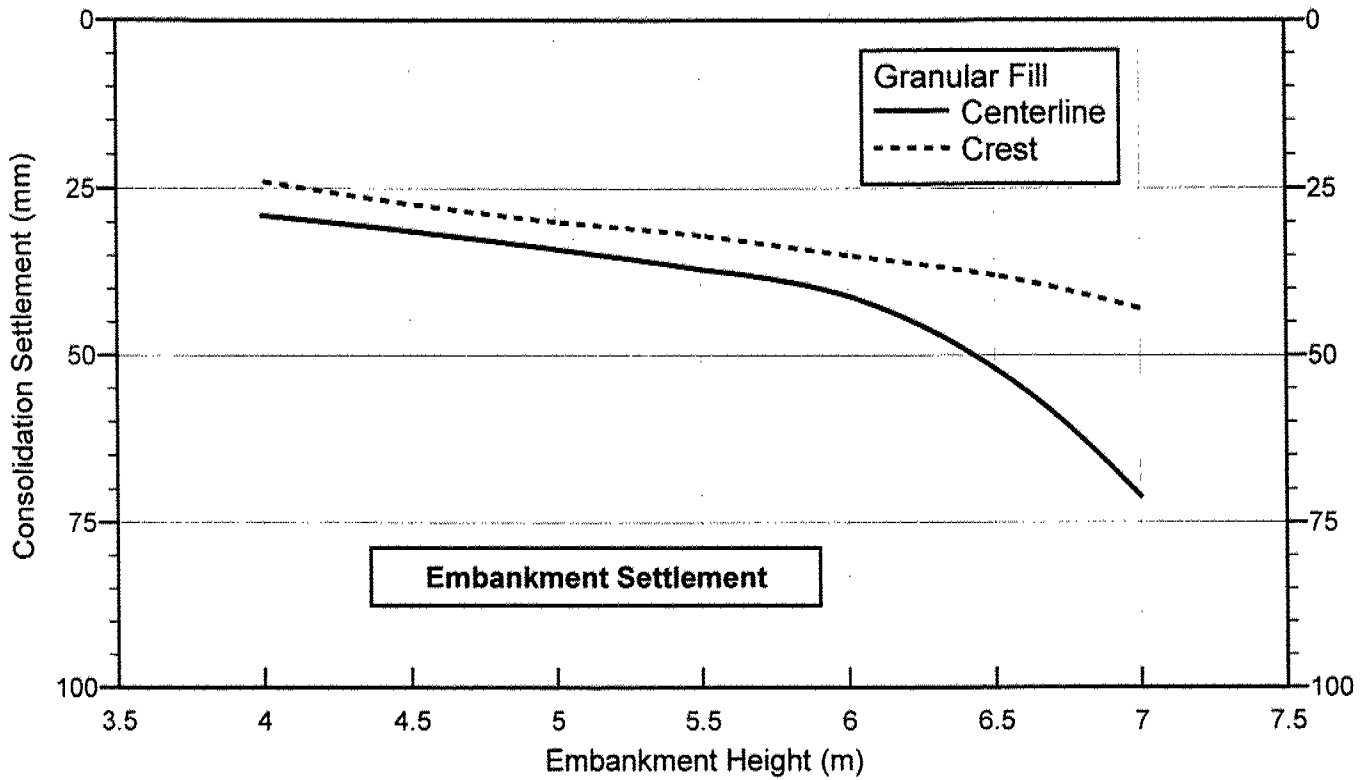
F98179-B/G

Mar 3/99

Marshall Macklin Monaghan

**Trout Creek Bridge - NORTHBOUND LANES**

Figure A6



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### Estimated Consolidation Settlement NORTH APPROACH

F98179-B/G

Mar 3/99

Marshall Macklin Monaghan

Trout Creek Bridge - NORTHBOUND LANES

Figure A7

B

# RECORD OF BOREHOLE BH-1NF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+564, offset 6 m left of centreline of Northbound Lane  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55  
 DATUM Geodetic DATE November 17, 1998  
 ORIGINATED BY I.D.  
 COMPILED BY M.D.  
 CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20	40	60	80					
313.80	GROUND SURFACE														
0.00	TOPSOIL, 150 mm over SAND, fine to medium, some SILT, brown. (loose)		1	SS	9										
312.60			2	SS	8										
1.20	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (stiff becoming firm)		3	SS	2										
			4	SS	2										
309.53															
4.27	SILTY SAND & GRAVEL, till like structure, some cobbles, grey, wet. (dense)		5	SS	27										
307.46			6	SS	60										
6.34	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER														
	Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 017.2 N, 313 985.1 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model.														



# RECORD OF BOREHOLE BH-2NF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+559, offset ~6 m right of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / B.Q. core COMPILED BY M.D.  
 DATUM Geodetic DATE November 17, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION			
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20 40 60 80				wp — w — wl							
								SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)							
						UNCONFINED QUICK TRIAXIAL X FIELD VANE LAB SHEAR													
						20 40 60 80				10 20 30 40									
314.30	GROUND SURFACE																		
0.00	TOPSOIL, ~150 mm over SAND, fine to medium, trace of SILT, brown, occasional organics. (compact)																		
			1	SS	25											0% 97% 3%			
			2	SS	15														
311.95																			
2.35	SILTY SAND & GRAVEL, till-like structure, some cobbles, grey, wet. (compact)		3	SS	17														
			4	SS	11														
310.43																			
3.87	BIOTITE HORNBLENDE GNEISS		5	BQ												Rec. 100% RQD 87%			
			6	BQ												Rec. 100% RQD 96%			
307.29																			
7.01	END OF BOREHOLE																		
Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 092 981.7 N, 313 998.0 E. 3) Borehole elevation obtained Marshall Macklin Monaghan terrain model. 4) Water level was ~2.3 & hole was open to ~2.4 m depth on completion.																			





# RECORD OF BOREHOLE BH-3NF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+601, offset ~6 m left of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / B.Q. core COMPILED BY M.D.  
 DATUM Geodetic DATE November 16, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION		
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20 40 60 80				wp		w			wl	
								SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL    FIELD VANE LAB SHEAR				WATER CONTENT (%)						
312.54	GROUND SURFACE															GR SA (SI & CL)		
0.00	SAND, fine to medium, occasional organics, brown, dry.																	
311.94	CLAYEY SILT, with bands of SILT, brown to grey, wet seams below ~2.5 m depth. (compact to loose)		1	SS	14													
0.60			2	SS	12													
			3	SS	4													
309.49			4	SS	3													
3.05			5	SS	2													
			6	SS	3													
304.62	SAND & GRAVEL, till-like structure, some cobbles, trace of SILT, grey, wet. (dense)		7	SS	18											0% 7% 93%		
7.92			8	SS	30											40% 50% 10%		
302.42	BIOTITE HORNBLENDE GNEISS		9	BQ												Rec. 100% RQD 78%		
10.12			10	BQ												Rec. 100% RQD 90%		
299.25																		
13.29	Notes: 1) This borehole forms part of the Trout Creek Bridge, Northbound Lane, Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 050.0 N, 313 968.9 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model. 4) Borehole caved dry at ~8.4 m depth on completion. 5) Area levelled with a bulldozer prior to advancing borehole. 6) This borehole is located at the top edge of a steep slope (dropping of northward).																	



# RECORD OF BOREHOLE BH-4NF 1 OF 1

## METRIC

W.P. 774-93-00      LOCATION Station ~10+602, offset ~6 m right of centreline of Northbound Lane      ORIGINATED BY I.D.  
 DIST 54      HWY 11      BOREHOLE TYPE Hollow stem augers / CME-55      COMPILED BY M.D.  
 DATUM Geodetic      DATE November 16, 1998      CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20    40    60    80		wp    —    w    —    wl				
								SHEAR STRENGTH: Cu, KPa ● UNCONFINED QUICK TRIAXIAL    X FIELD VANE LAB SHEAR		WATER CONTENT (%) 10    20    30    40				
313.51	GROUND SURFACE													
0.00	TOPSOIL, ~150 mm over SAND, fine to medium, occasional organics, brown, dry. (compact)		1	SS	11		313							
312.29	SILTY CLAY, grey, moist to wet, thinly laminated with bands of SILT, low to medium plasticity. (stiff becoming firm)		2	SS	11		312							
1.22			3	SS	3		311							0%    0%    100%
			4	SS	2		310							
			5	SS	1		309							0%    0%    100%
		6	SS	3	308									
		7	SS	38	307									
306.19	SAND & GRAVEL, till-like structure, some cobbles, trace of silt, grey, moist. (dense)						306							
7.32							305							
304.52	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER													
8.99	<p>Notes:</p> <p>1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation.</p> <p>2) Borehole located at U.T.M. coordinates 5 093 056.0 N, 313 979.3 E.</p> <p>3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew.</p> <p>4) Borehole caved wet at ~7.8 m depth on completion.</p> <p>5) This borehole was located at the top edge of a cliff (dropping off northward).</p>													



# RECORD OF BOREHOLE BH-5NF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+659, on centreline of Northbound Lane  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55  
 DATUM Geodetic DATE November 10, 1998  
 ORIGINATED BY I.D.  
 COMPILED BY M.D.  
 CHECKED BY E.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)					
								UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR	20	40	60	80	10			20
307.04	GROUND SURFACE														GR	SA	(SI & CL)
0.00	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		1	SS	5		306										
			2	SS	3		305										
			3	SS	2		304										
			4	SS	4		303										
			5	SS	6		302										
301.81	SILTY SAND & GRAVEL, brown, wet. (compact)		6	SS	20		301										
5.23							300										
300.12	BIOTITE HORNBLENDE GNEISS,		7	BQ			299										Rec. 100%R.Q.D. 10%
6.92			8	BQ			298										Rec. 100%R.Q.D. 20%
			9	BQ			297										Rec. 100%R.Q.D. 75%
296.77	END OF BOREHOLE																
10.27	<b>Notes:</b> 1) This borehole forms part of the Trout Creek Bridge, Northbound Lane, Foundation Investigation. 2) Borehole located at U.T.M. coordinates 7 093 104.7 N, 313 950.9 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model. 4) Water level was at surface & hole was open to ~5.7 m depth on completion. 5) This area was levelled with a bulldozer prior to advancing borehole. 6) This borehole is located at the top edge of a cliff (dropping off northward).																



# RECORD OF BOREHOLE BH-6NF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+656, offset 7 m left of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 13, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20	40	60	80	wp	w	wl		
GROUND SURFACE							SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL X FIELD VANE LAB SHEAR				WATER CONTENT (%)				
304.88															
0.00	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		1	SS	4										
			2	SS	5										
			3	SS	6										
			4	SS	8										
			5	SS	6										
			6	SS	3										
			7	SS	7										
296.50															
8.38	SAND, GRAVEL & COBBLES														
296.04															
8.84	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER														
	Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 099.2 N, 313 945.6 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Borehole caved wet at ~7.2 m depth on completion. 5) This area was levelled with a bulldozer prior to advancing borehole. 6) This borehole is located at the top edge of a cliff (dropping off northward).														



# RECORD OF BOREHOLE BH-7NF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+769, offset 6 m left of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / BQ core COMPILED BY M.D.  
 DATUM Geodetic DATE November 20, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT		NATURAL MOISTURE CONTENT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m							wp		w			
												w <sub>p</sub>		w			
							SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)						
							UNCONFINED QUICK TRIAXIAL				FIELD VANE LAB SHEAR						
							20 40 60 80				10 20 30 40						
310.51	GROUND SURFACE																
0.00	SAND, medium, brown, moist. (compact)		1	SS	20												
			2	SS	15												
308.38			3	SS	17												
2.13	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (stiff to firm)		4	SS	17												
			5	SS	3												
			6	SS	8												
			7	SS	6												
			8	SS	9												
			9	SS	9												
299.14			10	BQ													
11.37	BIOTITE HORNBLende GNEISS		11	BQ													
			12	BQ													
295.94																	
14.57	END OF BOREHOLE																
<p>Notes:</p> <p>1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation.</p> <p>2) Borehole located at U.T.M. coordinates 5 093 204.5 N, 313 907.7 E.</p> <p>3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model.</p> <p>4) Borehole caved dry at ~3.8 m depth on completion.</p> <p>5) This area was levelled with a bulldozer prior to advancing borehole.</p>																	

Notes:  
 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation.  
 2) Borehole located at U.T.M. coordinates 5 093 204.5 N, 313 907.7 E.  
 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model.  
 4) Borehole caved dry at 3.8 m depth on completion.  
 5) This area was levelled with a bulldozer prior to advancing borehole.



# RECORD OF BOREHOLE BH-8NF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+765, offset 7 m right of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 20, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20	40	60	80					
312.21	GROUND SURFACE														
0.00	TOPSOIL, 150 mm over SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (stiff to firm)		1	SS	22										
			2	SS	26										
			3	SS	15										
			4	SS	10										
			5	SS	5										
			6	SS	3										
			7	SS	3										
			8	SS	6										
			9	SS	7										
			10	SS	11										
298.51			11	SS	20										
13.79	SILTY SAND & GRAVEL, till-like structure, possible cobbles, wet. (dense)														
298.40															
13.81															
	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER														
	Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 204.7 N, 313 921.3 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Borehole caved wet at 11.6 m depth on completion.														



# RECORD OF BOREHOLE BH-9NF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+810, offset ~6 m left of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 24, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80		wp — w — wl			
								SHEAR STRENGTH: Cu, KPa		WATER CONTENT (%)			
							UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR					
							20 40 60 80			10 20 30 40	kN/m <sup>3</sup>	GR SA (SI & CL)	
312.80	GROUND SURFACE												
0.00	TOPSOIL, ~150 mm over SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (stiff to firm)		1	SS	15							0% 0% 100%	
			2	SS	15								
			3	SS	8								
			4	SS	6								
			5	SS	1								
			6	SS	2							0% 2% 98%	
			7	SS	3								
			8	SS	6								
			9	SS	6								
301.37	SILT, occasional thin CLAY seams, grey, wet. (loose)		10	SS	8								
299.39	SILTY SAND & GRAVEL, till-like structure, grey, wet. (compact to dense)		11	SS	13								
298.11	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER												
14.69	Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 243.3 N, 313 896.0 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model. 4) Borehole caved wet at ~12.7 m depth on completion.												

Notes:  
 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation.  
 2) Borehole located at U.T.M. coordinates 5 093 243.3 N, 313 896.0 E.  
 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model.  
 4) Borehole caved wet at ~12.7 m depth on completion.



# RECORD OF BOREHOLE BH-10NF 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station ~10+810, offset ~6 m right of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 24, 1998 CHECKED BY E.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	20	40	60	80	wp			w
312.93	GROUND SURFACE														
0.00	TOPSOIL, ~150 mm over SAND, fine, ~150 mm over SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (stiff to firm)		1	SS	17										
			2	SS	12										
			3	SS	9										
			4	SS	4										
			5	TW											
			6	SS	2										
			7	TW											
			8	SS	8										
			9	SS	12										
301.04			10	SS	21										
11.89	SILT, with occasional thin CLAY seams, grey, wet. (loose)														
299.82			11	SS	6										
13.11	SILTY SAND & GRAVEL, till-like structure, grey, moist. (compact to dense)														
298.18			12	BQ											
14.75	BIOTITE HORNBLENDE GNEISS  Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 246.6 N, 313 907.5 E.														
			13	BQ											
295.13															
17.80	END OF BOREHOLE Notes: (cont'd) 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model. 4) Borehole caved wet at ~11.4 m depth on completion.														





# RECORD OF BOREHOLE BH-2DF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+668, offset 7 m right of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 11, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20	40	60	80					
304.80 0.00	GROUND SURFACE														
	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		1	SS	7										
			2	SS	8										
			3	SS	5										
			4	SS	3										
			5	SS	4										
298.55 6.25	BIOTITE HORNBLENDE GNEISS, good to excellent rock quality, unweathered.		6	SS	8										0% 15% 85%
			7	BQ											Rec. 99% RQD 78%
			8	BQ											Rec. 100% RQD 96%
295.50 9.30	END OF BOREHOLE														
Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 115.5 N, 313 954.0 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Borehole caved wet at 76.0 m depth on completion.															



# RECORD OF BOREHOLE BH-3DF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+714, offset ~5 m left of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / Dynamic cone COMPILED BY M.D.  
 DATUM Geodetic DATE June 26, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION		
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80				wp  -----  w  -----  wl				
								SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL      FIELD VANE LAB SHEAR 20 40 60 80				WATER CONTENT (%) 10 20 30 40				
301.11	GROUND SURFACE															
0.00	TOPSOIL, ~150 mm over SILTY SAND, some wood inclusions, brown to grey, wet below ~1.5 m depth, CREEK ALLUVIUM. (loose to very loose)		1	SS	7											
			2	SS	9											
			3	SS	3											
			4	SS	0											
297.11																
4.00	SILTY SAND & GRAVEL, till-like structure, occasional cobbles, grey, wet. (dense)		5	SS	40											
			6	SS	10											
294.34																
6.77	BIOTITE HORNBLENDE GNEISS, excellent rock quality, unweathered.		7	BQ												
			8	BQ												
291.27																
9.84	END OF BOREHOLE															
Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 153.4 N, 313 926.4 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan survey crew. 4) Water level was at ~1.4 m & hole was open to ~3.9 m depth on completion.																



# RECORD OF BOREHOLE BH-4DF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+745, offset ~7 m right of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / Dynamic cone COMPILED BY M.D.  
 DATUM Geodetic DATE June 29, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT wp	NATURAL MOISTURE CONTENT w	LIQUID LIMIT wl	UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20	40	60	80					
306.06 0.00	GROUND SURFACE														
305.48 0.60	SAND														
	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		1	SS	9										
			2	SS	4										
			3	SS	5										
			4	SS	6										
			5	SS	6										
			6	SS	6										
298.86 7.20	SAND, GRAVEL & COBBLES														
298.50 7.56	BIOTITE HORNBLENDE GNEISS, fair to good rock quality, slightly weathered to weathered.		7	BQ											Rec. 100% RQD 71%
			8	BQ											Rec. 100% RQD 78%
295.36 10.70	END OF BOREHOLE														
Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 186.1 N, 313 927.5 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Water level was at ~5.2 m & hole was open to ~5.4 m depth on completion. 5) Dynamic cone penetration test driven at station ~10+745, offset ~7 m right of centreline as referenced to the Northbound Lane.															



# RECORD OF BOREHOLE BH-5DF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+663, offset ~2 m left of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 11, 1998 CHECKED BY I.G.

SOIL PROFILE				SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION			
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)						
								UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR									
304.31	GROUND SURFACE							20	40	60	80	wp	w	wl	GR	SA	(SI & CL)	
0.00	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		1	SS	5		304											
			2	SS	6		303	⊗						○				
			3	SS	5		302	⊗						○				
			4	SS	6		301	⊗						○				
			5	SS	7		300											
298.27	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER						299	⊗					○			0%	0%	100%
6.04	Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 107.6 N, 313 947.6 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Borehole was dry & open to full depth on completion.																	



# RECORD OF BOREHOLE BH-6DP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+620, offset ~8 m right of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 16, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80		wp — w — wl				WATER CONTENT (%) 10 20 30 40
								SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL X FIELD VANE LAB SHEAR						
313.74	GROUND SURFACE													
0.00	TOPSOIL, ~200 mm over SAND, fine to medium, some silt, brown, moist. (loose to very loose)		1	SS	3								0% 97% 3%	
311.74			2	SS	9									
2.00	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (stiff becoming firm)		3	SS	5									
			4	SS	4									
			5	SS	3									
			6	TW								18.30		
			7	SS	6									
305.24														
8.50	SILTY SAND & GRAVEL, till-like structure, some cobbles, grey, wet. (compact)		8	SS	24								25% 57% 18%	
303.99														
9.75	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER													
<p>Notes:</p> <p>1) This borehole forms part of the Trout Creek Bridge Northbound Lane Approach Foundation Investigation.</p> <p>2) Borehole located at U.T.M. coordinates 5 093 072.8 N, 313 973.6 E.</p> <p>3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model.</p> <p>4) Borehole caved wet at ~9.3 m depth on completion.</p>														



# RECORD OF BOREHOLE BH-8DP 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station +10+600, offset +2 m right of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 16, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		WATER CONTENT (%)			UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80		wp — w — wl				
								SHEAR STRENGTH: Cu, KPa		NATURAL MOISTURE CONTENT				
								● UNCONFINED QUICK TRIAXIAL	× FIELD VANE LAB SHEAR	PLASTIC LIMIT				
313.26	GROUND SURFACE													
0.00	TOPSOIL, ~230 mm over SAND, fine to medium, some silt, brown. (compact)		1	SS	6		313							
311.76	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (stiff to firm)		2	SS	7		312							
1.50			3	SS	3		311						0% 0% 100%	
			4	SS	4		310							
			5	TW			309							
			6	SS	5		308						19.60	
305.46	SILTY SAND & GRAVEL, till-like structure, some cobbles, grey, wet. (very dense)		7	SS	52		307							
7.80			8	SS	58		306							
303.51	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER						305							
9.75	Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 052.5 N, 313 976.5 e. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Borehole caved wet at ~8.9 m depth on completion.						304							



# RECORD OF BOREHOLE BH-9DP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+572, offset 3 m right of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 16, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA	NUMBER	TYPE			BLOWS/0.3m	20	40	60	80	wp	w	wl			10
313.59	GROUND SURFACE																
0.00	TOPSOIL, 200 mm over SAND, fine to medium, some silt, brown. (compact)		1	SS	16												
312.09	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		2	SS	10												
1.50			3	SS	1												
			4	TW													
309.63	SAND, GRAVEL & COBBLES																
3.96																	
309.17																	
4.42	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER																
	Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Approach Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 028.3, 313 989.5 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Water level was at 4.1 m & hole was open to 4.2 m depth on completion.																



# RECORD OF BOREHOLE BH-10DP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+548, offset ~1 m right of centreline of Northbound Lane  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55  
 DATUM Geodetic DATE June 17, 1998  
 ORIGINATED BY I.D.  
 COMPILED BY M.D.  
 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20    40    60    80				wp ——— w ——— wl				
								SHEAR STRENGTH: Cu, KPa ● UNCONFINED QUICK TRIAXIAL    × FIELD VANE LAB SHEAR				WATER CONTENT (%) 10    20    30    40				
314.56	GROUND SURFACE															
0.00	TOPSOIL, ~230 mm over SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (stiff to firm)		1	SS	17		314									
312.56			2	SS	21		313									
2.00	SILTY SAND & GRAVEL, some cobbles. (compact to dense)		3	SS	33		312									
311.97																
2.59	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER															
	Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Approach Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 006.4 N, 313 998.6 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Borehole was dry & open to depth on completion. 5) Drill moved ~2.0 m south of BH-10DF & met auger refusal at ~2.6 m depth.															





# RECORD OF BOREHOLE BH-12DP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+504, offset ~1 m left of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 17, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE		20	40	60	80	wp	w	wl	WATER CONTENT (%)		
316.32	GROUND SURFACE														
0.00	TOPSOIL, ~250 mm over SILTY SAND & GRAVEL, till-like structure, some cobbles, brown to grey, wet below ~1.2 m depth. (compact to dense)		1	SS	12										
			2	SS	35										29% 53% 18%
			3	SS	25										
			4	SS	21										
312.45	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER														
3.87	Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Approach Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 092 967.5 N, 314 017.8 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Water level was at ~1.5 m & hole was open to ~3.7 m depth on completion. 5) Drill moved ~2.0 m north of BH-12DF & met auger refusal at ~3.8 m depth.														



# RECORD OF BOREHOLE BH-17DP 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station 10+628, offset 9 m right of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 19, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60	80	wp	w		
304.11	GROUND SURFACE														
0.00	TOPSOIL, 150 mm over SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm to soft)		1	SS	4										
			2	SS	2										
			3	SS	4										
301.11	SILTY SAND & GRAVEL, till-like structure, grey, some cobbles. (compact to dense)		4	SS	50										
3.00			5	SS	28										
298.93	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER														
5.18	Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Approach Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 07.0 N, 313 954.9 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Borehole caved wet at ~2.6 m depth on completion.														



# RECORD OF BOREHOLE BH-18DF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+712, offset ~7 m right of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 26, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR		
300.76	GROUND SURFACE										
0.00	TOPSOIL, ~150 mm over SAND, traces of organics & roots, brown to grey, wet below ~2.0 m depth. (loose)		1	SS	17						
			2	SS	5						
			3	SS	6						
			4	SS	10						
			5	AS							
295.09	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER										
5.67	Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 155.7 N, 313 938.3 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Water level was at ~2.1 m & hole was open to ~2.4 m depth on completion. 5) Dynamic cone penetration test driven at station ~10+712, offset ~6 m right of centreline as referenced to the Northbound Lane.										



# RECORD OF BOREHOLE BH-19DF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+745, offset ~5 m left of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 30, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT		UNIT WEIGHT  KN/m³	REMARKS & GRAIN SIZE DISTRIBUTION					
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	SHEAR STRENGTH: Cu, KPa		WATER CONTENT (%)							
							UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR	wp			wl				
306.87	GROUND SURFACE					20	40	60	80	10	20	30	40	GR	SA	(SI & CL)
0.00	SILT, grey, moist, local sand layers. (loose)		1	SS	13											
304.37 2.50	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm to soft)		2	SS	6											
			3	SS	3											
			4	TW												
299.87 7.00	SILTY SAND & GRAVEL, till-like structure, grey, wet. (compact)		5	SS	13											
297.63 9.24	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER		6	SS	10											
<p>Notes:</p> <p>1) This borehole forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation.</p> <p>2) Borehole located at U.T.M. coordinates 5 093 182.2 N, 313 916.1 E.</p> <p>3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew.</p> <p>4) Borehole caved wet at ~7.2 m depth on completion.</p>																



# RECORD OF BOREHOLE BH-21DP 1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station +10+775, on centreline of Northbound Lane

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Hollow stem augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE July 2, 1998

CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION		
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20 40 60 80				wp — w — wl				
							SHEAR STRENGTH: UNCONFINED QUICK TRIAXIAL (●) FIELD VANE LAB SHEAR (X) Cu, KPa				WATER CONTENT (%)				
					20 40 60 80				10 20 30 40			kN/m³	GR SA (SI & CL)		
312.76	GROUND SURFACE														
0.00	TOPSOIL, ~150 mm over SAND, medium, brown, moist. (loose)														
311.76	SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (stiff to firm)		1	SS	8										
1.00			2	SS	5										
			3	SS	13										
			4	SS	9										
			5	SS	5										
			6	SS	1										
			7	TW								18.30			
			8	SS	7										
			9	SS	7										
			10	SS	10										
			11	SS	12										
298.46	SAND, GRAVEL & COBBLES														
14.30	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER														
298.19															
14.57															
Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Approach Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 211.9 N, 313 911.6 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model. 4) Borehole caved dry at ~11.4 m depth on completion.															



# RECORD OF BOREHOLE BH-23DP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+825, on centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE September 23, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				WATER CONTENT (%)				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20	40	60	80	10	20	30	40		
312.14	GROUND SURFACE															
0.00	TOPSOIL, ~175 mm over SILTY CLAY, grey, moist to wet, thinly laminated with SILT, low to medium plasticity. (firm)		1	SS	11											
			2	SS	7											0% 0% 100%
			3	SS	6											
			4	SS	4											
			5	TW											19.50	
			6	SS	4											0% 0% 100%
			7	SS	8											
			8	SS	7											
301.39	SILT, grey, wet. (loose to compact)		9	SS	11											
10.75			10	SS	16											
299.64	SAND, brown. (dense)															
12.50	END OF BOREHOLE															
299.49	Notes: 1) This borehole forms part of the Trout Creek Bridge Northbound Lane Approach Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 259.2 N, 313 897.8 E. 3) Borehole elevation obtained from Dearden and Stanton Ltd., O.L.S. terrain model. 4) Borehole caved wet at ~5.8 m depth on completion.															
12.65																



# RECORD OF BOREHOLE BH-18EP 1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station +10+560, on centreline of Median

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Hollow stem augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE June 19, 1998

CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20	40	60	80	wp	w	wl		
313.75	GROUND SURFACE															
0.00	TOPSOIL, ~125 mm over CLAYEY SILT, seams of SILT, brown to grey, wet below ~2.0 m depth. (stiff)															
			1	SS	6											
			2	SS	4											
	Cobbles at base.															
309.48	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER															
4.27	Notes: 1) This borehole forms part of Trout Creek Bridge Southbound Lane Approach Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 007.8 N, 313 975.6 E. 3) Borehole elevation obtained by Marshall Macklin Monaghan survey crew. 4) Borehole caved dry at ~3.3 m depth on completion.															



# RECORD OF BOREHOLE AP-1NF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+559, offset ~6 m left of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 17, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT				UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20    40    60    80				wp ——— w ——— wl					
								SHEAR STRENGTH: Cu, KPa ● UNCONFINED QUICK TRIAXIAL    × FIELD VANE LAB SHEAR				WATER CONTENT (%) 10    20    30    40					
313.82 0.00	GROUND SURFACE														GR    SA    (SI & CL)		
312.60 1.22	Probable SAND						313										
	Probable SILT and SILTY CLAY						312										
							311										
310.47 3.35	Probable SAND, GRAVEL & COBBLES TILL						310										
308.58 5.24	END OF PROBE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER						309										
Notes: 1) This auger probe forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Probe located at U.T.M. coordinates 5 093 012.8 N, 313 987.4 E. 3) Probe elevation obtained from Marshall Macklin Monaghan terrain model.																	





# RECORD OF BOREHOLE AP-2NF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+561, on centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 17, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT				UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20    40    60    80				wp ——— w ——— wl					
								SHEAR STRENGTH: Cu, KPa ● UNCONFINED    × FIELD VANE QUICK TRIAXIAL    LAB SHEAR				WATER CONTENT (%) 10    20    30    40					
313.91 0.00	GROUND SURFACE														GR    SA    (SI & CL)		
	Probable SAND						313										
312.54 1.37	-----						312										
	Probable SILT and SILTY CLAY																
310.71 3.20	-----						311										
	Probable SAND, GRAVEL & COBBLE TILL						310										
309.76 4.15	END OF PROBE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER.  Notes: 1) This auger probe forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Borehole located at U.T.M. coordinates 5 093 017.3 N, 313 991.8 E. 3) Borehole elevation obtained from Marshall Macklin Monaghan terrain model.																



# RECORD OF BOREHOLE AP-3NF 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station ~10+563, on centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 17, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT				UNIT WEIGHT  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION  GR   SA   (SI & CL)	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20   40   60   80				wp   ——— w   ——— wl					
								SHEAR STRENGTH: Cu, KPa ● UNCONFINED   QUICK TRIAXIAL   × FIELD VANE LAB SHEAR				WATER CONTENT (%) 10   20   30   40					
313.90 0.00	GROUND SURFACE																
312.53 1.37	Probable SAND						313										
310.85 3.05	Probable SILT and SILTY CLAY						312										
309.72 4.18	Probable SAND, GRAVEL & COBBLES						311										
	END OF PROBE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER.						310										
	Notes: 1) This auger probe forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Probe located at U.T.M. coordinates 5 093 019.0 N, 313 990.9 E. 3) Probe elevation obtained from Marshall Macklin Monaghan terrain model.																



# RECORD OF BOREHOLE AP-4NF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 10+563, offset 7 m right of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE November 17, 1998 CHECKED BY E.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT				UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20    40    60    80				wp    —    w    —    wl					
								SHEAR STRENGTH: Cu, KPa ● UNCONFINED      X FIELD VANE QUICK TRIAXIAL      LAB SHEAR				WATER CONTENT (%) 10    20    30    40					
314.21 0.00	GROUND SURFACE <b>TOPSOIL</b> , ~ 150 mm over						314										
	Probable SAND																
312.99 1.22	-----  Probable SILT and SILTY CLAY						313										
							312										
311.47 2.74	-----  Probable SAND, GRAVEL & COBBLES						311										
310.60 3.61	END OF PROBE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER.  <u>Notes:</u> 1) This auger probe forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Probe located at U.T.M. coordinates 5 093 022.2 N, 313 997.1 E. 3) Probe elevation obtained from Marshall Macklin Monaghan terrain model.																



# RECORD OF BOREHOLE AP-1DF 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station ~10+639, offset ~7 m right of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 19, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT				UNIT WEIGHT kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	20	40	60	80	wp	w		
311.05 0.00	GROUND SURFACE				311	SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL      FIELD VANE LAB SHEAR 20    40    60    80      20    40    60    80				WATER CONTENT (%) 10    20    30    40					GR    SA    (SI & CL)
	Probable SILTY CLAY				310										
					309										
					308										
					307										
					306										
					305										
					304										
					303										
					302										
					301										
				300											
299.62 11.43	END OF AUGER PROBE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER  Notes: 1) This auger probe forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Auger probe located at U.T.M. coordinates 5 093 089.4 N, 313 965.1 E. 3) Probe elevation obtained from Marshall Macklin Monaghan terrain model.														



# RECORD OF BOREHOLE AP-2DF 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station ~10+667, offset ~6 m left of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 19, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT				UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR   SA   (SI & CL)		
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)					
								UNCONFINED QUICK TRIAXIAL	X	FIELD VANE LAB SHEAR	wp	w	wl				
303.83 0.00	GROUND SURFACE																
	Probable SILTY CLAY																
297.86 5.97	END OF AUGER PROBE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER																
	Notes: 1) This auger probe forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Auger probe located at U.T.M coordinates 5 093 109.7N, 313 942.4 E. 3) Probe elevation obtained by Marshall Macklin Monaghan survey crew.																



# RECORD OF BOREHOLE C-1DF

1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station 10+637, offset 7 m right of centreline of Northbound Lane

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Dynamic cone /

COMPILED BY M.D.

DATUM Geodetic

DATE June 9, 1998

CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION		
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	20	40	60	80	wp	w			wl	10
312.16	GROUND SURFACE																
0.00	Dynamic cone penetration test only.																
312																	
311																	
310																	
309																	
308																	
307																	
306																	
305																	
304																	
303																	
302																	
301																	
300.33	END OF CONE TEST DUE TO 'BOUNCING' REFUSAL ON BEDROCK OR BOULDER																
11.83	Notes: 1) This cone test forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Cone test located at U.T.M. coordinates 5 093 087.6 E, 313 965.9 E. 3) Cone test elevation obtained by Marshall Macklin Monaghan survey crew.																



## 1 OF 1

METRIC

CHECKED BY I.G.

[illegible]

# RECORD OF BOREHOLE C-3DF 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station +10+714, offset +10 m right of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Dynamic cone / COMPILED BY M.D.  
 DATUM Geodetic DATE June 26, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT				UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION				
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)							
300.64	GROUND SURFACE					UNCONFINED QUICK TRIAXIAL      FIELD VANE LAB SHEAR				wp      w      wl									
0.00						20	40	60	80	20	40	60	80	10	20	30	40		
0.00	Dynamic cone penetration test only.																		
294.95	END OF CONE TEST DUE TO 'BOUNCING' REFUSAL ON BEDROCK OR BOULDER																		
5.69	<b>Notes:</b> 1) This cone test forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Cone test located at U.T.M. coordinates 5 093 158.6 N, 313 940.4 E. 3) Cone elevation obtained by Marshall Macklin Monaghan survey crew.																		





# RECORD OF BOREHOLE C-4DF

1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station ~10+745, offset ~5 m left of centreline of Northbound Lane

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Dynamic cone /

COMPILED BY M.D.

DATUM Geodetic

DATE June 30, 1998

CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR	wp	w			
306.87	GROUND SURFACE													
0.00	Dynamic cone penetration test only.													
297.60	END OF CONE TEST DUE TO 'BOUNCING' REFUSAL ON BEDROCK OR BOULDER													
9.27	Notes: 1) This cone test forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Cone test located at U.T.M. coordinates 5 093 185.5 N, 313 925.6 E. 3) Cone elevation obtained by Marshall Macklin Monaghan survey crew.													



# RECORD OF BOREHOLE C-5DF

1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~10+660, offset ~6 m right of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Dynamic cone / COMPILED BY M.D.  
 DATUM Geodetic DATE June 9, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			UNCONFINED QUICK TRIAXIAL	Cu, KPa	FIELD VANE LAB SHEAR	wp	w	wl	WATER CONTENT (%)			
305.90	GROUND SURFACE															
0.00	Dynamic cone penetration test only.															
289.42																
6.48	END OF CONE TEST DUE TO 'BOUNCING' REFUSAL ON BEDROCK OR BOULDER															
	Notes: 1) This cone test forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Cone test located at U.T.M. coordinates 5 093 107.9 N, 313 956.1 E. 3) Cone elevation obtained by Marshall Macklin Monaghan survey crew.															



# RECORD OF BOREHOLE C-6DF

1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station +10+642, offset +1 m right of centreline of Northbound Lane ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Dynamic cone / COMPILED BY M.D.  
 DATUM Geodetic DATE June 9, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT NUMBER	TYPE			BLOWS/0.3m	SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL    FIELD VANE LAB SHEAR				WATER CONTENT (%) wp — w — wl			
308.50	GROUND SURFACE					20	40	60	80	10	20	30	40	
0.00	Dynamic cone penetration test only.													
299.81	END OF CONE TEST DUE TO 'BOUNCING' REFUSAL ON BEDROCK OR BOULDER													
8.69	Notes: 1) This cone test forms part of the Trout Creek Bridge Northbound Lane Foundation Investigation. 2) Cone test located at U.T.M. coordinates 5 093 089.7 N, 313 958.4 E. 3) Cone elevation obtained by Marshall Macklin Monaghan survey crew.													



SO7524G/N/F

**TABLE 1**  
**ROCK CORE DESCRIPTION**

BH#	Core Recovery				Core Description	
	RC #	Depth (m)	% CR*	% RQD**	Depth (m)	Description
<b>SOUTH ABUTMENT - NORTH BOUND LANE</b>						
2-NF	1	3.87 to 4.68	100	80	3.87 to 7.01	<b>Biotite Hornblende Gneiss (Garnetiferous)</b> , light grey to grey pink, medium to coarse grained, unweathered sulfate inclusions in joints, moderately close spacing of fractures dipping 45° to 90° from vertical, planar to smooth
	2	4.68 to 7.01	100	85		
3-NF	1	10.12 to 11.60	100	75	10.12 to 13.29	<b>Biotite Hornblende Gneiss</b> , grey to grey pink, medium to coarse grained, unweathered with sulfide inclusions, moderately spaced fractures and joints dipping 45° to 90° from vertical, planar and rough
	2	11.60 to 13.29	100	93		
5-NF	1	6.92 to 8.33	100	53	6.92 to 10.27	<b>Biotite Hornblende Gneiss (Garnetiferous)</b> , grey pink to pinkish red, medium to coarse grained, weathered, fractures very closely spaced, dipping 0° to 90° from vertical, planar, smooth to slightly undulated
	2	8.45 to 10.27	100	81.4		

SO7524G/N/F

**TABLE 1**  
**ROCK CORE DESCRIPTION**

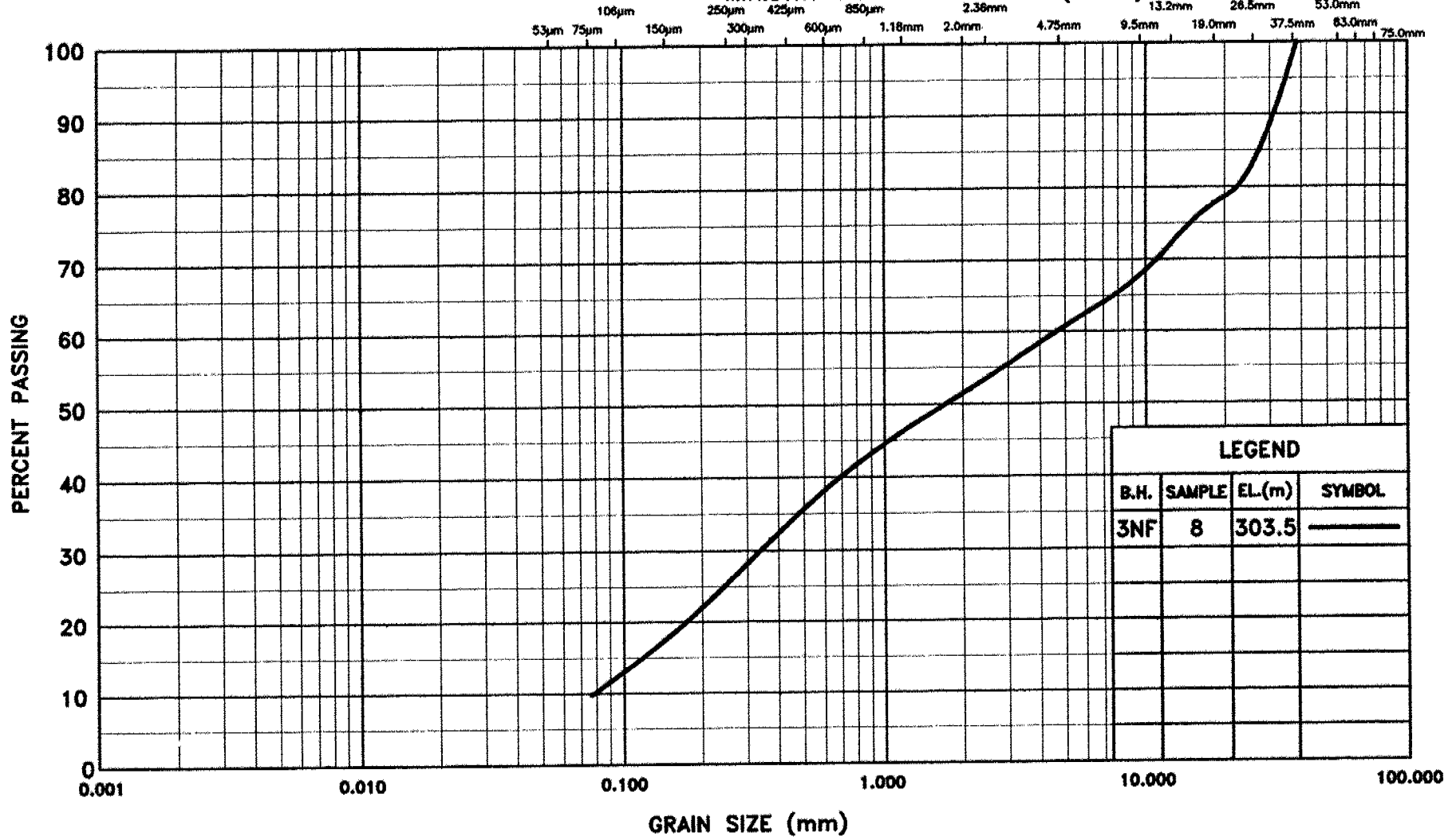
BH#	Core Recovery				Core Description	
	RC #	Depth (m)	% CR*	% RQD**	Depth (m)	Description
7-NF	1	11.37 to 12.86	100	67	11.37 to 14.57	<b>Biotite Hornblende Gneiss</b> , light grey to grey-pink, medium to coarse grained, unweathered with sulfide inclusions, moderately spaced fractures and joints dipping 45° to 90° from vertical, rough and slightly undulating
	2	12.86 to 14.57	100	63		
10-NF	1	14.76 to 16.29	100	90	14.76 to 17.80	<b>Biotite Hornblende Gneiss (Garnetiferous)</b> , light grey to grey-pink, unweathered, some sulfide inclusions, moderately close spacing of fractures and joints dipping from 40° to 90° from vertical, planar and smooth
	2	16.29 to 17.80	100	100		
<div>*CR = Core Recovery</div> <div>**RQD = Rock Quality Designation</div>						

c

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

BH-3NF, SS-8

### GRAIN SIZE DISTRIBUTION

SAND & GRAVEL

FIGURE C-1

W.P. 774-93-00

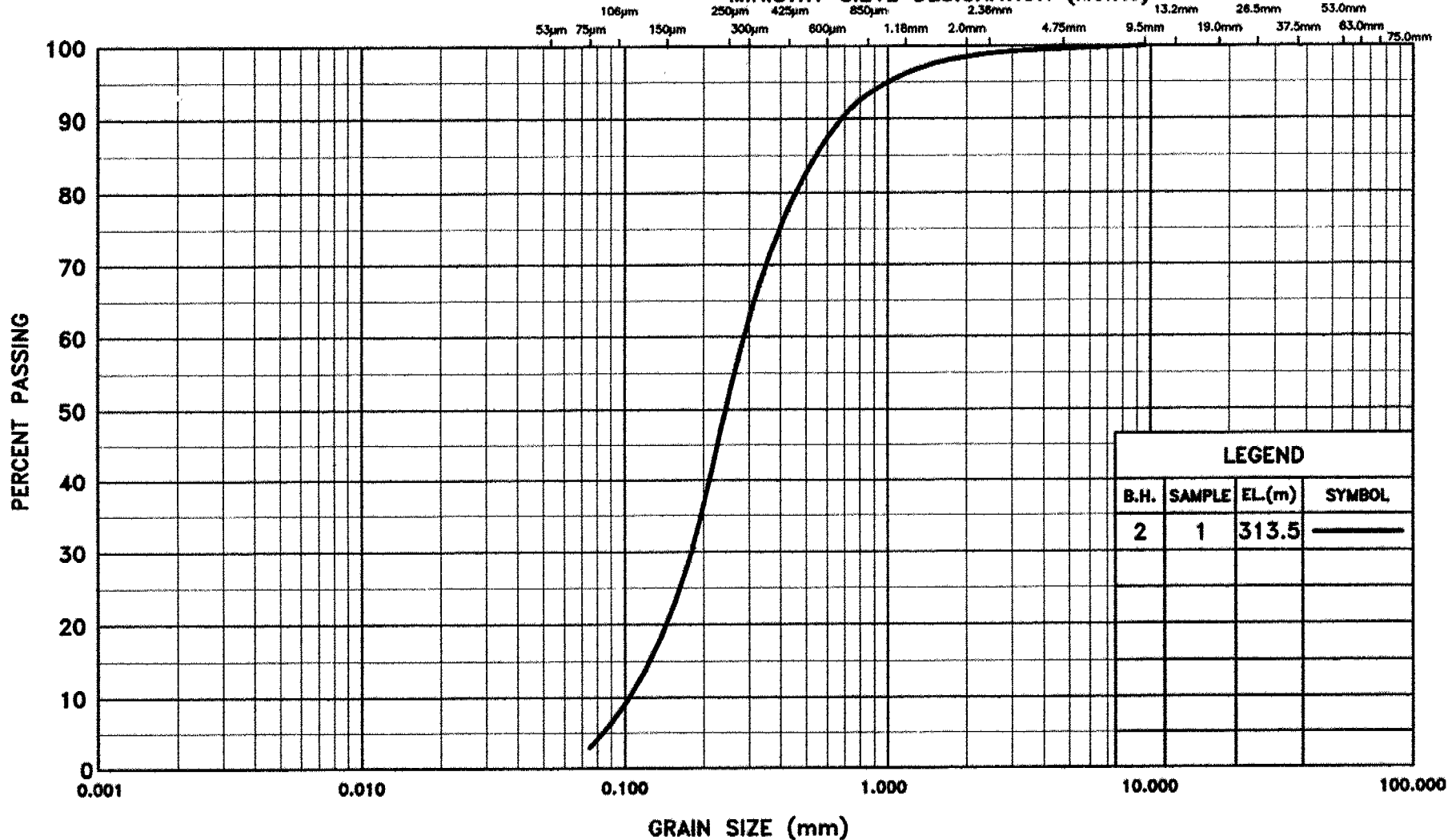


PROJ. No. S07524GN

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

BH-2NF, SS-1

### GRAIN SIZE DISTRIBUTION

SAND

FIGURE C-2

W.P. 774-93-00



PROJ. No. S07524GN



# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT

SAND

GRAVEL

FINE

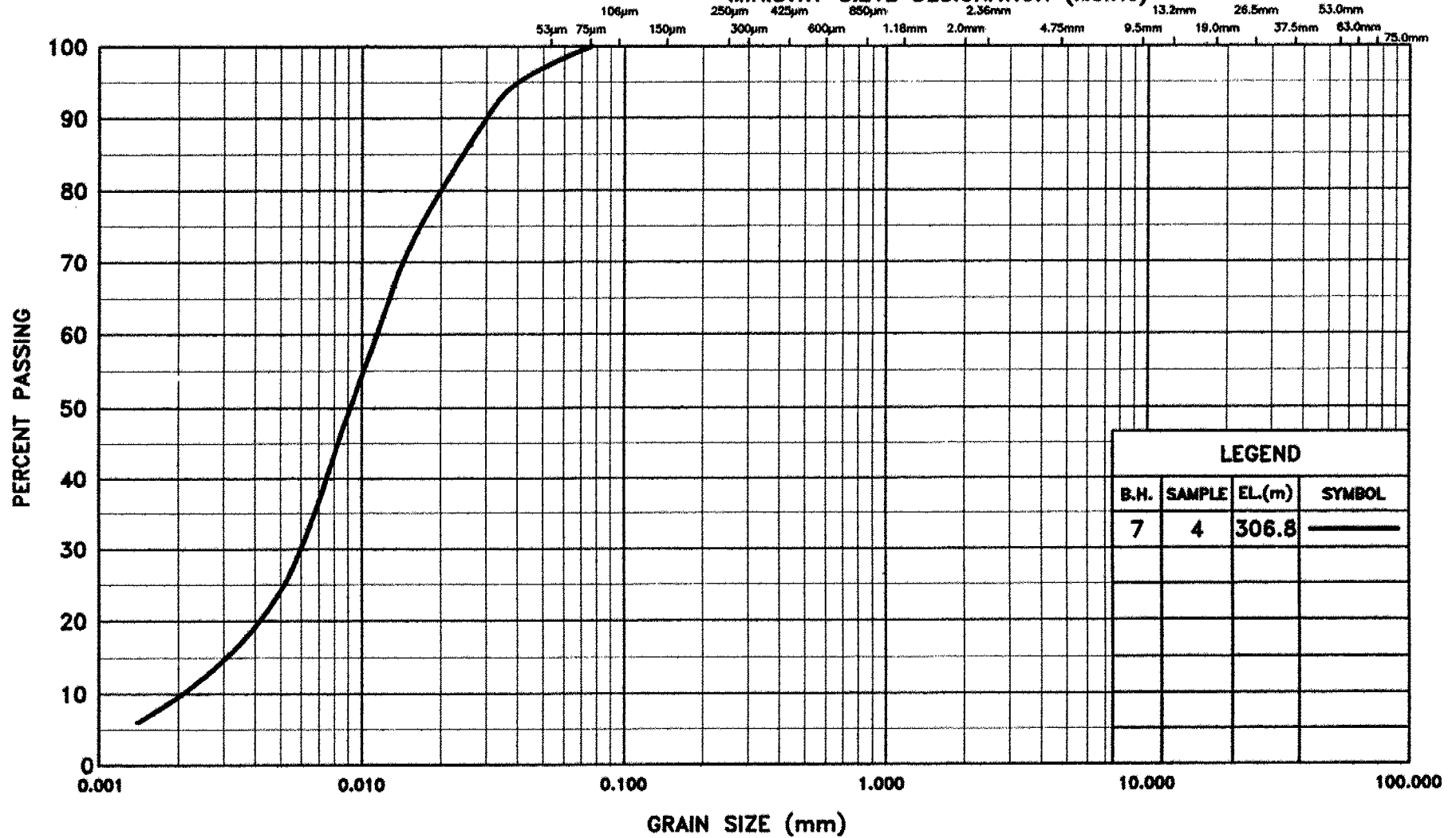
MEDIUM

COARSE

FINE

COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

### GRAIN SIZE DISTRIBUTION

BH-7NF, SS-4 SILT

FIGURE C-3

W.P. 774-93-00

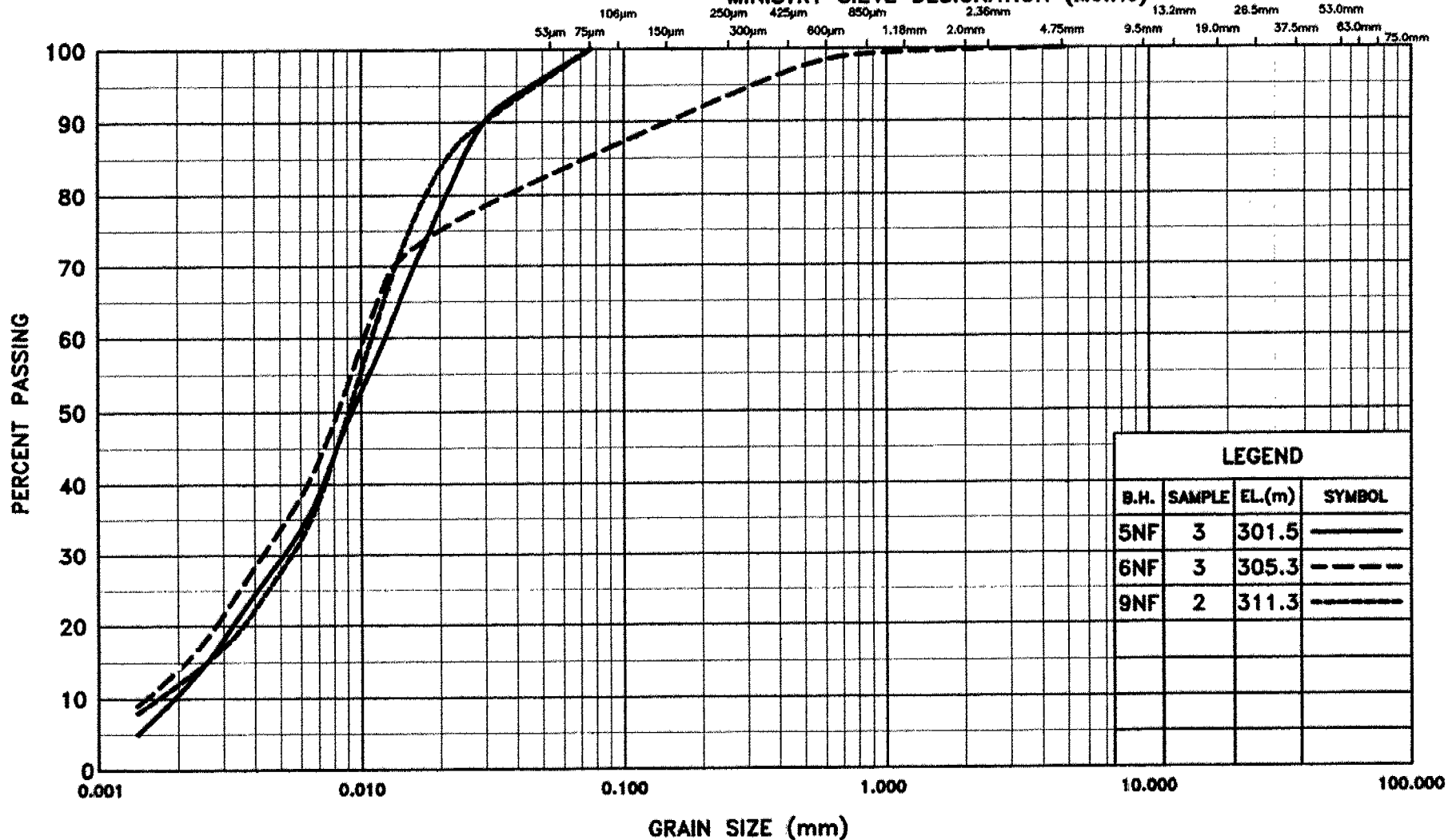


PROJ. No. S07524GN

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION

CLAYEY SILT

FIGURE C-4

W.P. 774-93-00



PROJ. No. S07524GN

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT

SAND

GRAVEL

FINE

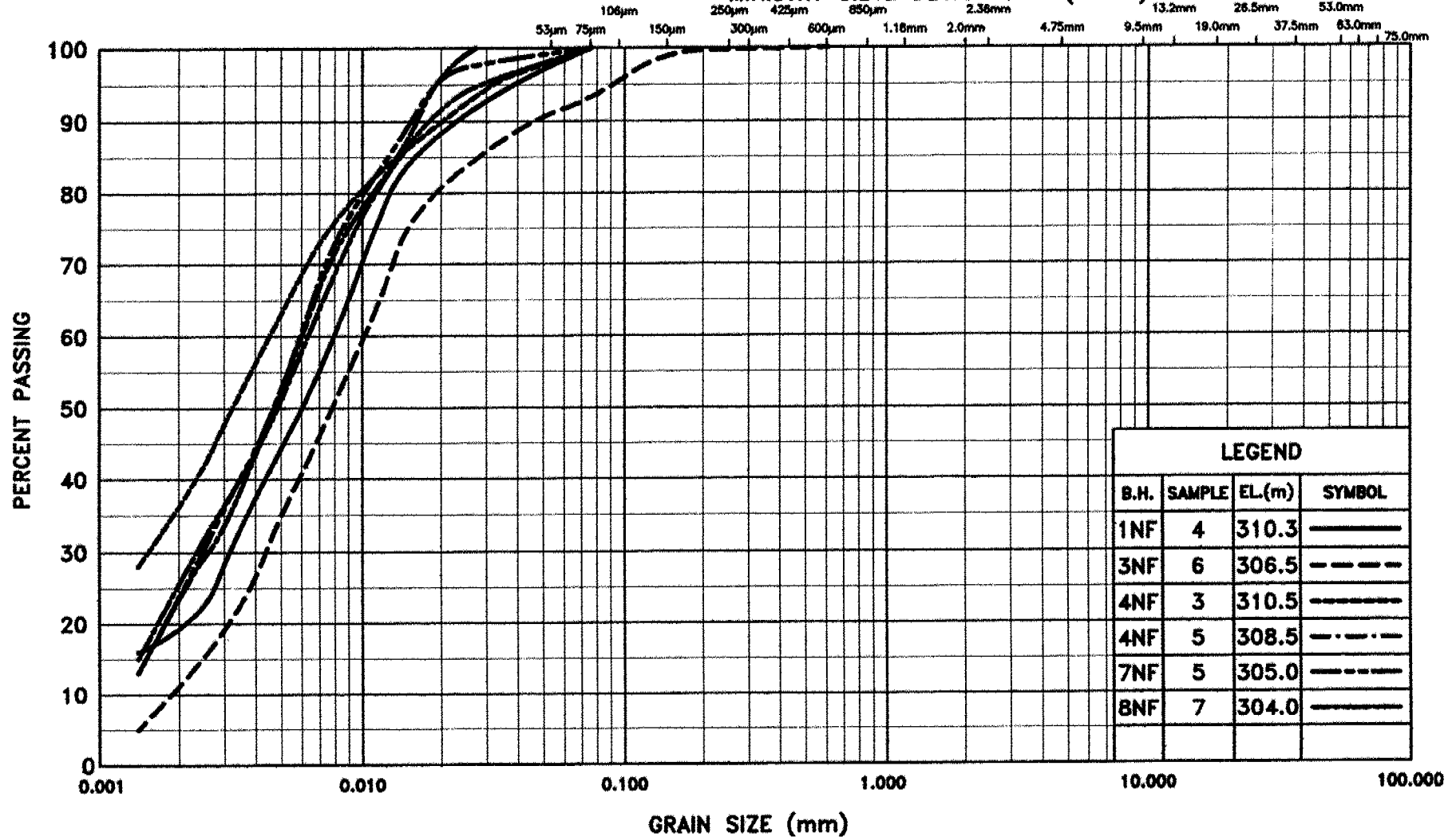
MEDIUM

COARSE

FINE

COARSE

MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION

SILTY CLAY

FIGURE C-5

W.P. 774-93-00



PROJ. No. S07524GN

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT

SAND

GRAVEL

FINE

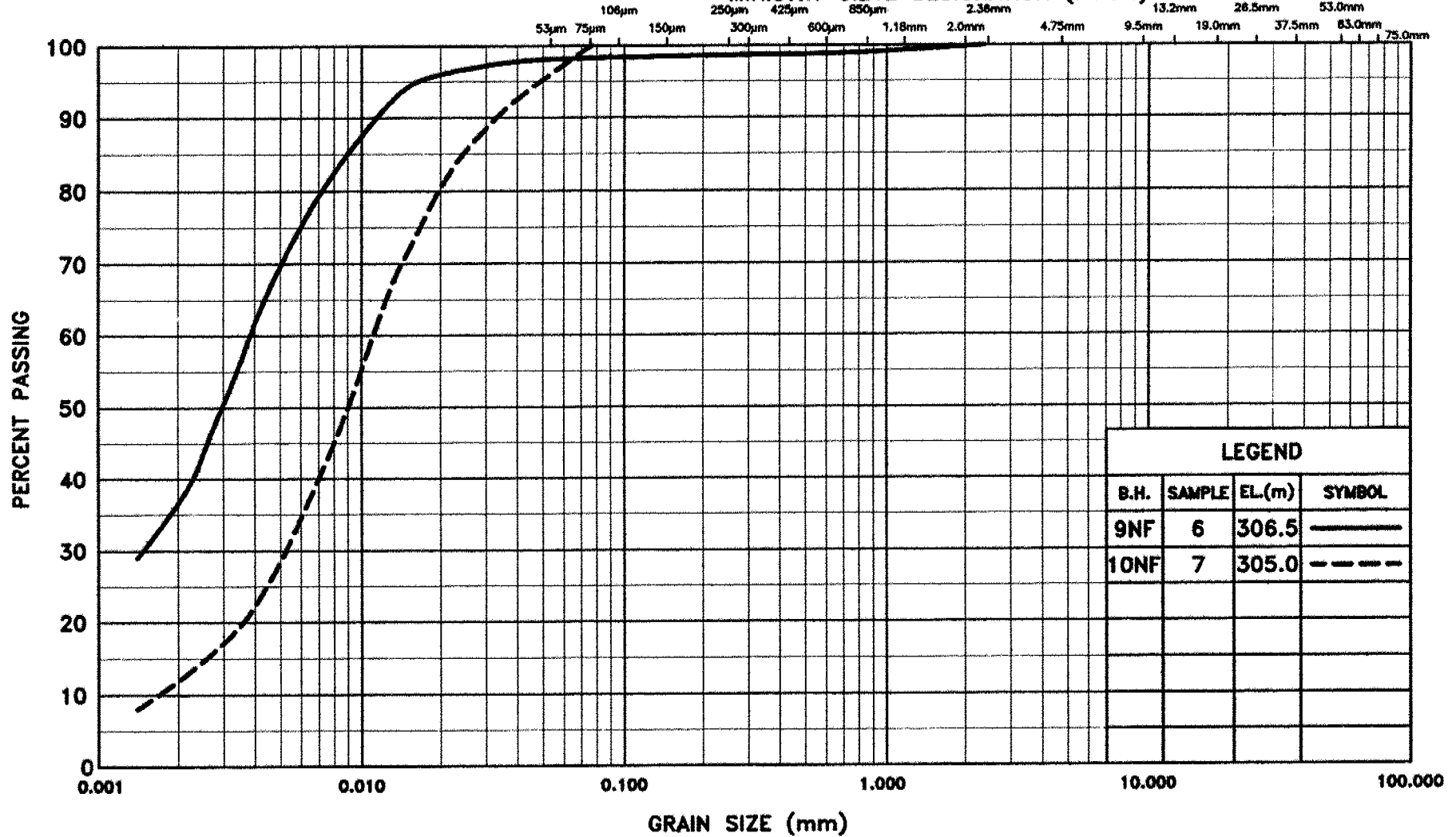
MEDIUM

COARSE

FINE

COARSE

MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION

SILTY CLAY

FIGURE C-6

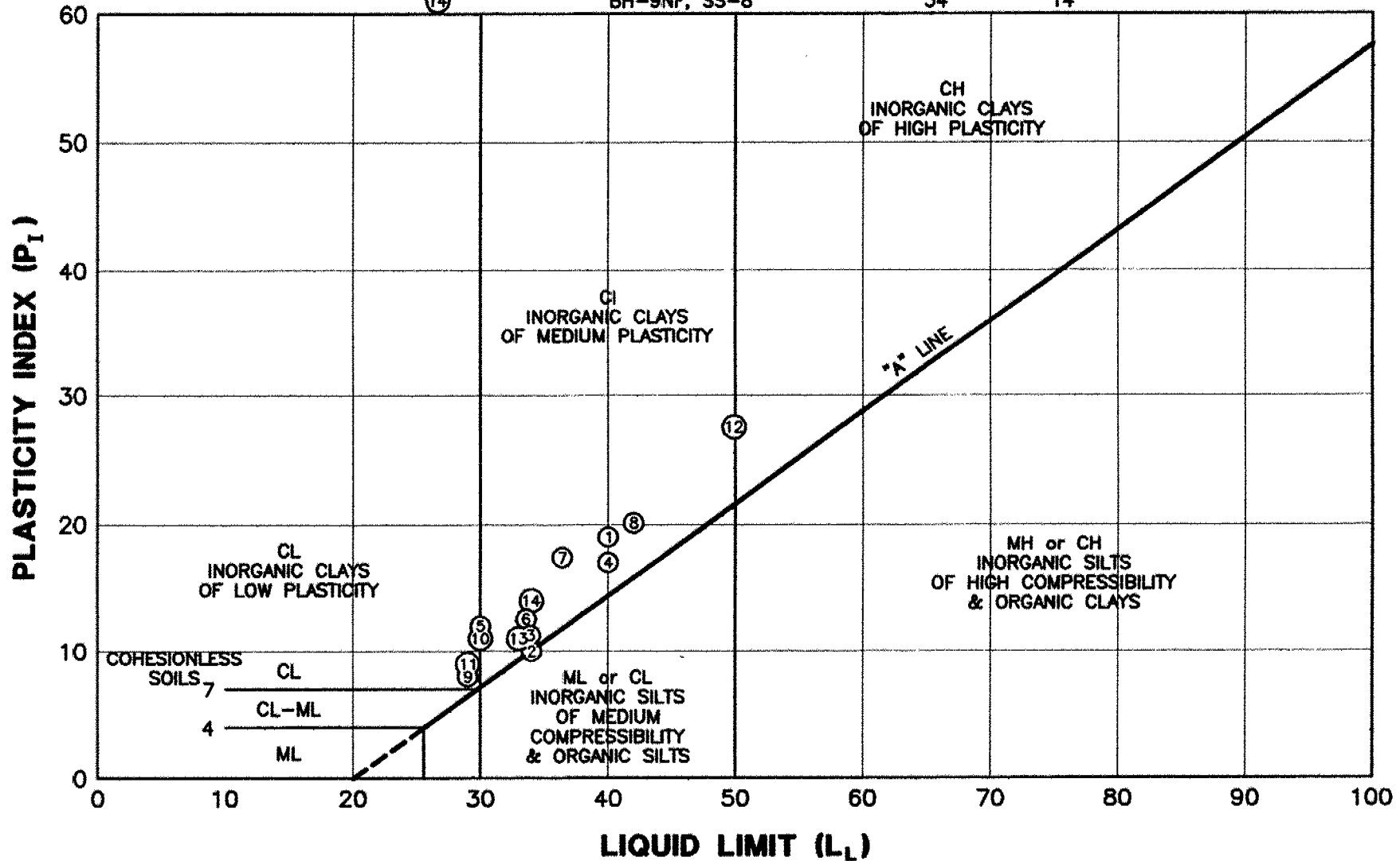
W.P. 774-93-00



PROJ. No. S07524GN

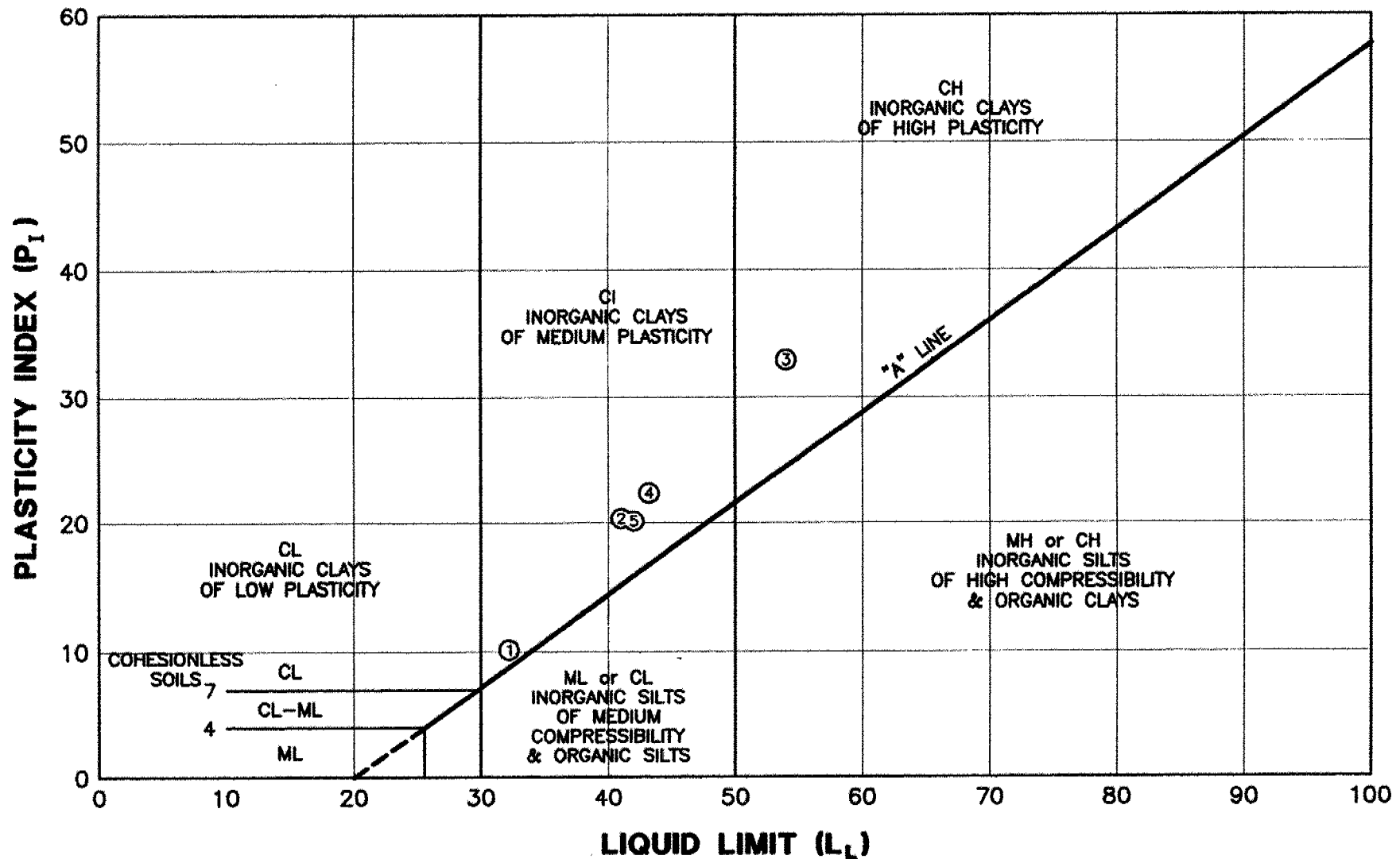
# **ATTERBERG LIMITS - PLASTICITY CHART**

SYMBOL	DESCRIPTION	LL	PI
①	BH-1NF, SS-2	40	19
②	BH-1NF, SS-3	34	10
③	BH-1NF, SS-4	34	11
④	BH-3NF, SS-3	40	17
⑤	BH-5NF, SS-3	30	12
⑥	BH-6NF, SS-5	34	12
⑦	BH-7NF, SS-5	36	17
⑧	BH-8NF, SS-7	42	20
⑨	BH-9NF, SS-3	29	8
⑩	BH-9NF, SS-4	30	11
⑪	BH-9NF, SS-6	29	9
⑫	BH-9NF, SS-6	50	28
⑬	BH-9NF, SS-7	33	11
⑭	BH-9NF, SS-8	34	14



# **ATTERBERG LIMITS - PLASTICITY CHART**

SYMBOL	DESCRIPTION	LL	PI
①	BH-1DF, SS-3	32	10
②	BH-21DP, SS-6	41	21
③	BH-4DF, SS-2	54	33
④	BH-6DP, SS-5	43	22
⑤	BH-8NF, SS-8	42	20



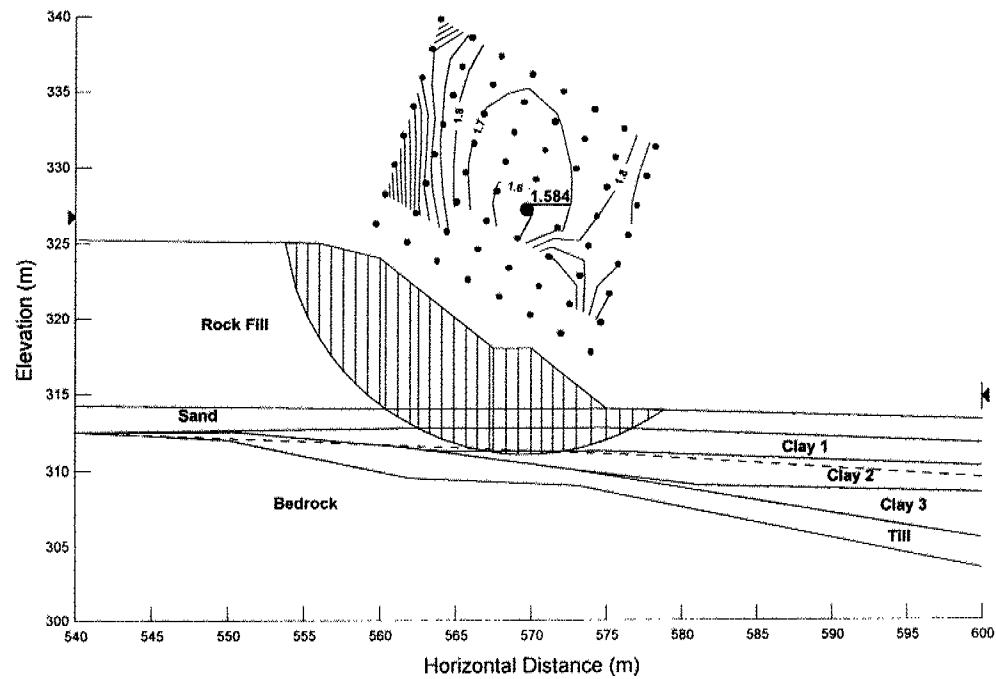
S07524GN

FIG. No. C-8



D

Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-B/G)  
 Northbound Lane, South Abutment  
 NBL\_SALU.SLP



Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Sand  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 32

Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 19  
 Cv 60  
 Rate of Increase -16.67  
 Cv - Minimum 35  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model Undrained (Phi=0)  
 Unit Weight 19  
 Cohesion 35

Soil 5  
 Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 19  
 Cv 35  
 Rate of Increase 2.86  
 Cv - Maximum 70  
 Ch/Cv Ratio 1

Till  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Bedrock  
 Soil Model Bedrock



Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-B/G)  
 North Abutment  
 4 metre embankment height, 1.25:1 side slopes  
 NS\_N4H.SLP

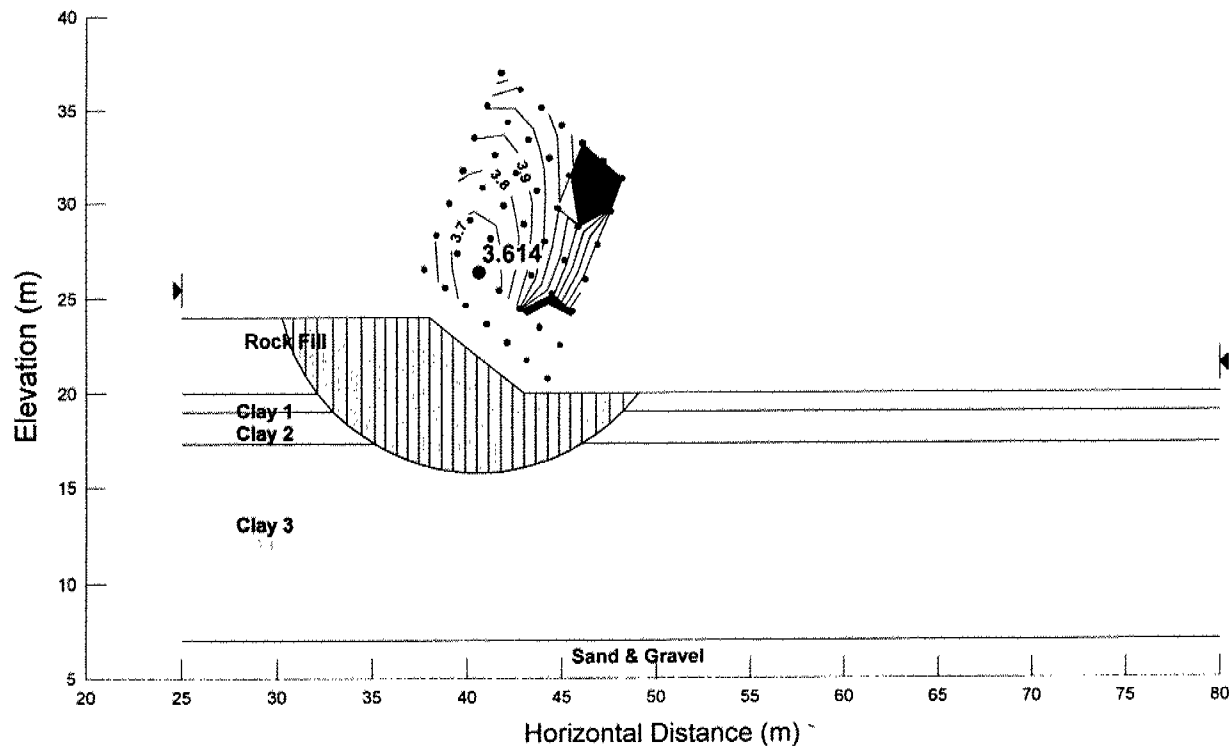
Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 100  
 Rate of Increase -30  
 Cv - Minimum 70  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 70  
 Rate of Increase -28.57  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Clay 3  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 50  
 Rate of Increase 1.95  
 Cv - Maximum 70  
 Ch/Cv Ratio 1

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39



Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-B/G)  
 North Abutment  
 5 metre embankment height, 1.25:1 side slopes  
 NS\_N5H.SLP

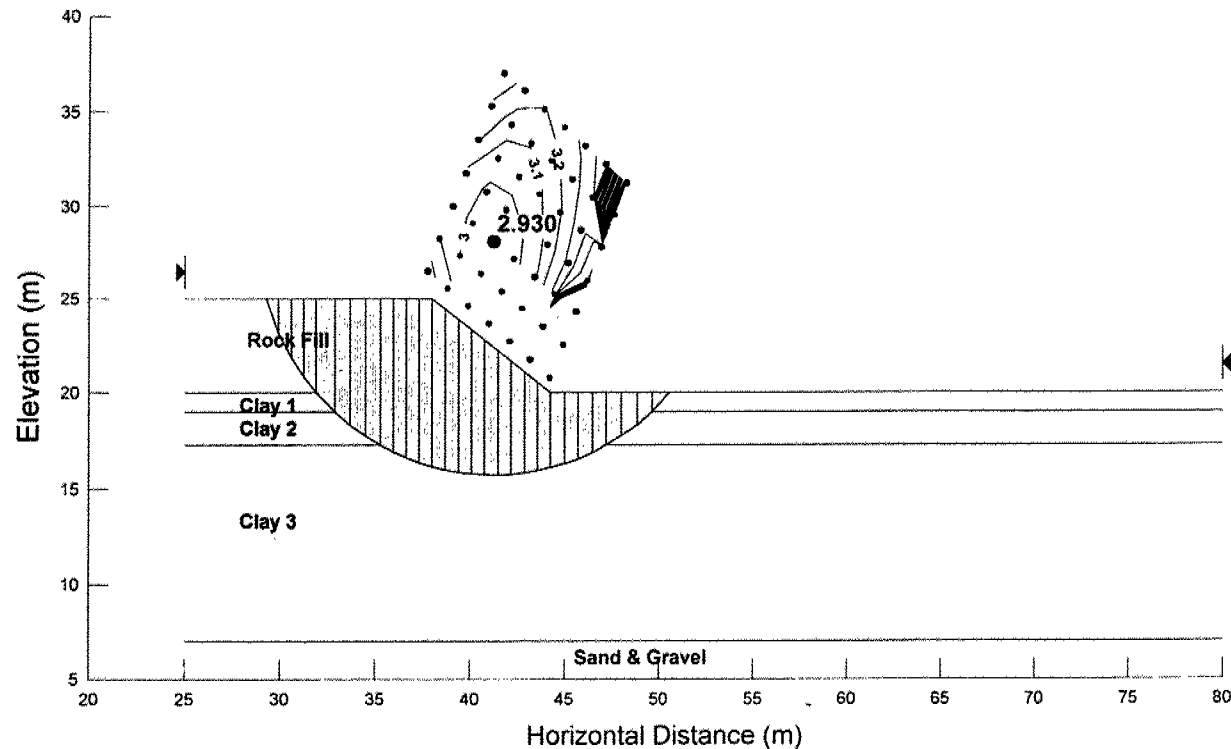
Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 100  
 Rate of Increase -30  
 Cv - Minimum 70  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 70  
 Rate of Increase -28.57  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Clay 3  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 50  
 Rate of Increase 1.95  
 Cv - Maximum 70  
 Ch/Cv Ratio 1

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39



Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-B/G)  
 North Abutment  
 6 metre embankment height, 1.25:1 side slopes  
 NS\_N6H.SLP

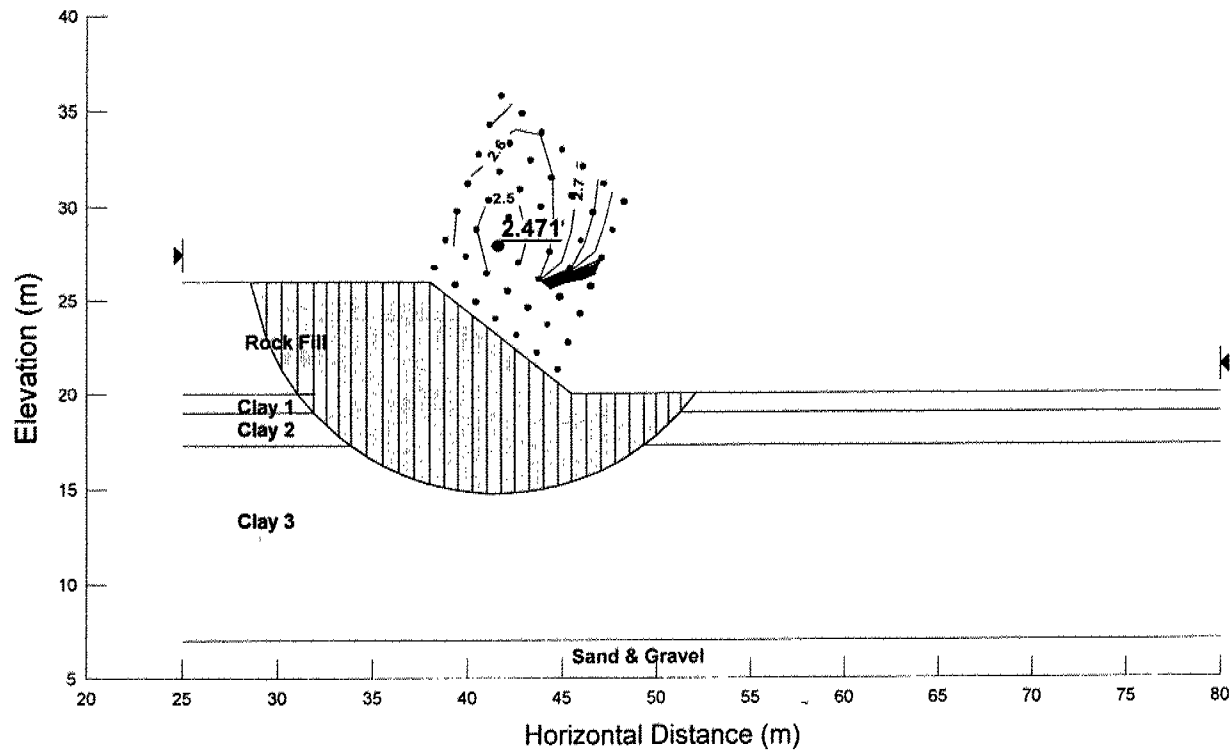
Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 100  
 Rate of Increase -30  
 Cv - Minimum 70  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 70  
 Rate of Increase -28.57  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Clay 3  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 50  
 Rate of Increase 1.95  
 Cv - Maximum 70  
 Ch/Cv Ratio 1

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39



Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-B/G)  
 North Abutment  
 7 metre embankment height, 1.25:1 side slopes  
 4 metre high, 2 metre wide bench  
 NS\_N7H.SLP

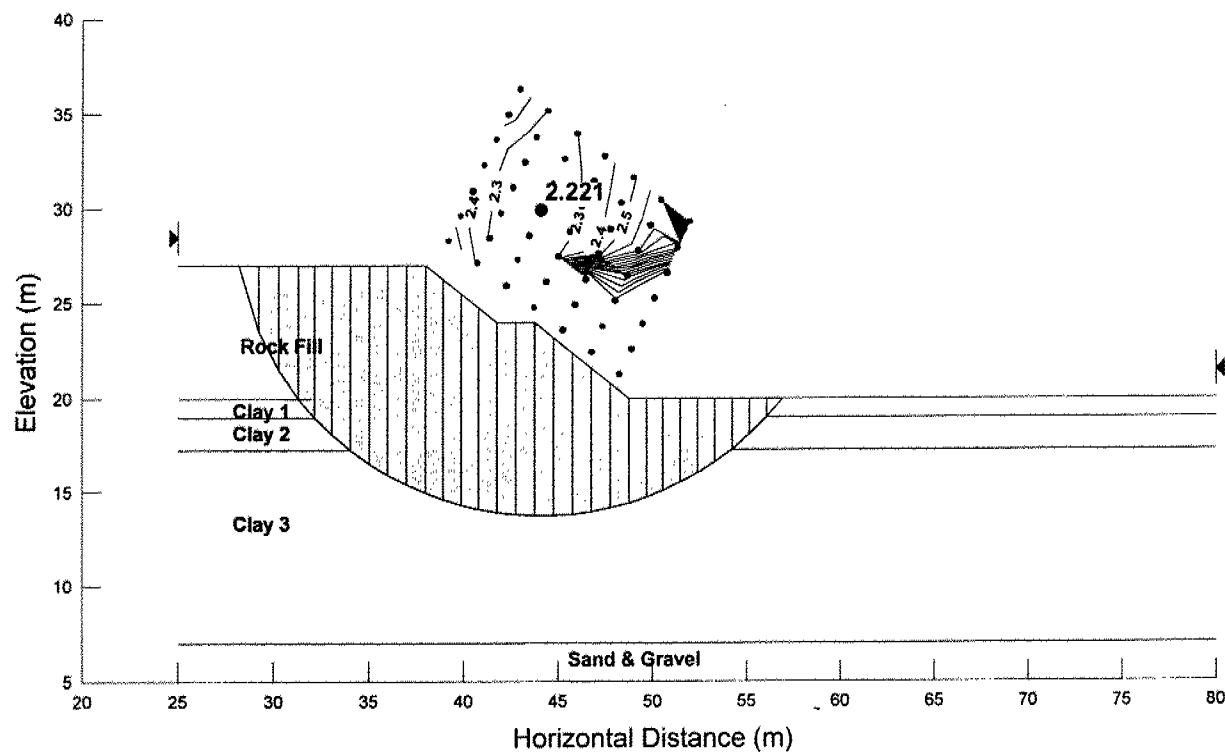
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 Unit Weight 20  
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 Phi 42

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 Unit Weight 19  
 Cv 100  
 Rate of Increase -30  
 Cv - Minimum 70  
 Ch/Cv Ratio 1

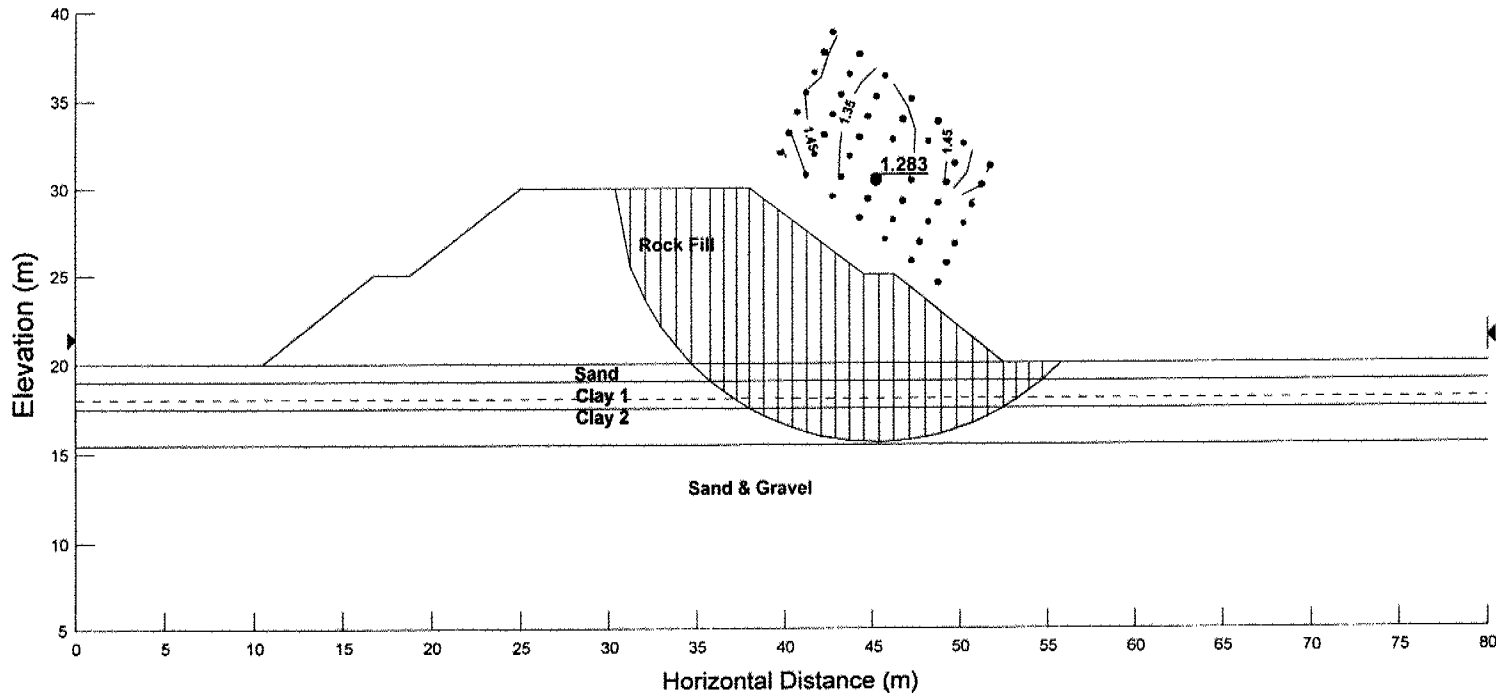
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 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 70  
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 Cv - Minimum 50  
 Ch/Cv Ratio 1

Clay 3  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 50  
 Rate of Increase 1.95  
 Cv - Maximum 70  
 Ch/Cv Ratio 1

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39



Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-B/G)  
 Northbound Lane, South Abutment  
 10 metre embankment height, 1.25:1 side slopes  
 5 metre high, 2 metre long bench  
 N\_S10H.SLP



Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

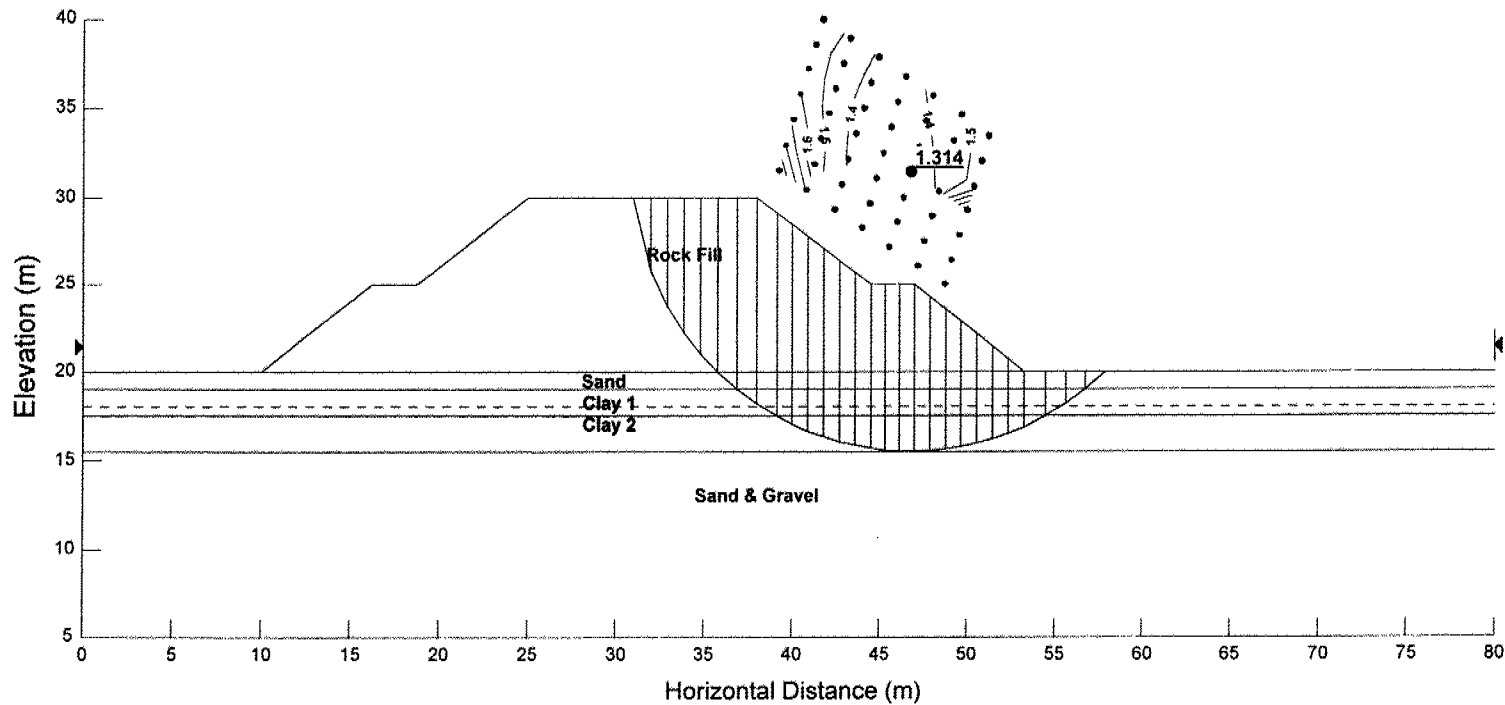
Sand  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 32

Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 60  
 Rate of Increase -16.67  
 Cv - Minimum 35  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 19  
 Cohesion 35  
 Phi 0

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39

Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-B/G)  
 Northbound Lane, South Abutment  
 10 metre embankment height, 1.25:1 side slopes  
 5 metre high, 2.5 metre long bench  
 N\_S10H1.SLP



Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

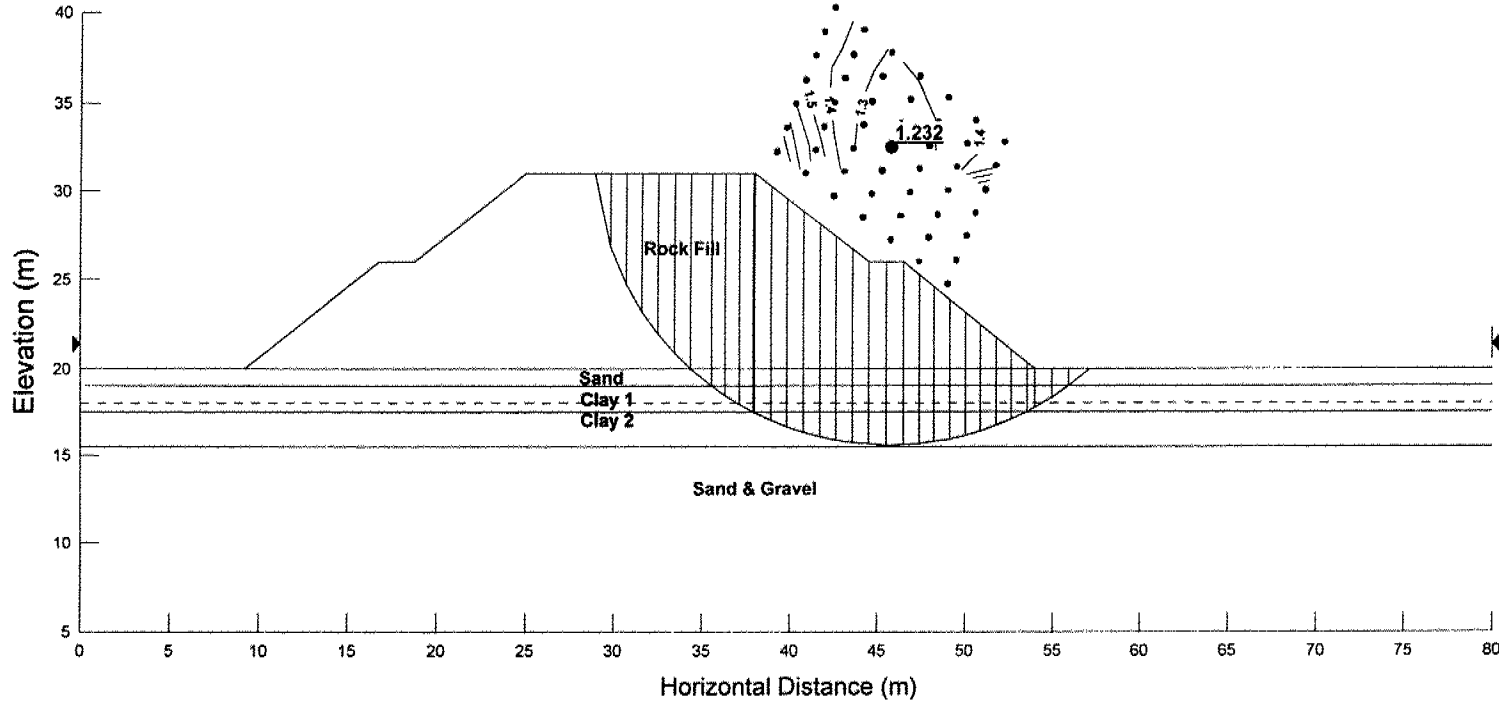
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 Cv 60  
 Rate of Increase -16.67  
 Cv - Minimum 35  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 19  
 Cohesion 35  
 Phi 0

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39

Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-B/G)  
 Northbound Lane, South Abutment  
 11 metre embankment height, 1.25:1 side slopes  
 6 metre high, 2 metre long bench  
 N\_S11H.SLP



Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

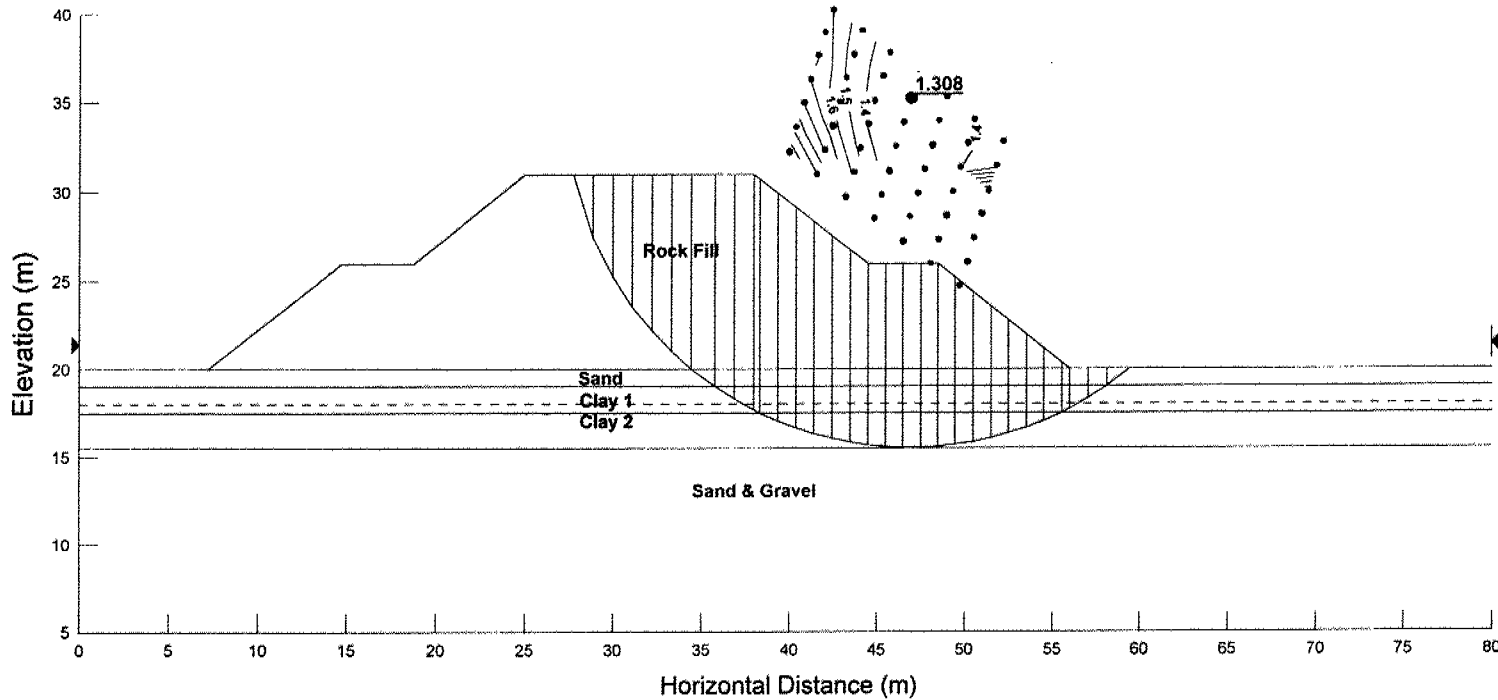
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 Unit Weight 20  
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 Phi 32

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 Soil Model S=f(depth)  
 Unit Weight 19  
 Cv 60  
 Rate of Increase -16.67  
 Cv - Minimum 35  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 19  
 Cohesion 35  
 Phi 0

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39

Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-B/G)  
 Northbound Lane, South Abutment  
 11 metre embankment height, 1.25:1 side slopes  
 6 metre high, 4 metre long bench  
 N\_S11H1.SLP



Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Sand  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 32

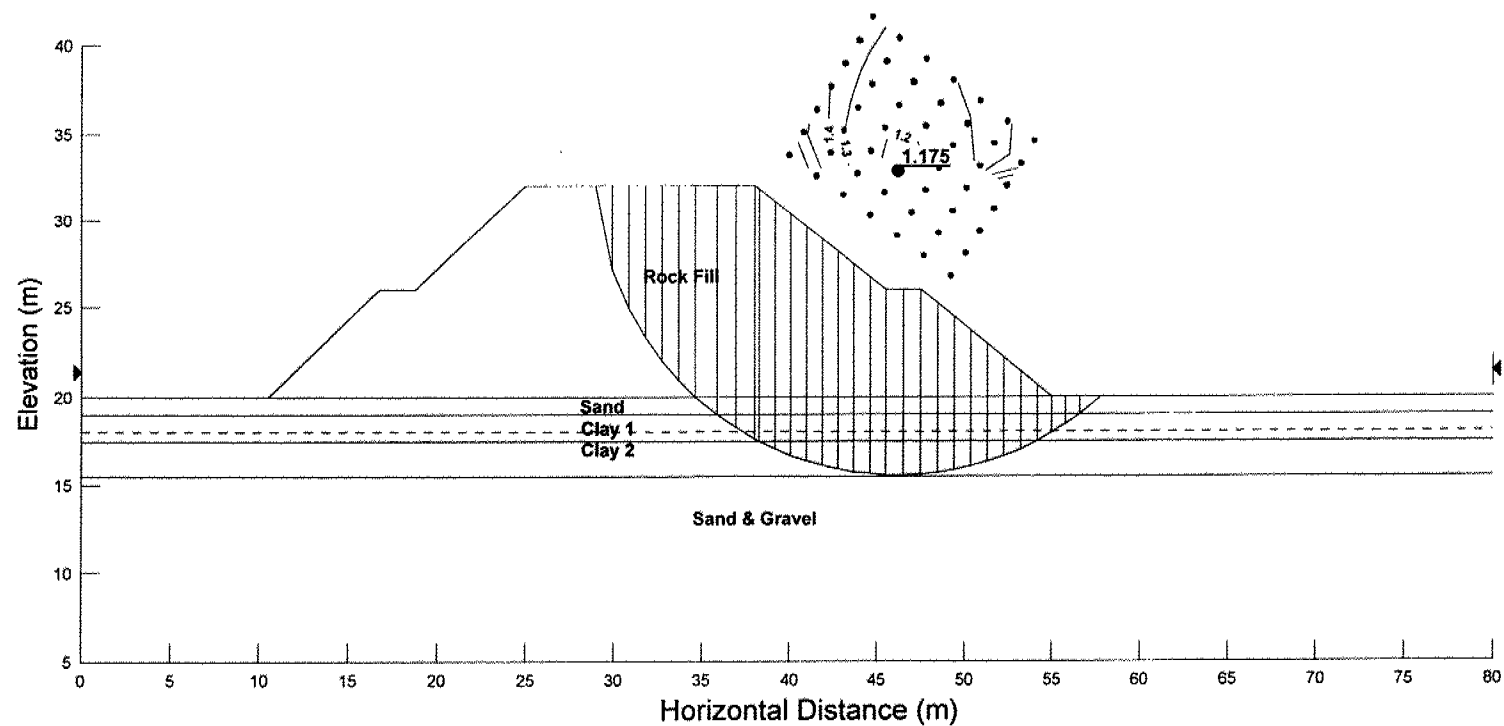
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 Unit Weight 19  
 Cv 60  
 Rate of Increase -16.67  
 Cv - Minimum 35  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 19  
 Cohesion 35  
 Phi 0

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39



Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-B/G)  
 Northbound Lane, South Abutment  
 12 metre embankment height, 1.25:1 side slopes  
 6 metre high, 2 metre long bench  
 N\_S12H.SLP



Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

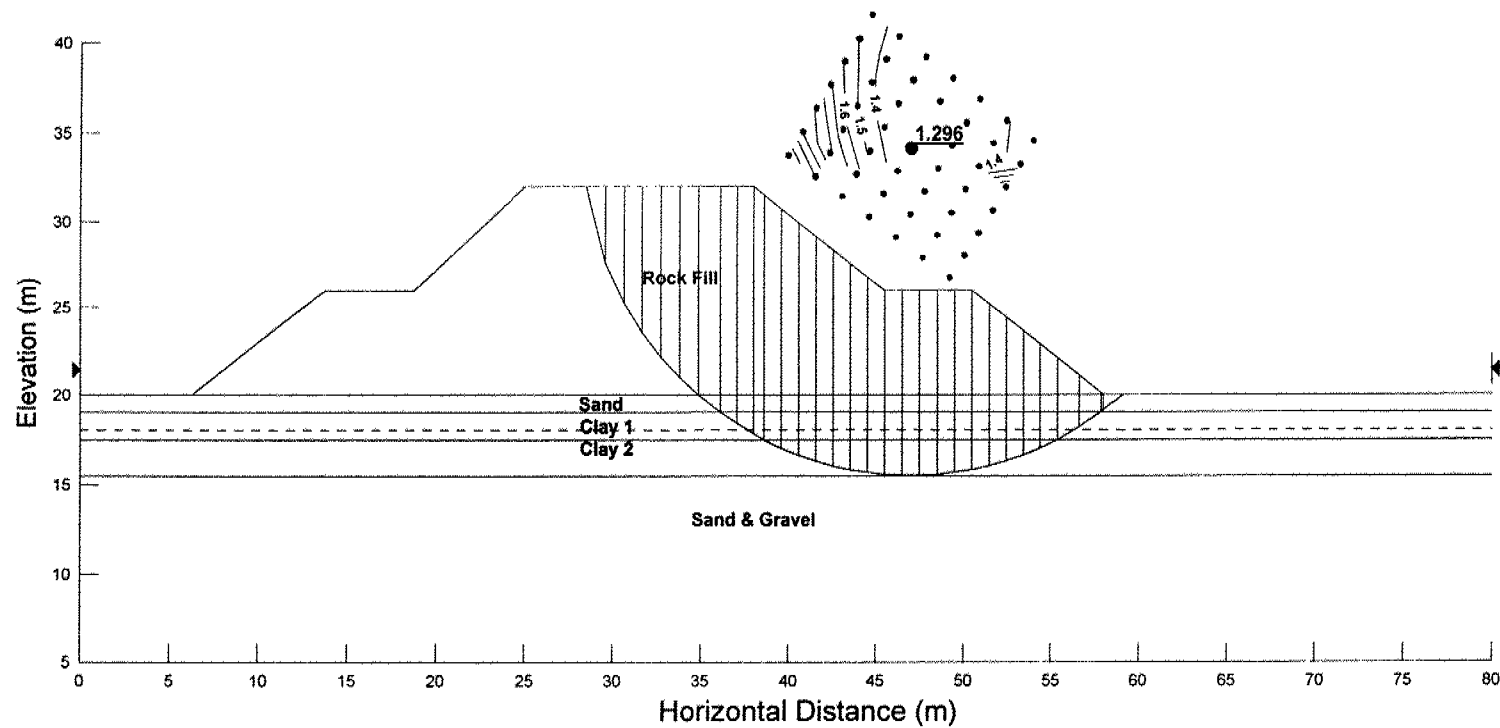
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 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 32

Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 60  
 Rate of Increase -16.67  
 Cv - Minimum 35  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 19  
 Cohesion 35  
 Phi 0

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39

Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-B/G)  
 Northbound Lane, South Abutment  
 12 metre embankment height, 1.25:1 side slopes  
 6 metre high, 5 metre long bench  
 N\_S12H1.SLP



Rock Fill  
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 Unit Weight 20  
 Cohesion 0  
 Phi 42

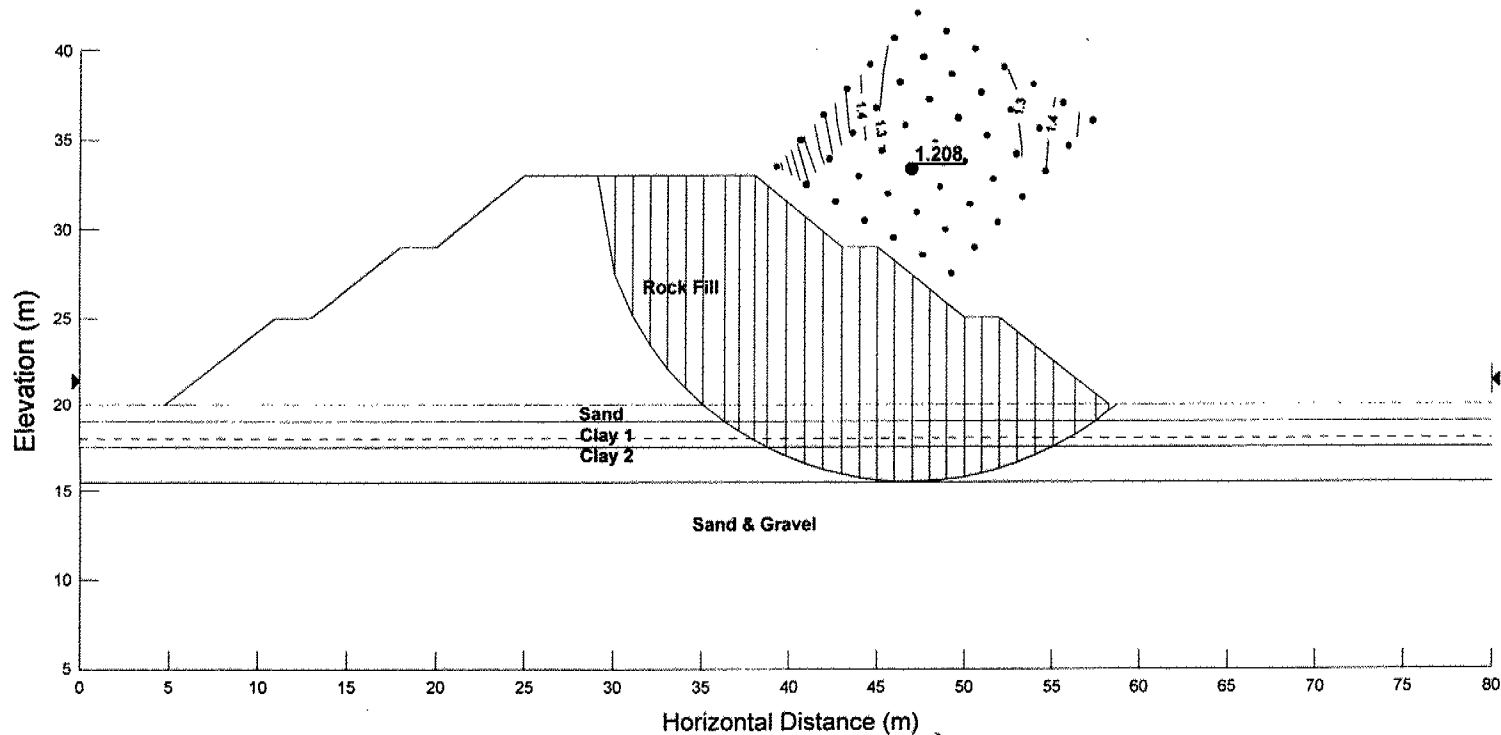
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 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 32

Clay 1  
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 Unit Weight 19  
 Cv 60  
 Rate of Increase -16.67  
 Cv - Minimum 35  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 19  
 Cohesion 35  
 Phi 0

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39

Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-B/G)  
 Northbound Lane, South Abutment  
 13 metre embankment height, 1.25:1 side slopes  
 5 metre high and 4 metre high, 2 metre long benches  
 N\_S13H.SLP



Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Sand  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 32

Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 60  
 Rate of Increase -16.67  
 Cv - Minimum 35  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 19  
 Cohesion 35  
 Phi 0

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39

Slope Stability - Total Stress Analysis  
 Trout Creek - Highway 11 (F-98179-B/G)  
 Northbound Lane, South Abutment  
 13 metre embankment height, 1.25:1 side slopes  
 5 metre high, 4 metre long and 4 metre high, 2 metre long benches  
 N\_S13H1.SLP

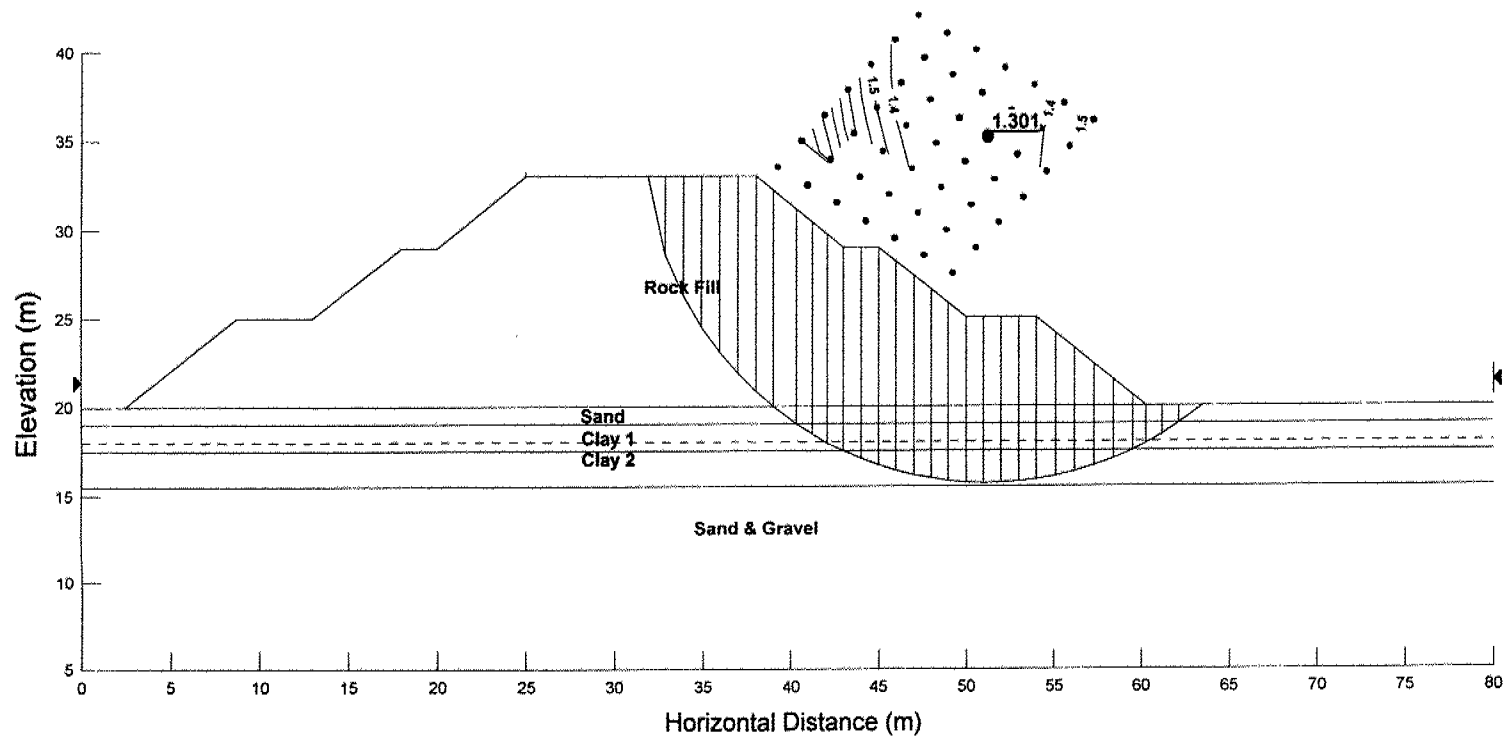
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 Unit Weight 20  
 Cohesion 0  
 Phi 42

Sand  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 32

Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 19  
 Cv 60  
 Rate of Increase -16.67  
 Cv - Minimum 35  
 Ch/Cv Ratio 1

Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 19  
 Cohesion 35  
 Phi 0

Sand & Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 39



DOCUMENT MICROFILMING IDENTIFICATION

G.I.-30 SEPT. 1976

GEOCRES No. \_\_\_\_\_

DIST. 54 REGION \_\_\_\_\_

W.P. No. 774-93-00

CONT. No. \_\_\_\_\_

W. O. No. \_\_\_\_\_

STR. SITE No. \_\_\_\_\_

HWY. No. 11

LOCATION Trout CREEK By-PASS

North Bound & South Bound LANES

No. of PAGES -

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. \_\_\_\_\_

REMARKS: \_\_\_\_\_

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**Geotechnical Foundation Report  
Embankment Section  
Trout Creek ByPass  
North Bound & Sound Bound Lanes  
Station 12+350 to 12+850  
GWP No. 774-93-00  
District 54, Sudbury**

Prepared for:

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Thornhill, ON  
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SO7524G/IF  
March, 1999

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Grading Analyses .....	Figures 1 - 3
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**SO7524G/IF**

**MEMORANDUM NO. 2**

**TO:** R.D. Kivi, P.Eng.  
Senior Project Manager  
Marshall, Macklin, Monaghan

**FROM:** I.W. Gore, P.Eng.  
Principal Engineer  
Trow Consulting Engineers Ltd.

E.A. Gonneau, P.Eng.  
Project Manager  
Trow Consulting Engineers Ltd.

**SUBJECT:** Geotechnical Foundation Report  
Embankment Section  
Trout Creek ByPass, North Bound & South Bound Lanes  
Station 12+350 to 12+850  
GWP No. 774-93-00  
District 54 Sudbury

---

This memorandum addresses the geotechnical report for the design and construction of an embankment section between approximate stations 12+350 to 12+850 along the proposed NBL and SBL of the proposed Trout Creek ByPass, Highway 11, as part of GWP 774-93-00. The approximately 500 m long embankment is located from the existing McCarthy Street, extending southwesterly to the section where the existing ground rises, and the ByPass enters a "cut" section.

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## 1.0 Introduction

This embankment section for the NBL and SBL of the proposed Trout Creek ByPass, Highway 11, runs from approximate station 12+350 up to approximate station 12+850, a distance of 500 m.

In the initial section, i.e. between stations 12+600 to 12+850, the terrain is flat and contains many tag-alders within the existing ground level at approximate elevation 315 m. In this area, the embankment is approximately 4 m high at station 12+800, rising to 8 m high at station 12+600. Beyond station 12+850, up to the present McCarthy Street at station 13+750, the proposed embankment height is less than 4 m, and the terrain is flat and open.

The existing ground is level, southwesterly from station 12+600, then rises gradually from the flat land at elevation 315 m through densely wooded terrain, up to approximate elevation 324 m at station 12+470. Correspondingly, the proposed NBL and SBL embankment grades also rise to elevation 327 m at this location. Southwesterly from station 12+325, the existing grade then rises steeply, and the proposed NBL and SBL enter into cut, at approximate station 12+300.

## 2.0 Field Work

The field work comprised thirteen (13) sampled boreholes (BH's 11F to 13IF, inclusive), together with five additional dynamic cone penetration tests driven adjacent to boreholes 11F, 6IF, 11IF, 12IF and 13IF. The locations of the boreholes are included on the attached site plan, Drawing 1, and on the individual borehole logs. The drilling was completed using a track-mounted soils drill equipped with hollow and solid stem flight augers between September 14 to 17, 1998, inclusive.

Details of the soil strata encountered in the boreholes and cones are included in the attached logs, and plotted on the profile included on Drawing 1. Further information on soil descriptions are contained on Drawings 2A and 2B.

### 3.0 Subsurface Conditions

#### 3.1 Station 12+600 to 12+850 (BH's 5IF to 12IF)

Based on the borehole data, the subsoil conditions along this flat terrain portion are reasonably uniform and consist of the following soil strata:

- *Organics*

The organics comprise a thin, surficial deposit of topsoil generally 100 mm to 300 mm thick. At borehole 11IF, however, the topsoil veneer is locally thicker, i.e. 600 mm.

- *Sand*

The predominant soil stratum throughout this section consists of sand, which has a generally loose to compact consistency. The deposit is at least 6 m thick.

Based on the laboratory gradation analyses (see attached data), the sand is predominantly fine-grained with less than 10% silt and clay sizes.

Typical standard penetration test results ("N" values) vary from 2 to 27 (average 12). The lower "N" values (less than 5) are not considered to be representative, since sampling disturbance likely occurred during the drilling and in-situ testing. It is noted that the dynamic cone penetration tests (cones driven adjacent to borehole 6IF, 11IF, 12IF and 13IF) confirm consistent blows, exceeding 20 blows/300 mm penetration.

Moisture contents of the fine sand are in the range of 18% to 22%.

- *Clayey Silt*

A localized deposit of clayey silt was encountered underlying the sand in boreholes 6IF and 11IF.

In borehole 6IF, the clayey silt deposit is less than 2.5 m thick and occurs at depths between 6.5 m (~El. 308.5 m) and 8.8 m (~El. 300.2 m). In borehole 11IF, the deposit is slightly thicker, i.e. 3.6 m

thick, and occurs at depths between 5.5 m (~El. 310.4 m) and 9.1 m (~El. 306.8 m). The clayey silt is stratified with intermittent, horizontal silt layers and undrained shear strengths (as measured with in-situ vanes and laboratory shear tests) varying from 22 kPa to 30 kPa.

Natural moisture contents range from 40% to 55%, with Atterberg limits of 22% (plastic limit) to 38% and 42% (liquid limit).

The clayey silt is similar in characteristics and properties to the clayey deposit encountered at the adjacent North Interchange. For the North Interchange, extensive detailed laboratory analyses have been undertaken.

- *Silt*

A lower zone of loose to compact silt was encountered in borehole 11IF below the clayey silt. This silt stratum was intercepted at a depth of 9.2 m (~El. 306.8 m) and continues for approximately 4.5 m, i.e. down to a depth of 13.7 m (~El. 302.2 m). Standard penetration blows varied from 3 to 9 blows/300 mm, indicating a loose to compact consistency. Dynamic cone blows, through the deposit, exceed 40 blows/300 mm penetration. Laboratory shear strength tests and a field vane test (completed in "clayey" seams) confirmed undrained shear strength values exceeding 45 kPa. In-situ moisture contents of the deposit are in the range 25% to 35% with one Atterberg limit test confirming a plastic limit of 18% and a liquid limit of 28%.

- *Silty Sand and Gravel Till*

A 2 m thick zone of dense silty sand and gravel till ("N" blows >40) was encountered at the base of borehole 5IF, i.e., below the upper compact sand.

### **3.2 Stations 12+350 to 12+600 (BH's 1IF to 4IF)**

As the ground rises in this section, the upper veneer of sand is thinner and the lower levels of the boreholes encountered a dense glacial till. The properties and sequence of soil deposits is described below:

- *Organics*

The organics in this section are thin and comprise less than 100 mm of topsoil.

- *Sand*

The sand is generally a fine-grained deposit with traces of silt and occasional gravel inclusions. The thickness of the sand varies from less than 2.5 m in boreholes 3IF and 4IF to 5.4 m in borehole 1IF.

Standard penetration tests established "N" blows of 6 to 16, confirming a loose to compact condition. In-situ moisture contents are less than 20%.

- *Clayey Silt*

A localized, approximately 1.5 m thick, pocket of clayey silt was intercepted in borehole 1IF at a depth of 5.4 m (~El. 317.6 m). The clayey silt has the following characteristics:

- undrained shear strength ~50 kPa (laboratory shear test)
- in-situ moisture content - 36%
- Atterberg limits      Plastic Limit - 21%  
                                 Liquid Limit - 33%
- "N" value - 5 blows/300 mm
- dynamic cone penetration blows - >20 blows/300 mm

- *Sand and Gravel Till*

Sand and gravel till was encountered at the base of the boreholes and varies in thickness from about 1.0 m in borehole 3IF to slightly greater than 3.5 m thick in borehole 2IF.

Standard penetration blows ("N" values) are in the range of 15 to 50 blows/300 mm penetration (average 30 blows), with in-situ moisture contents of 8% to 20%.

## 4.0 Groundwater

In the embankment portion where the existing terrain is flat and poorly drained (boreholes 5IF to 13IF, stations 12+600 to 12+850), the groundwater table is at grade.

In the higher terrain (boreholes 1IF to 4IF, station 12+350 to 12+600), the groundwater table is slightly lower, i.e. at a depth of 1 m to 2 m below grade.

Seasonal fluctuations in the level of the groundwater table can be expected.

## 5.0 Recommendations

The proposed NBL and SBL will require embankment heights of up to about 8 m.

Since the majority of the subsoil is competent, granular soils (loose to compact sands and/or dense till), no stability nor long-term consolidation problems are envisaged. However, where the localized clayey silt deposit was encountered, estimated to be within a localized section beneath the NBL and SBL between stations 12+615 and 12+645, some precautionary measures must be considered. In these sections, the following procedures are recommended:

- *Stability*

The embankment height(s) in this localized section is 7.5 m. The soil strata consists of 5.5 m of compact sand over 3.6 m of loose to compact, clayey silt. The undrained shear strength of the clayey silt is 22 kPa (minimum). Although the factor of safety, under these conditions, meets or exceeds 1.3, (see stability analysis, Figure 4 to 6), it is nevertheless recommended that the embankment be constructed gradually, increasing the height over a minimum of 3 months.

- *Settlement*

The calculated settlement of the embankments, due to consolidation of the 3.7 m thick zone of clayey silt, is estimated to be in the order of 150 mm. However, 50% of this settlement will be completed within 3 months and 90% within a period of 7 months, after full height has been reached. As such,

all the anticipated consolidation settlement should occur within 8 months, once the embankment has been constructed to full height.

## 6.0 Embankment Design

The proposed embankments are 8 m in height or less. Since only fairly minor consolidation settlements are expected, i.e. a maximum of 150 mm within one localized section, which, in any event, should occur within 8 months after construction, it is recommended that the embankments be constructed either with rock fill having a side slope of 1.25H:1V, allowing for a 2.0 m wide, mid-height bench where the height exceeds 6 m, or the bouldery material contained within the cut area to the south of Trout Creek, having a side slope of 2.5H:1V, allowing for a 3.0 m wide, mid-height bench.

Settlement within the rock fill or till fill embankment itself should be expected over the first 2 to 3 years after construction. It is likely that this movement could be in the order of 1% to 2% of the height, i.e. up to about 150 mm beneath the 8 m high embankment section. This settlement could, however, be reduced significantly (to less than 50 mm) if the rock fill was restricted in size to a maximum of 600 mm diameter and then thoroughly compacted, using a 10 ton vibratory drum roller, with four or five passes in 1 m lifts of rock fill.

To allow for future potential grade raises, the rock fill embankments should be constructed a minimum of 1 m wider than standard on each side, to the bottom of the subgrade, in the section between stations 12+615 and 12+645.

## 7.0 Construction Considerations

The upper organic veneer 200 mm (average) of topsoil should be stripped off down to firm bottom before placement of the fill.




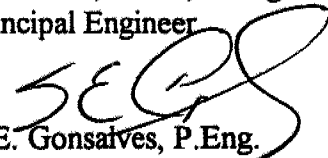
## 8.0 Closure

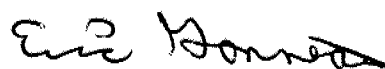
The field investigations were supervised by Mr. E.A. Gonneau, P.Eng., Project Manager. The memorandum report was written by Mr. I.W. Gore, M.Sc., P.Eng., Principal Geotechnical Engineer, and reviewed by Mr. S.E. Gonsalves, P.Eng.

Yours truly,

**TROW CONSULTING ENGINEERS LTD.**

  
I.W. Gore, M.Sc., P.Eng.  
Principal Engineer

  
S.E. Gonsalves, P.Eng.  
Vice-President

  
E.A. Gonneau, P.Eng.  
Project Manager

Encl.  
Dist:

**NOTES ON SAMPLE DESCRIPTIONS**

1. All descriptions included in this report follow the I.S.S.M.F.E. as suggested in the Canadian Foundation Manual. The laboratory grain-size analysis also follows this classification system. Others may designate the unified classification system as their source; a comparison of the two is shown for your information. Please note that, with the exception of those samples where the grain-size analysis has been carried out, all samples are classified visually and the accuracy of visual examination is not sufficient to differentiate between the classification systems or exact grain sizing.

UNIFIED SOIL CLASSIFICATION	Fines (silt or clay)			Sand			Gravel		Cobbles																						
				Fine	Medium	Coarse	Fine	Coarse																							
I.S.S.H.F.E. SOIL CLASSIFICATION	Clay	Silt			Sand			Gravel			Cobbles																				
		Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse																					
	Sieve Sizes																														
	Particle Size (mm)																														
	0.001	0.002	0.003	0.004	0.006	0.008	0.01	0.02	0.03	0.04	0.06	0.08	0.1	0.2	0.3	0.4	0.5	0.6	0.8	1.0	2.0	3.0	4.0	6.0	8.0	10	20	30	40	60	80
														-200																	

2. **FILL:** Where fill is designated on the borehole log, it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of the site fill material. All fills should be expected to contain obstructions such as large concrete pieces of subsurface basements, floors, tanks, etc.; none of these may have been encountered in the borehole. Since boreholes cannot accurately define the contents of fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact and correct composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant on-going and future settlements. Some fill material may be contaminated by toxic waste that renders it unacceptable for deposition in any but designated land fill sites. Unless specifically stated, the fill on this site has not been tested for contaminants that may be considered hazardous. This testing and a potential hazard study can be carried out if you so request. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common but are not detectable using conventional geotechnical procedures.
3. **TILL:** The term till on the borehole logs indicate that the material originates from a geological process associated with glaciation. As a result of this geological process, the till must be considered heterogeneous in composition and, as such, may contain pockets and/or seams of material such as sand, gravel silt or clay. As till often contains cobbles (60 to 200 mm) or boulders (over 200 mm), contractors may encounter them during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size, or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited areas; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till material.



# NOTES ON SAMPLE DESCRIPTIONS (Cont'd)



Project No: S07524G/IP

Drawing No: 2B

4. The following table gives a description of the soil based on particle sizes. With the exception of those samples where grain-size analyses have been performed, all samples are classified visually. The accuracy of visual examination is not sufficient to differentiate between this classification system or exact grain size.

Soil Classification		Terminology	Proportion
Clay	< 0.002 mm	"trace" (eg. trace sand)	1% - 10%
Silt	0.002 to 0.06 mm	"some" (eg. some sand)	10% - 20%
Sand	0.06 to 2 mm	adjective (eg. sandy)	20% - 35%
Gravel	2 to 60 mm	and (eg. and sand)	> 35%
Cobbles	60 to 200 mm	noun (eg. boulders)	> 35% and
Boulders	> 200 mm		main fraction

Classification system as suggested in the Canadian Foundation Engineering Manual, 3rd Edition, unless otherwise noted.

The compactness of cohesionless soils and the consistency of cohesive soils are defined by the following:

Cohesionless Soil		Cohesive Soil	
Compactness	Standard Penetration Resistance "N" Blows/0.3 m	Consistency	Undrained Shear Strength (kPa)
Very Loose	0 to 4	Very Soft	< 12
Loose	4 to 10	Soft	12 - 25
Compact	10 to 30	Firm	25 - 50
Dense	30 to 50	Stiff	50 - 100
Very Dense	Over 50	Very Stiff	100 - 200
		Hard	> 200

## 5. Rock Coring

Where rock drilling was carried out, the term RQD (Rock Quality Designation) is used. The RQD is an indirect measure of the number of fractures and soundness of the rock mass. It is obtained from the rock cores by summing the length of core recovered, counting only those pieces of sound core that are 100 mm or more in length. The RQD value is expressed as a percentage and is the ratio of the summed core lengths to the total length of core run. The classification based on the RQD value is given below.

RQD Classification	RQD
Very poor quality	< 25
Poor quality	25 - 50
Fair quality	50 - 75
Good quality	75 - 90
Excellent quality	90 - 100

$$\text{Recovery Designation \% Recovery} = \frac{\text{Length of Core Per Run} \times 100}{\text{Total Length of Run}}$$

# RECORD OF BOREHOLE BH-11F 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station 12+387, on centreline of Northbound Lane  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / Dynamic cone  
 DATUM Geodetic DATE September 14, 1998

ORIGINATED BY S.M.  
 COMPILED BY M.D.  
 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT wp	NATURAL MOISTURE CONTENT w	LIQUID LIMIT wl	UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20	40	60	80					
323.00 0.00	GROUND SURFACE						323									
	TOPSOIL, 50 mm over SAND, fine to medium grading to fine silty sand with depth, brown then grey below 3.5 m depth, moist then wet below 2 m depth. (loose to compact)		1	SS	8		322									0% 92% 8%
			2	SS	16		320									
			3	SS	13		318									
317.60 5.40	CLAYEY SILT, grey, wet. (loose)		4	SS	5		317									
316.00 7.00	SILTY SAND & GRAVEL TILL, brown. (compact)		5	SS	14		315									
314.92 8.08	END OF BOREHOLE						314									
313.25 9.75	Probable TILL															
	END OF CONE TEST DUE TO 'BOUNCING' REFUSAL ON PROBABLE BEDROCK OR POSSIBLE BOULDER															
	Notes: 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Water level was at 2.3 m & hole was open to 5.1 m depth on completion. 3) Dynamic cone penetration test driven adjacent BH-11F.															



# RECORD OF BOREHOLE BH-2IF 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station ~12+450, on centreline of Northbound Lane ORIGINATED BY S.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / COMPILED BY M.D.  
 DATUM Geodetic DATE September 14, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT				UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	20	40	60	80	wp	w		
321.30	GROUND SURFACE														
0.00	TOPSOIL, ~75 mm over SAND, fine with SILT content, occasional stiff CLAYEY SILT layers, brown then grey below ~3 m depth, wet. (loose to compact)		1	SS	6										
			2	SS	10										
317.30	SILTY SAND & GRAVEL TILL, some cobble sizes, grey. (compact to dense)		3	SS	27										
4.00			4	SS	44										
313.98	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER														
7.32	Notes: 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Water level was at ~2.0 m & hole was open to ~4.1 m depth on completion.														



# RECORD OF BOREHOLE BH-31F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 12+495, on centreline of Median ORIGINATED BY S.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / COMPILED BY M.D.  
 DATUM Geodetic DATE September 14, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60	80	wp	w	wl		
320.00	GROUND SURFACE															
0.00	TOPSOIL, 50 mm over SAND, fine, brown, moist. (compact)															
318.60																
1.40	SAND & GRAVEL TILL, brown, moist. (dense)		1	SS	42											
317.71																
2.29	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER															
Notes: 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Borehole was dry & open to 0.8 m depth on completion.																



# RECORD OF BOREHOLE BH-4IF 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station ~12+550, on centreline of Median ORIGINATED BY S.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / COMPILED BY M.D.  
 DATUM Geodetic DATE September 14, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE		20	40	60	80	wp	w	wl	WATER CONTENT (%)		
318.50	GROUND SURFACE														
0.00	TOPSOIL, ~100 mm over SAND, fine, trace of SILT, brown to grey, wet at base. (loose to compact)														
316.50			1	SS	7										0% 77% 23%
2.00	SILTY SAND & GRAVEL TILL, cobbles & possible boulders, grey. (compact to dense)														
			2	SS	27										
			3	SS	48										
312.75															
5.75	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER														
Notes: 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Water level was at ~2.4 m & hole was open to ~3.1 m depth on completion.															



# RECORD OF BOREHOLE BH-51F 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~12+600, on centreline of Median ORIGINATED BY S.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / COMPILED BY M.D.  
 DATUM Geodetic DATE September 14, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20	40	60	80	wp	w	wl		
316.40	GROUND SURFACE														
0.00	TOPSOIL, 100 mm over SAND, fine, trace of SILT, brown, moist to wet. (compact to dense)					316									
			1	SS	27	315									
313.90	SILTY SAND & GRAVEL TILL, grey. (dense)					314									
2.50			2	SS	48	313									
311.98	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER					312									
4.42	Notes: 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Water level was at ~0.6 m & hole was open to ~2.2 m depth on														



# RECORD OF BOREHOLE BH-6IF 1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station 12+650, on centreline of Median

ORIGINATED BY S.M.

DIST 54 HWY 11

BOREHOLE TYPE Standard augers / Dynamic cone

COMPILED BY M.D.

DATUM Geodetic

DATE September 17, 1998

CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)		CONE PENETRATION TEST		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR	wp	wl	wp	wl	wp	wl	wp		
315.00	GROUND SURFACE																	
0.00	PEATY TOPSOIL, ~200 mm over SAND, mostly fine with a trace of SILT & occasional GRAVEL sizes, grey brown, wet. (loose to compact)		1	SS	7													
			2	SS	13													
			3	SS	2													
308.50			4	SS	0													
6.50	CLAYEY SILT, grey, wet. (loose)		5	SS	4													
306.16	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER																	
8.84	Notes: 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Water level was at ~0.2 m & hole was open to ~4.0 m depth on completion. 3) Dynamic cone penetration test driven adjacent BH-6IF.																	



# RECORD OF BOREHOLE BH-7IF 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station +12+700, on centreline of Median ORIGINATED BY S.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / COMPILED BY M.D.  
 DATUM Geodetic DATE September 17, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT KN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	20	40	60	80	wp		
315.00	GROUND SURFACE				315									
0.00	PEATY TOPSOIL, ~300 mm over SAND, fine with a trace of SILT content, grey, wet. (loose to compact)													
			1	SS	4									
			2	SS	17									
			3	SS	20									
			4	SS	27									
308.45	END OF BOREHOLE													
6.55	Notes: 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Water level was at ~0.1 m & hole was open to ~1.8 m depth on completion.													





# RECORD OF BOREHOLE BH-8IF

1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station ~12+750, on centreline of Median

ORIGINATED BY S.M.

DIST 54 HWY 11

BOREHOLE TYPE Standard augers /

COMPILED BY M.D.

DATUM Geodetic

DATE September 17, 1998

CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)				CONE PENETRATION TEST				PLASTIC LIMIT			NATURAL MOISTURE CONTENT			LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	20	40	60	80	20	40	60	80	wp	w	wl	10	20	30	40		
315.00	GROUND SURFACE																							
0.00	PEATY TOPSOIL, ~300 mm over SAND, mostly fine with a trace of SILT, brown to grey, wet. (compact)		1	SS	10																			
			2	SS	15																			
			3	SS	13																			
			4	SS	26																			
308.45	END OF BOREHOLE																							
6.55	Notes: 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Water level was at ~0.1 m & hole was open to ~2.6 m depth on completion.																							



# RECORD OF BOREHOLE BH-9IF 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station ~12+800, on centreline of Median ORIGINATED BY S.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / COMPILED BY M.D.  
 DATUM Geodetic DATE September 17, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION					
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60	80	wp	w	wl			10	20	30	40	KN/m <sup>3</sup>
315.00	GROUND SURFACE					315															
0.00	TOPSOIL, ~150 mm over SAND, mostly fine with a trace of SILT, grey. (loose to compact)		1	SS	5																
			2	SS	16																
			3	SS	14																
			4	SS	12																
308.45	END OF BOREHOLE																				
6.55	Notes: 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Water level was at ~0.5 m & hole was open to ~1.9 m depth on completion.																				



# RECORD OF BOREHOLE BH-10IF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~12+850, on centreline of Median ORIGINATED BY S.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / COMPILED BY M.D.  
 DATUM Geodetic DATE September 17, 1998 CHECKED BY I.G.

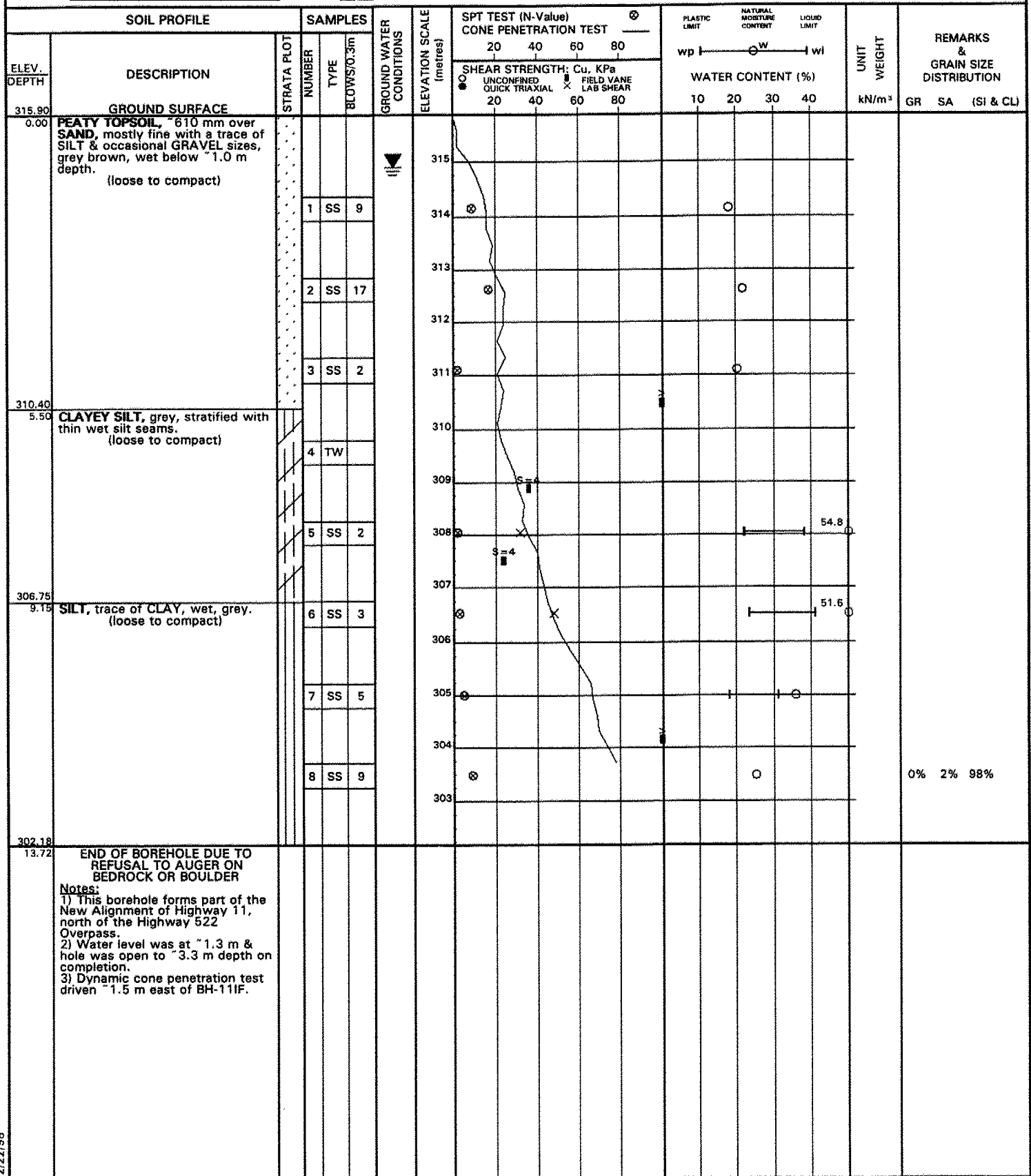
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT				UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION  GR    SA    (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20    40    60    80				wp ——— w ——— wl					
								SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)					
								UNCONFINED QUICK TRIAXIAL		FIELD VANE LAB SHEAR							
								20	40	60	80	10	20	30	40		
314.80	GROUND SURFACE																
0.00	PEATY TOPSOIL, ~300 mm over SAND, mostly fine with a trace of SILT, brown to grey, wet below ~0.6 m depth, (compact)																
			1	SS	8											0% 97% 3%	
			2	SS	15												
			3	SS	14												
			4	SS	12												
308.25 6.55	END OF BOREHOLE																
<b>Notes:</b> 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Water level was at ~0.7 m & hole was open to ~2.3 m depth on completion.																	



# RECORD OF BOREHOLE BH-11IF 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station ~12+630, on centreline of Southbound Lane ORIGINATED BY S.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / Dynamic cone COMPILED BY M.D.  
 DATUM Geodetic DATE September 15, 1998 CHECKED BY I.G.



# RECORD OF BOREHOLE BH-12IF 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station +12+750, on centreline of Southbound Lane ORIGINATED BY S.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / Dynamic cone COMPILED BY M.D.  
 DATUM Geodetic DATE September 15, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	20	40	60	80	wp		
315.30 0.00	GROUND SURFACE													
	PEATY TOPSOIL, ~360 mm over SAND, mostly fine with a trace of SILT, wet below ~0.8 m depth. (compact)		1	SS	14									
			2	SS	14									
			3	SS	11									
			4	SS	12									
			5	SS	10									
307.22 8.08	END OF BOREHOLE													
<b>Notes:</b> 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Borehole caved wet at ~0.9 m depth on completion.														



# RECORD OF BOREHOLE BH-13IF 1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station 12+850, on centreline of Southbound Lane

ORIGINATED BY S.M.

DIST 54 HWY 11

BOREHOLE TYPE Hollow stem augers / Dynamic cone

COMPILED BY M.D.

DATUM Geodetic

DATE September 14, 1998

CHECKED BY I.G.

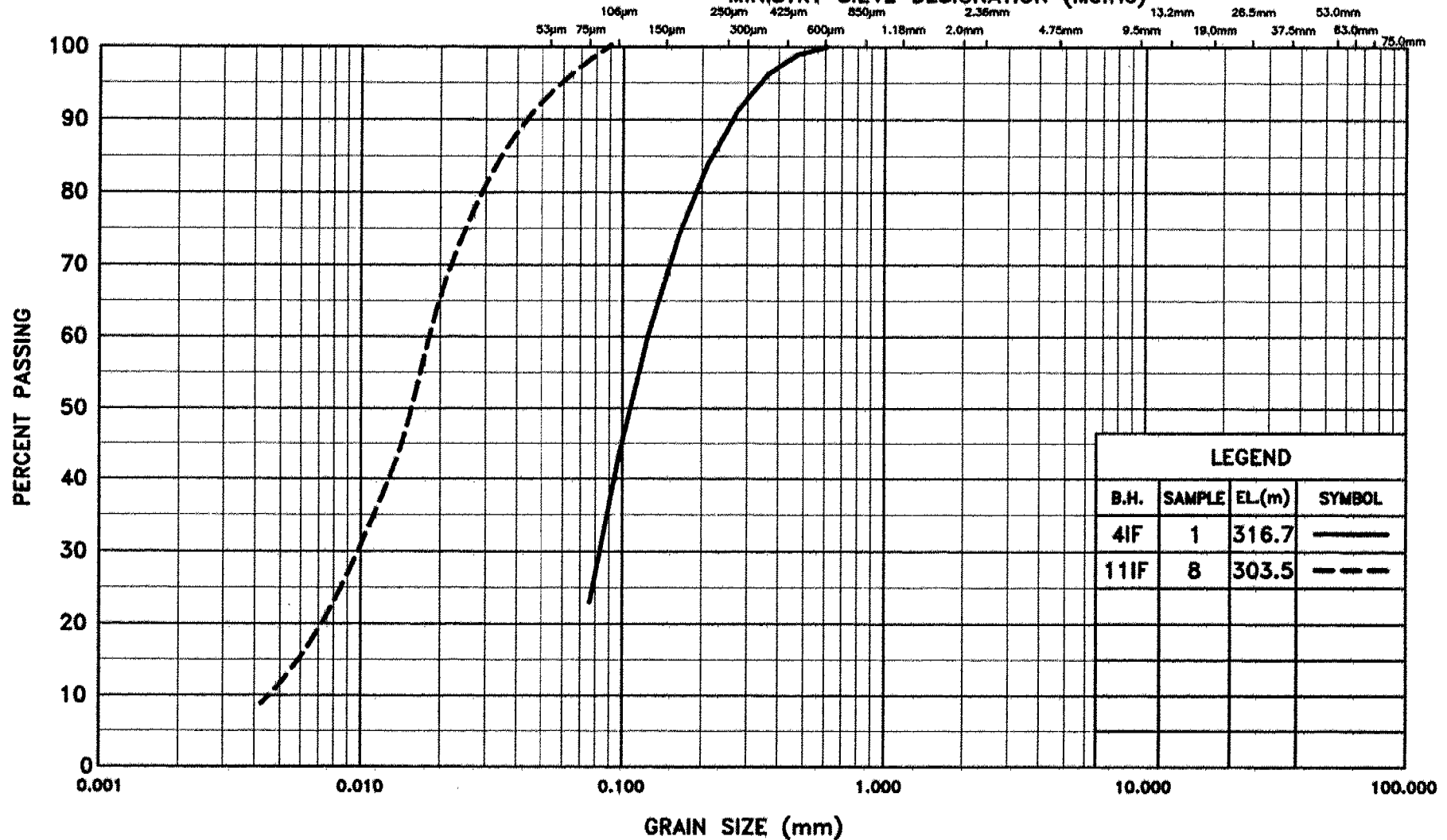
SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)		CONE PENETRATION TEST		WATER CONTENT (%)			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT		
314.80	GROUND SURFACE													
0.00	PEATY TOPSOIL, ~300 mm over SAND, mostly fine with a trace of SILT, wet below ~0.9 m depth. (compact)		1	SS	13									
			2	SS	17									
			3	SS	10									
			4	SS	5									
			5	SS	15									
306.72	END OF BOREHOLE													
8.08	Notes: 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 11 Overpass. 2) Borehole caved wet at ~1.0 m depth on completion. 3) Dynamic cone penetration test driven ~1.5 m west of BH-13IF.													



# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION

SILTY SAND & SILT

FIGURE 1

W.P 774-93-00

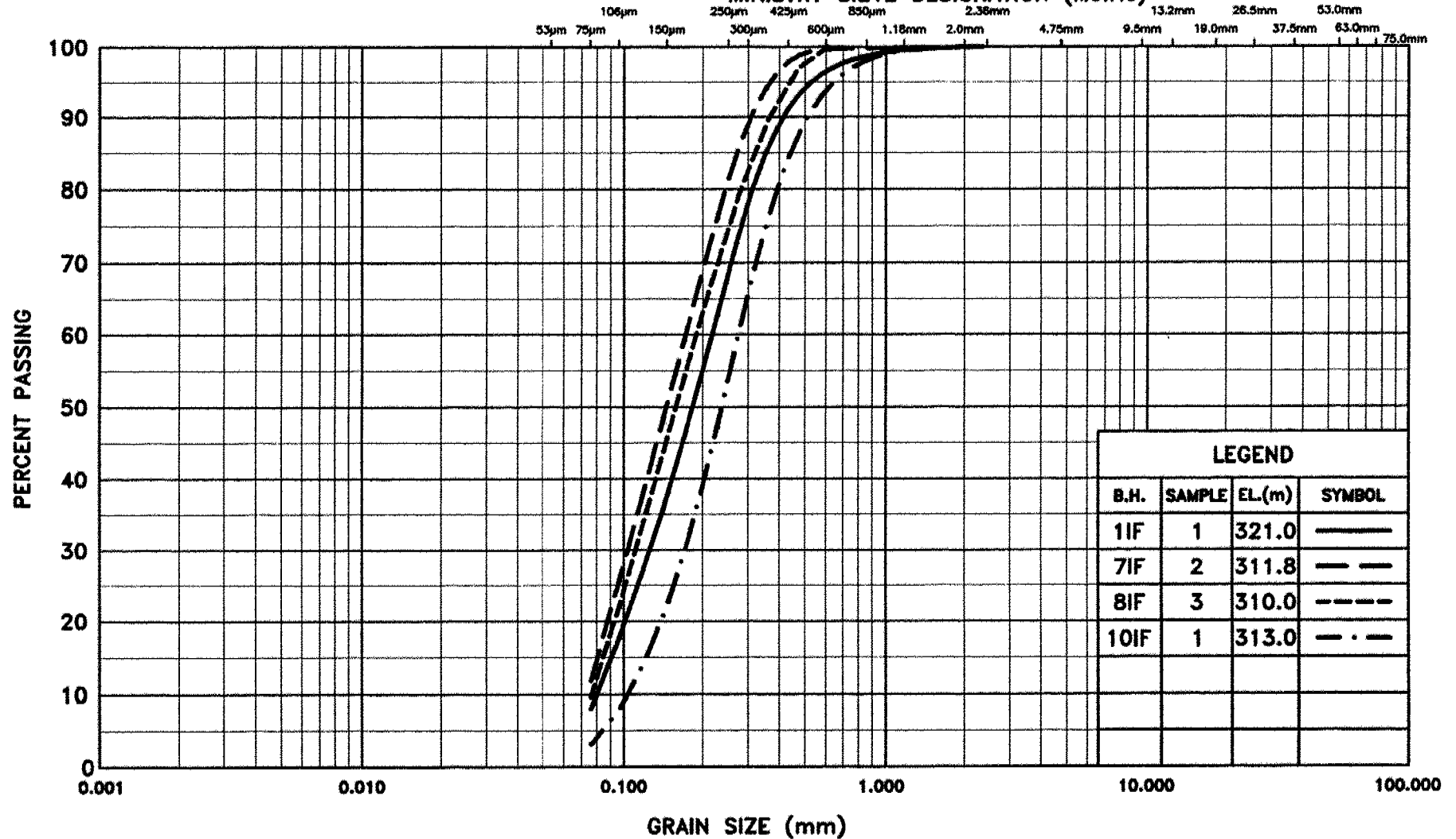


PROJ. No. S07524GIF

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION

SAND

FIGURE 2

W.P 774-93-00



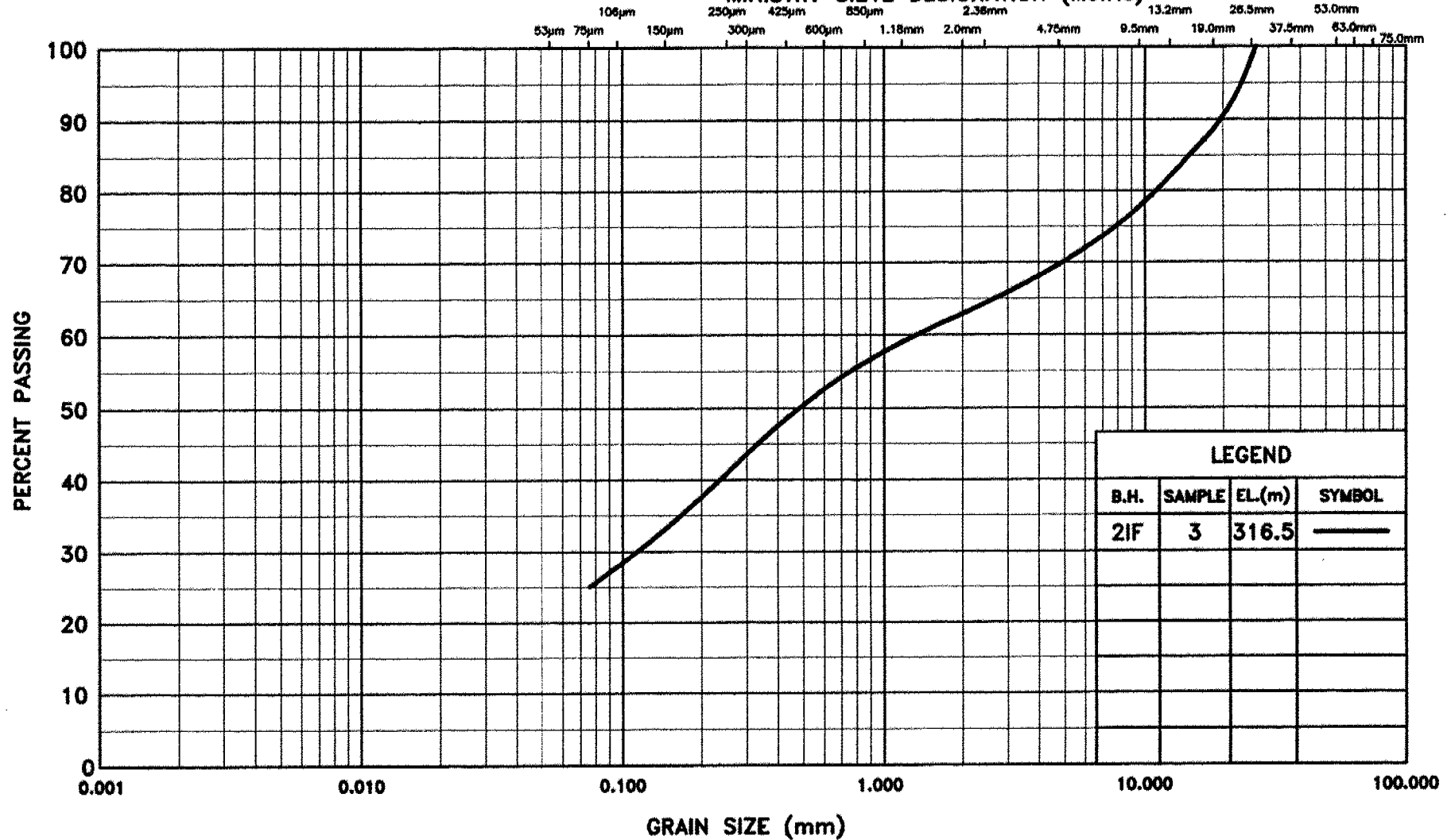
PROJ. No. S07524GIF



# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION

SAND & GRAVEL TILL

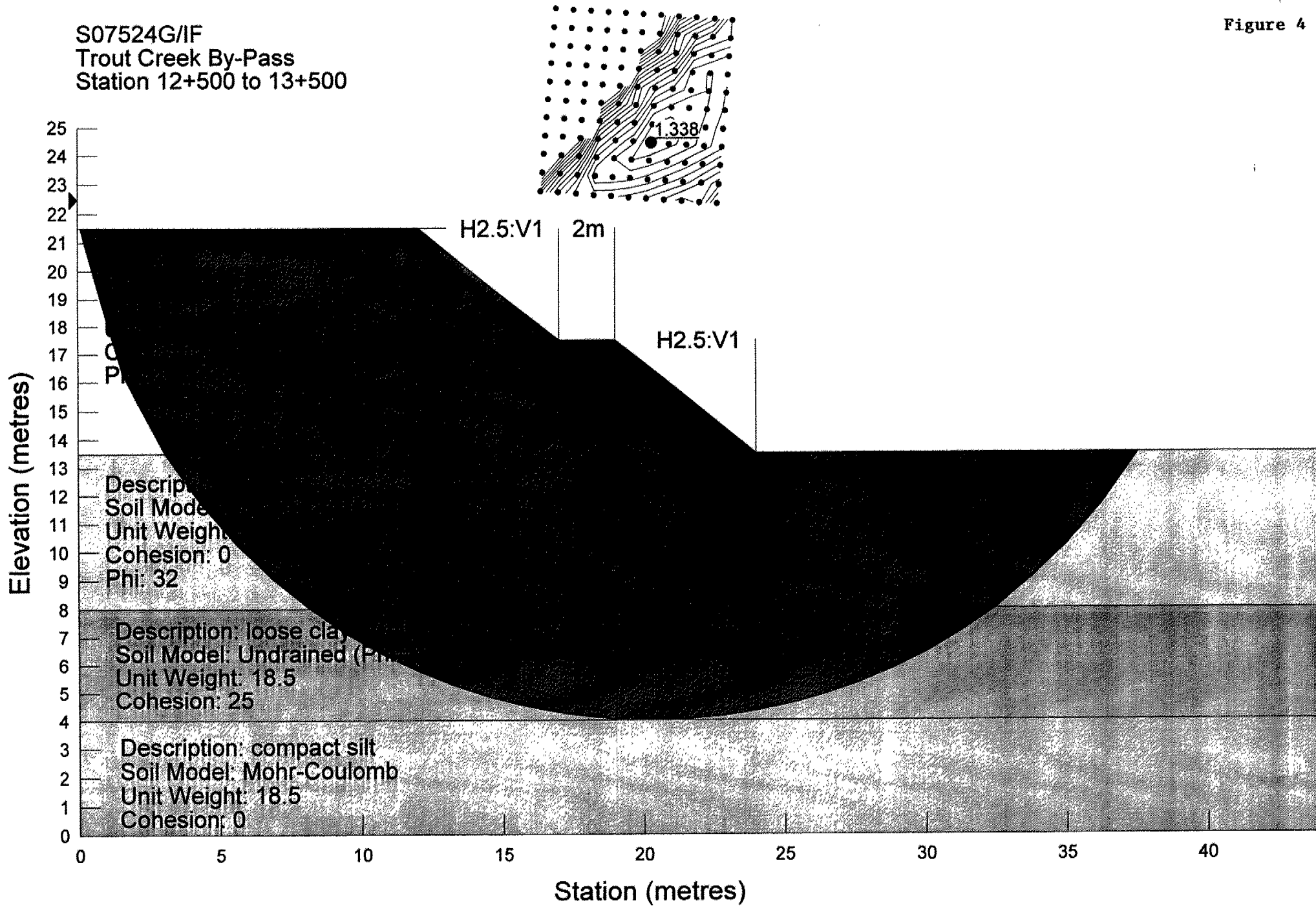
FIGURE 3

W.P 774-93-00



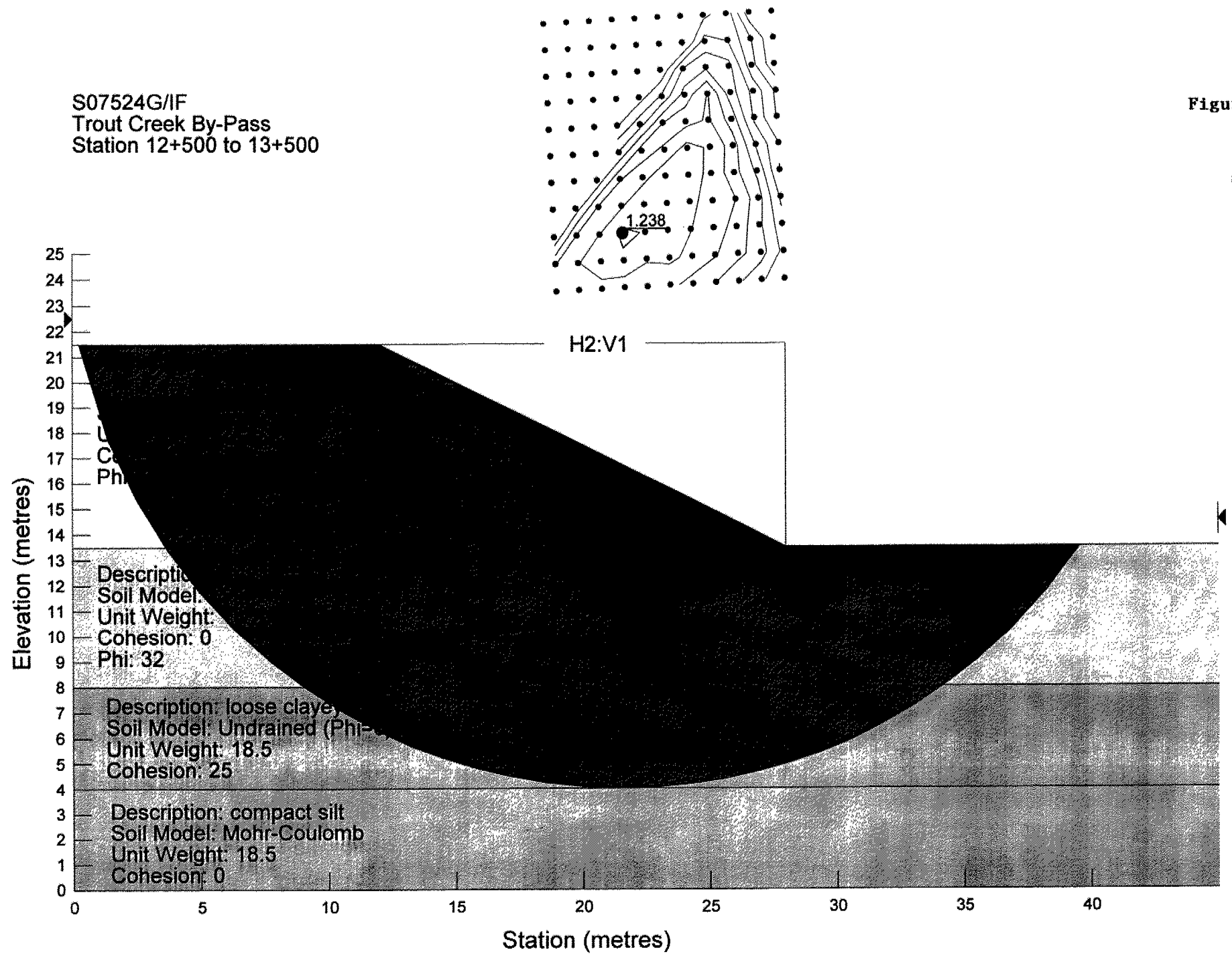
PROJ. No. S07524GIF

Figure 4



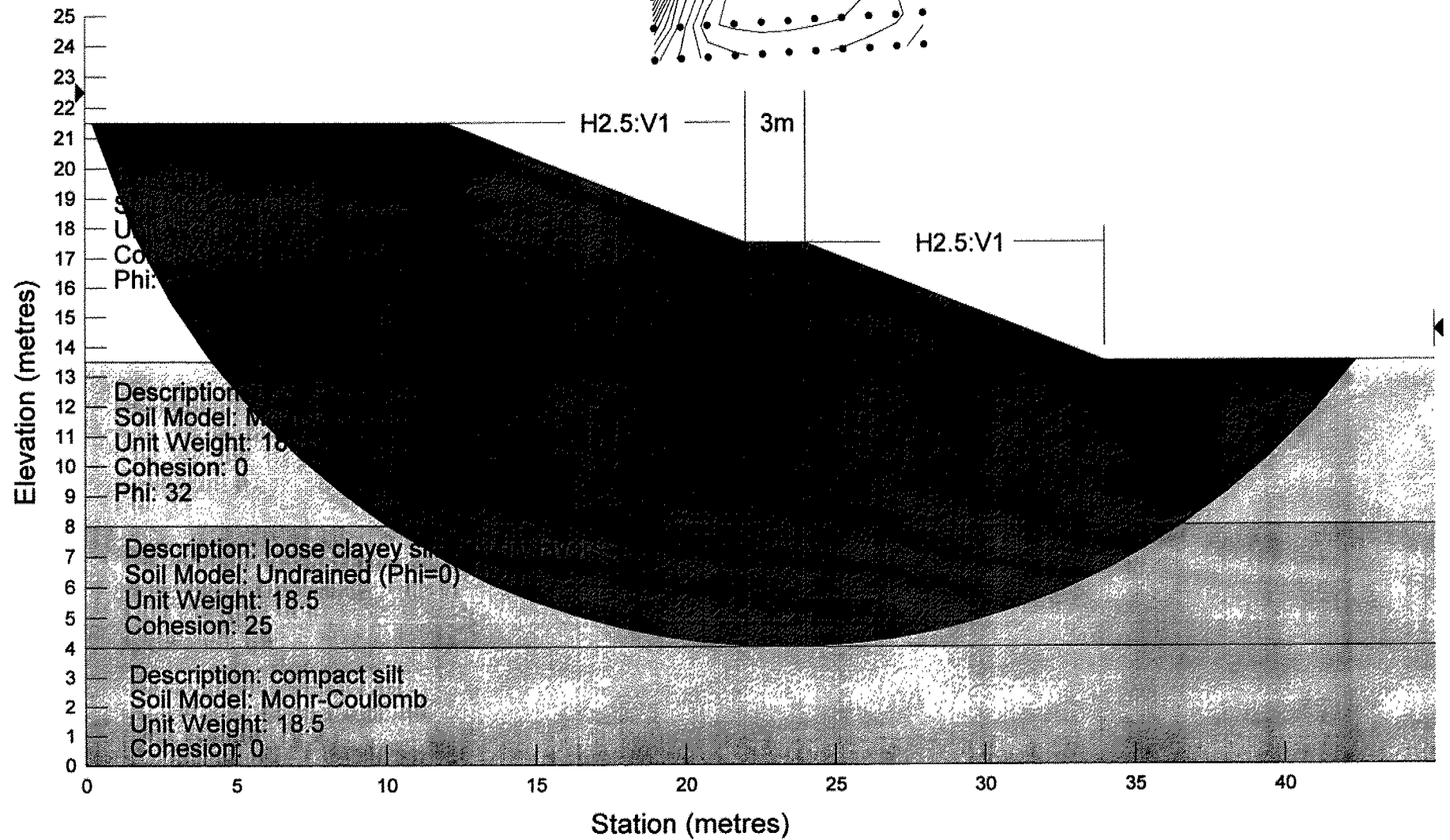
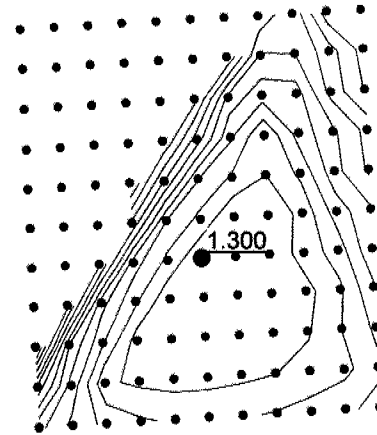
S07524G/IF  
 Trout Creek By-Pass  
 Station 12+500 to 13+500

Figure 5



S07524G/IF  
Trout Creek By-Pass  
Station 12+500 to 13+500

Figure 6



# OVERSIZE DRAWING(S)

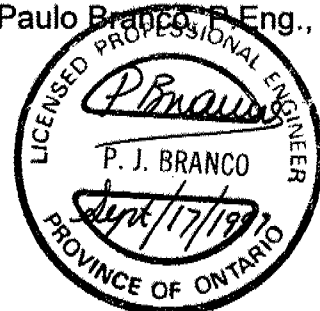
**DESIGN REPORT  
TROUT CREEK BY-PASS - KING'S HIGHWAY 11  
WICK DRAIN DESIGN AND MONITORING PROGRAM  
NORTH INTERCHANGE EMBANKMENTS  
DISTRICT 54, SUDBURY, ONTARIO  
GWP No. 774-93-00**

Report  
to  
Trow Consulting Engineers  
1074 Webbwood Drive  
Sudbury, Ontario, P3C 3B7

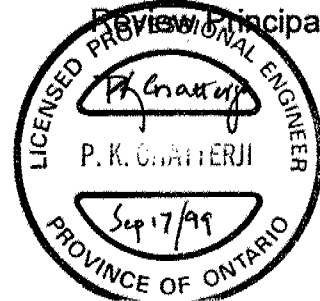
Direction of fieldwork and engineering analysis by:

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

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**DESIGN REPORT  
TROUT CREEK BY-PASS - KING'S HIGHWAY 11  
WICK DRAIN DESIGN AND MONITORING PROGRAM  
NORTH INTERCHANGE EMBANKMENTS  
DISTRICT 54, SUDBURY, ONTARIO  
GWP No. 774-93-00**

**1. Introduction**

This report presents the results of a supplementary geotechnical investigation and engineering analysis carried out by Thurber Engineering Ltd. (Thurber) for the design of wick drains and monitoring program for the proposed approaches and embankments at the North Interchange located north of Trout Creek, at the intersection of McCarthy Street and the proposed King's Highway 11 Trout Creek By-Pass.

Thurber carried out the investigation as a sub-consultant to Trow Consulting Engineers (Trow). The Terms of Reference for this work have been included in a letter by Trow to Thurber dated February 23, 1999. Authorization to proceed with this work was given in a letter by Mr. Eric Gonneau, P.Eng. of Trow, dated March 12, 1999.

**2. Background Information and Scope of Work**

Trow have been retained by Marshall Macklin Monaghan (MMM) to provide geotechnical services as part of the Total Project Management, Detailed Design Services for the above noted project. Trow's scope of work included geotechnical, pavement and foundation investigation and design recommendations for a number of proposed structures along this section of four-laning of Highway 11. The results of Trow's investigation program for the North Interchange were summarized in the following draft report:

- Foundation Investigation, Bridge Structure, Approaches and Embankment Fills - North Interchange (McCarthy Street) - Trout Creek by-Pass, King's Highway 11 - District 54, Sudbury, Ontario, GWP No.774-93-00, January 7, 1999



The investigation by Trow at the North Interchange revealed the presence of thick soft foundation clayey deposits. Trow's analysis indicated that a combination of side berms and wick drains are required for successful construction of the high approach embankments, with final design heights up to 10.5 m. The side berms and wick drains are required to prevent a foundation failure during construction and to accelerate the foundation settlements so that most of the settlements are completed prior to bridge foundation construction and paving of the road.

Thurber Engineering Ltd. (Thurber) has been retained by Trow for the detailed design of the wick drains and to design a geotechnical instrumentation monitoring program to control the embankment performance during and after construction. In order to carry out this task Thurber has been provided with the following:

- portions of the above noted report containing the factual geotechnical data, excluding Trow's recommendations for the embankment design;
- drawings including a site plan, longitudinal profiles and simplified subsurface conditions;
- McCarthy Street Bridge General Arrangement and embankment typical cross sections.

This report should be read in conjunction with Trow's report.

### **3. Methodology**

The work presented herein was developed in the following stages:

- Review of available information;
- Visit to Trow's office in Sudbury for visual inspection of soil samples, on March 18, 1999, by Mr. Scott Peaker, P.Eng. of Thurber. Some soil samples were brought to Thurber's office in Toronto for visual inspection;
- Site visit on March 19, 1999, by Mr. Scott Peaker, P.Eng. of Thurber,

for site reconnaissance and evaluation of site access by a drill rig for piezocone testing

- Piezocone testing on March 25 and 26, 1999
- Engineering Analysis
- Design Recommendations

#### **4. Proposed Interchange**

The North Interchange consists of one bridge structure that will carry the proposed McCarthy Street over the proposed realigned and widened Hwy 11, approach embankments to the bridge and access ramp embankments. The proposed bridge consists of a two span structure with integral abutments, with a length of 67.2 m between abutments. A site plan view is shown on Figure A1 in Appendix A.

The embankments at this site will be constructed to a height of up to 10.5 m using blast rock with side slopes of 1.25H:1V and headslopes at the bridge abutments inclined at 2H:1V.

#### **5. Site Description**

Details about the site location and surface conditions have been included in Trow's report and they will not be repeated herein.

#### **6. Piezocone Testing**

Piezocone testing was carried out with the purpose of:

- confirming the subsurface conditions encountered by Trow
- obtaining continuous strength information at depth
- carrying out pore pressure dissipation tests at selected depths for assessment of the horizontal coefficient of consolidation required for optimizing wick drain design
- measuring the piezometric head at the base of the fine sediments to verify the presence of artesian condition

Piezocone testing (CPTU) was carried out on March 26, 1999, by ConeTec Investigations Ltd. of Vancouver, B.C. The piezocone was pushed using a track mounted CME 75 owned and operated by All Terrain Drilling Ltd. of Waterloo, Ontario.

A total of five CPTUs were carried out at the North Interchange at the approximate locations shown on attached Figure A1. The CPTUs were numbered CPTUN1 through CPTUN5. Table B1 in Appendix B presents approximate coordinates and ground surface elevations at the CPTU locations and the maximum depth of testing where refusal to penetration was encountered.

The results of the CPTUs are summarized in a report by ConeTec included in Appendix C.

Figures A2 to A6 in Appendix A present a summary of both the results of CPTUs and the nearby borehole and laboratory information presented in Trow's report.

## **7. Description of Subsurface Conditions**

### **7.1 Subsurface Soil Conditions**

The subsurface conditions at this site were characterized based on a drilling and laboratory program carried out by Trow and on the results of the CPTUs carried out by ConeTec.

The bridge is located at the western edge of a flat area where the bedrock is overlain by a sequence of non-plastic sediments and a layer of compressible silty clay. The plastic soils are present to the east of the bridge West Abutment. West of the bridge the soil sediments consist mainly of non-plastic silt and sand overlying bedrock.

Of interest to this work is the area located at and east of the bridge, where plastic compressible sediments are present. A description of this area of interest is presented in the following sections. For a detailed analysis of the area west of the bridge the reader should refer to Trow's report.

The subsurface conditions at and east of the bridge consist mainly of a layer of organic soils, to a depth of up to 1 m, overlying loose to compact layers of silty sand, sand, sandy silt and silt. The thickness of these non-plastic deposits increase from 2 m at the bridge to up to 8 m close to the existing Hwy 11.

Of interest to this project is a layer of compressible silty clay that underlies the non-plastic sediments referred to above. The Silty Clay typically increases in thickness and in depth towards the east.

At the bridge site the Silty Clay is absent at the West Abutment but it was encountered underlying the Sand/Silty Sand deposit up to 7.5 m depth at the Central Pier and East Abutment. Several layers of silt, up to 0.5 m thick, were detected by the CPTUN5 in the Silty Clay. The undrained shear strength ( $S_u$ ) interpreted from the CPTU ranged from 40 kPa to 75 kPa, with values higher than 60 kPa typically measured in the silt lenses. The undrained cohesion ( $C_u$ ) measured using a field vane indicated values of about 30 Kpa. One pore pressure dissipation test carried out in the Silty Clay at 4.0 m depth in CPTUN5 indicated a horizontal coefficient of consolidation ( $C_h$ ) value of 142 m<sup>2</sup>/y. Atterberg limits from tests carried out on two samples indicated that the Silty Clay is medium plastic. The CPTU interpretation of the stratigraphy shows that Silty Clay is sensitive which was confirmed by the field vane tests.

East of the bridge the Silty Clay layer is in a relatively uniform condition, similar to those described above, from the East Abutment to approximate Station 10+120. East of Station 10+120 Silty Clay increases in thickness and in depth and it is underlain by interbedded layers of clayey silt, silt and sandy silt. In the stratigraphic profile presented in Trow's Drawing 1E, the interbedded layers of clayey silt, silt and sandy silt were presented as part of the Silty Clay layer. The undrained shear strength ( $S_u$ ) in the Silty Clay interpreted from the CPTU ranged typically from 45 kPa to 60 kPa. The undrained cohesion ( $C_u$ ) measured with the field vane indicated values ranging from 25 kPa to 50 kPa. Pore pressure dissipation tests carried out in the Silty Clay at eight different locations and depths indicated horizontal coefficient of consolidation ( $C_h$ ) values of 74 m<sup>2</sup>/y to 258 m<sup>2</sup>/y. Atterberg Limits obtained from tests carried out on Silty Clay samples indicate that

the material is of low to medium plasticity.

The interpretation of CPTUN1, CPTUN2 and CPTUN3 indicate that south of the proposed McCarthy Street, along the EW-N Ramp, the Silty Clay deposit is 3 m to 6 m thick, and it extends to a depth of 10.5 m at CPTUN3. CPTUN2 did not confirm the stratigraphy encountered at nearby Borehole BH-9BP. With reference to Dwg 11F in Trow's report, the abrupt change in elevation of the Silty Clay layer at BH-9BP, shown in the stratigraphic profile along the centreline of the EW-N Ramp, was not confirmed in CPTUN2, which was carried out relatively close to BH-9BP. The CPTUs south of McCarthy Street indicate a strength profile similar to that described above, along McCarthy Street. One Shelby tube sample collected at 7.7 m depth from BH-10BP, was tested in an oedometer cell and resulted in the following parameters:

$C_v$  (vertical coefficient of consolidation) = 4.7 m<sup>2</sup>/y

$e_o$  (initial void ratio) ~ 1.64

$C_c$  (compression index) = 0.57

$C_r$  (reload index) = 0.032

Underlying the Silty Clay interbedded layers of silt and sandy silt, referred to as Lower Silt, was encountered with total thickness ranging from 5m to 8m. The SPT "N" in the Lower Silt, interpreted from the CPTUs, ranged typically from 6 to 12, which is consistent with the SPT "N" values from the augered holes. Pore pressure dissipation tests carried out in the Lower Silt indicated a wide range of horizontal coefficient of consolidation ( $C_h$ ) values, ranging from 210 m<sup>2</sup>/y to 3,574 m<sup>2</sup>/y. The higher values of  $C_h$  were obtained from tests carried out in sandy lenses.

The Lower Silt is underlain by a silty sand and gravel till. The SPT "N" values in this material were larger than 17.

A more detailed description of the subsurface conditions encountered in the boreholes are presented on the borehole logs in Appendix B of Trow's report. Stratigraphic profiles inferred from the borehole information have been prepared by Trow and are summarized in Appendix A of Trow's report. Laboratory test results are summarized in Appendix C of Trow's

report

## 7.2 Groundwater

The groundwater level at and east of the bridge, observed in the boreholes after completion of drilling carried out by Trow, was at or 0.3 m below ground surface. The stabilized pore pressure measurements carried out at the bottom of the CPTUs in the Lower Silt or the Silty Sand and Gravel Till deposit indicated a piezometric head at or up to 0.45 m above ground surface, implying a small artesian head.

## 7.3 Summary

In summary, with the exception of the subsurface condition at BH-9BP, the CPTUs generally confirmed the stratigraphy presented in Trow's report. The CPTUs indicated, however, that the bottom portion of the compressible fine soils consist mostly of silt and sandy silt. The undrained shear strength values were generally higher in the CPTUs.

Of significant importance to the consolidation analysis and wick drain design was the fact that the  $C_h$  values obtained from the CPTUs were significantly higher than those obtained from one oedometer test and that a slight artesian condition was encountered below the soft sediments.

# 8. **Engineering Analysis**

## 8.1 General

The engineering analysis was carried out in the following stages:

- Selection of cross sections for analysis that represent typical subsurface conditions and embankment configurations with respect to embankment height and width;
- Stability analysis to identify the required stabilizing berm dimensions, required construction staging and required gain in strength after each construction stage due to consolidation in the clayey layers, for a minimum factor of safety of 1.3 during construction;

- Settlement analysis to identify the required height of surcharge and the need for and the spacing of wick drains to accommodate the construction schedule.

Based on the analysis of the subsurface conditions and the geometry of the embankments the following test holes and embankment geometries were selected for analysis:

- *Bridge East Approach - McCarthy St. (Sta. 10+040 to 10+120) and Hwy 11 at and north of the bridge:*  
Characteristics: High embankment close to structure;  
Low and wide embankments at Hwy 11  
Subsurface Conditions: CPTUN5  
Embankment Height (excluding surcharge):  
9 m to 10.5 m at McCarthy St.; 2 m along Hwy 11  
Embankment Width (at the top): 17.4 m at McCarthy St. and 60 m  
at Hwy 11  
Berm Elevation: 6 m below the top of the embankment
- *McCarthy St. (Sta. 10+120 to 10+240) and W-N (Sta. 13+091 to 13+160) and S-EW (Sta. 13+630 to 13+540) Ramps:*  
Characteristics: Intermediate embankment height, deep soft deposits  
Subsurface Conditions: CPTUN1  
Embankment Height (excluding surcharge): 6 m to 9 m  
Embankment Width (at the top):  
17.4 m to ~ 40 m (at "T" intersection)  
Berm Elevation: 6 m below the top of the embankment
- *McCarthy St. (Sta. > 10+240) :*  
Characteristics: Intermediate to low embankment height, thick soft deposits  
Subsurface Conditions: CPTUN4  
Embankment Height (excluding surcharge): 3 m to 6 m  
Embankment Width (at the top): 17.4 m  
Berm Elevation: No berm

- *EW-N (Sta. 13+160 to 13+270) and S-EW (Sta. 13+540 to 13+450)*  
*Ramps:*  
Characteristics: Intermediate to low embankment, thick soft deposits  
Subsurface Conditions: CPTUN2  
Embankment Height (excluding surcharge): 4.5 to 6.0 m  
Embankment Width (at the top): 14 m  
Berm Elevation: no berm
  
- *EW-N (Sta. 13+270 to 13+350) and S-EW (Sta. 13+450 to 13+380)*  
*Ramps:*  
Characteristics: Low embankment height, thick soft deposits  
Subsurface Conditions: CPTUN3  
Embankment Height (excluding surcharge): 3.0 m to 4.5 m  
Embankment Width (at the top): 14 m  
Berm Elevation: no berm
  
- *EW-N (Sta. 13+350 to 13+458) and S-EW (Sta. 13+380 to 13+125)*  
*Ramps and*  
*Hwy 11 south of the bridge:*  
Characteristics: Low embankment height, thick soft deposits  
Subsurface Conditions: BH-11BP  
Embankment Height (excluding surcharge): < 3.0 m  
Embankment Width (at the top): 14 m and 60 m  
Berm Elevation: no berm

Figure A1, Appendix A, shows the Stations used to limit the study regions above.

Table B2, Appendix B, presents a summary of the soil properties used in the stability and settlement analysis for each of the test holes above. The soil properties presented in Table B2 were selected based on the interpretation of the field and laboratory data. In order to avoid an extensive parametric analysis the following criteria was used for the selection of soil properties:



- *Strength:*  
select most likely values in view of the slight conservatism inherent to the undrained analysis and the selected factor of safety during constructions (F.S.~1.3)
- *Pore Pressure Generation:*  
select conservative values of  $B_{bar}$  equal to 1 for cohesive deposits.
- *Time Independent Deformation:*  
Elastic Properties:  
select most likely to conservative values since these parameters, with the exception of selection of the minimum required height of surcharge, have only a minor impact on the cost and performance of the embankment  
Compression Ratio:  $\{C_c/(1+e_o)$  and  $C_s/(1+e_o)\}$  - same as above.  
Pre-Consolidation Pressure ( $P_c$ ):  
This parameter impacts both the time-independent and time-dependent settlements. The latter occurs because the coefficient of consolidation ( $C_v$  and  $C_h$ ) values are significantly impacted by the over-consolidation ratio. A  $P_c$  value obtained from one oedometer test has been provided in Trow's report (Table 1-2). Due to the importance of properly assessing the  $P_c$  values, two  $P_c$  values were selected for a sensitivity analysis:
  - Most Likely Value:  $P_c = S_u/0.235$  ( $S_u$  is the undrained shear strength)<sup>1</sup>
  - Reduced Values: 50% of the Most Likely Value above and not lower than the anticipated in situ vertical effective stress
- *Time Dependent Deformation:*  
Coefficient of Consolidation:  
 $C_h$  (horizontal): Over-consolidated: select values

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<sup>1</sup>

Ladd, C.C. (1991). "Stability Evaluation During Staged Construction", ASCE Journal of Geotechnical Engineering, Vol.17, No.4, 1991

interpreted from the CPTUs  
Normally Consolidated: select the minimum  $C_h$  value interpreted from the CPTUs in that deposit;  
 $C_v$  (vertical): 20% of  $C_h$  above (lower bound values<sup>2</sup>)  
Secondary Compression Ratio ( $C_\alpha$ ):  
Select the values measured in the pre-consolidated range of oedometer tests assuming that the surcharge will be removed after 100% completion of primary consolidation.

## 8.2 Stability Analysis

The stability analysis was carried out based on the following assumptions:

- Embankment Geometry:
  - Side slopes: 1.25H:1V
  - The width of the embankment at the top of the surcharge will be the same as the final design width. Hence, the embankment side slopes above the berm will be temporarily steeper than 1.25H:1V. This is required to maintain the minimum required embankment width at the top after settlements due to primary consolidation take place.
  - The berm height was maintained 6 m below the top of the final embankment height
- Surcharge: Up to 1.5 m above the embankment design height. Actual height of surcharge to be verified based on the settlement analysis
- Site Preparation: All organic soils will be removed within the footprint of the embankment

<sup>2</sup>

Hansbo, S. (1979). "Consolidation of clay by band-shaped prefabricated drains". Ground Engineering, July, Vol.12, NO.5, 16-25, 1979

- Limit Equilibrium Analysis:  
Bishop Modified using G-Slope, developed by Mitre Software.
- Soil Shear Strength:  
Undrained shear strength ( $S_u$ ) for cohesive soils; Drained ( $\phi'$ ) for cohesionless soils.  $S_u$  increases with vertical stress; for vertical stress larger than the pre-consolidation pressure ( $P_c$ ): use  $S_u = 0.235 \cdot \sigma'_v$ , for  $\sigma'_v > P_c$ . The values of  $\sigma'_v$  at depth and at different times were obtained from the consolidation analysis presented in Section 8.3.
- Groundwater Table:  
At the original ground surface

The results of the stability analysis are summarized in Table B3 and Figures A7 to A10. The analysis of Table B3 indicates the following:

*Location: East Abutment - McCarthy St. - CPTUN5*

- The construction of the embankment to a height of 12 m, including surcharge, with a berm width of 11 m is feasible in one construction stage.
- The construction of the embankment to a height of 12 m, including surcharge, with a berm width of 9 m is feasible in two construction stages:
  - Stage 1: from 0 m to 11.5 m
  - Stage 2: from 11.5 m to 12 m with 75% dissipation of excess pore pressure (EPP) after Stage 1
- The embankment temporary headslopes at the abutment locations will be constructed in two stages according to the above schedule. The temporary headslope crest will be located at the abutment location, with maximum height of 12 m, sloping towards Hwy 11 inclined at 1.25H:1V, provided that the Hwy 11 embankments under the bridge are constructed prior to the temporary abutment embankments.

*Location: East Approach to Bridge-McCarthy St. and E-W and S-EW  
Ramps - CPTUN1*

- The construction of the embankment, with a berm width of 8 m, to a height of 9.5 m, including 1.5 m surcharge, is feasible in two construction stages:
  - Stage 1: from 0 m to 8.0 m
  - Stage 2: from 8.0 m to 9.5 m with 100% dissipation of EPP after Stage 1

Due to the fact that it is not practical to wait for 100% consolidation between Stages 1 and 2, different berm widths were considered in the analysis, as follows:

- The construction of the embankment, with a berm width of 9 m, to a height of 9.5 m, including 1.5 m surcharge, is feasible in two construction stages:
  - Stage 1: from 0 m to 8.8 m
  - Stage 2: from 8.8 m to 9.5 m with 75% dissipation of EPP after Stage 1
- The construction of the embankment, with a berm width of 11 m, to a height of 9.5 m, including 1.5 m surcharge, is feasible in two construction stages:
  - Stage 1: from 0 m to 9 m
  - Stage 2: from 9 m to 9.5 m with 75% dissipation of EPP after Stage 1

It should be noted from an analysis of Table B3 that the Factor of Safety for this case, with a berm width of 11 m is slightly larger than for a berm width of 9 m, as expected.

*Location: McCarthy St.-Transition to high embankments- BH-3BP*

- The construction of the embankment to a target height of 8.5 m, including 1.5 m of surcharge, is feasible in one stage with a minimum berm width of 8 m.

*Location: McCarthy St.- Embankments up to 6m high- BH-3BP to BH4-BP*

- The construction of the embankment to a target height of 7.5 m,

including 1.5 m of surcharge, is feasible in one stage without side berms.

*Location: EW-N and S-EW Ramps up to 6m high - CPTUN2*

- The construction of the embankment to a target height of 8.5 m, including 1.5 m of surcharge, is feasible in one stage with a minimum berm width of 9 m.

### 8.3 Settlement Analysis

#### 8.3.1 General

The settlement analysis was carried out in the following steps:

- One dimensional primary consolidation analysis: no wick drains
- Pseudo three dimensional consolidation analysis: with wick drains
- One dimensional secondary consolidation analysis

#### 8.3.2 One Dimensional Consolidation Analysis - No Wick Drains

The one dimensional consolidation analysis was carried out in order to:

- establish the required height of surcharge;
- establish the need for wick drains;
- provide input for the vertical consolidation component in the wick drain design
- provide excess pore pressure dissipation at depth for the assessment of gain of shear strength with time for the stability analysis

The analysis was carried out using the finite difference software Consol Version 2.0, developed at Virginia Polytechnic Institute and State University. The program allows the one dimensional consolidation analysis of multilayered soil masses, taking into account non-linear constitutive law, variable parameters as a function of the over-consolidation ratio, impeded drainage and variable boundary conditions. The ability to model impeded drainage was considered a key factor in the selection of this software, due

to the presence of layers of silt above and below the Silty Clay layer deposit.

The vertical stress distribution under the embankment was estimated using Boussinesq's stress distribution under an infinite strip loaded area.

The following simplified embankment construction schedule was used in our analysis:

- Stage 1: the embankment load was applied instantly at time zero
- Stage 2: the additional load was applied instantly at the time after the EPP had dissipated enough for a minimum FS of 1.3 against global stability according to the stability analysis presented in the preceding section.

This is a simplified model of the actual construction process in which several days or weeks will be required to construct the embankment to the specified height. The adopted approach predicts larger settlements and lower EPP in the soft sediments at any point in time provided the time elapsed between the construction stages is adopted as the time elapsed between the end of the embankment construction at Stage 1 and beginning of Stage 2.

Figures A11 to A21, Figures A11B to A21B and Tables B4 to B17 present a summary of the results of the one dimensional consolidation analysis for a range of embankment heights within each representative area. The bottom portion of Tables B4 to B17 show the "*minimum amount of time*" after the end of the embankment construction when the surcharge may be removed for stabilization of settlements due primary consolidation. The "*minimum amount of time*" is defined herein as the time required for the EPP in the cohesive soils to dissipate to values that, when the surcharge is removed, the EPP will disappear due to a relief of vertical total stresses. Therefore, after the removal of the surcharge, the cohesive soils are normally consolidated. However, in order to minimize long term settlements due to secondary consolidation it is desirable that the cohesive soils be slightly overconsolidated after the removal of the surcharge. This is possible with

the full dissipation of EPP and stabilization of settlements due to primary consolidation for an embankment height 0.5 to 1.0 m higher than the final embankment height. Therefore the elapsed times shown in the bottom part of Tables B4 to B17 should be treated as the minimum and not necessarily the ideal elapsed times after the end of construction for removal of the surcharge and reshaping of the embankment.

The construction schedules that Thurber has been requested to analyse are:

Schedule 1:

- Site Preparation: 2 months (removal of organics, wick drain installation)
- Embankment construction to the final target height including surcharge: 3 months
- Waiting period for primary consolidation: 12 months

Schedule 2:

- Site Preparation: 2 months (removal of organics, wick drain installation)
- Embankment construction to the final target height including surcharge and stabilization of settlements: 12 months

Schedule 3:

- Site Preparation: 2 months (removal of organics, wick drain installation)
- Embankment construction to the final target height including surcharge and stabilization of settlements: 6 months

Based on these construction schedules and on the analysis of Figures A11 to A21, Figures A11B to A21B and Tables B4 to B17, the following can be concluded:

- *General:* The pre-consolidation pressure has a significant impact on the magnitude and the time required for the dissipation of EPP
- *Bridge East Approach - McCarthy St. (Sta. 10+040 to 10+120)*  
*H=10.5 m plus 1.5 m surcharge - CPTUN5 - Figure A19 and*

*Tables B4 and B5:*

- Time delay between Stages 1 and 2: 90 days
  - Time after the end of construction required for stabilization of settlements due to primary consolidation for embankment with surcharge: 400 days
  - Minimum time required after the end of construction for removal of surcharge and stabilization of primary consolidation of embankment at final design elevation:
    - 4 to 6 months for Most Likely Pc;
    - 6 to 8 months for Reduced Pc
  - Based on time required between Stage 1 and 2 for embankment with berm width <11m, and the time required for stabilization of settlements, wick drains will be required to accommodate the proposed construction Schedules 1 and 3. Wick drains are not required for Schedule 2.
- *Location: Hwy 11 at and north of the bridge - H=2 m plus 1 m surcharge - CPTUN5 - Figure A20; Tables B6 and B7*
    - Time delay between Stages 1 and 2: N/A
    - Time after the end of construction required for stabilization of settlements due to primary consolidation for embankment with surcharge: 300 days
    - Minimum time required after the end of construction for removal of surcharge and stabilization of primary consolidation of embankment at final design elevation:
      - 1 to 2 months for Most Likely Pc;
      - 1 to 2 months for Reduced Pc
    - Wick drains are not required for Schedules 1, 2 and 3.
  - *Location: McCarthy St. (Sta. 10+120 to 10+240) and W-N (Sta. 13+091 to 13+160) and S-EW (Sta. 13+630 to 13+540) Ramps - H=9m and 6m plus 1.5 m surcharge CPTUN1 - Figures A11 and A12, Tables B8 and B9:*  
*For H=9m plus 1.5 m surcharge:*
    - Time delay between Stages 1 and 2: 60 days
    - Time after the end of construction required for stabilization of settlements due to primary consolidation for embankment with



- surchage: 300 days
  - Minimum time required after the end of construction for removal of surcharge and stabilization of primary consolidation of embankment at final design elevation:
    - 3 to 4 months for Most Likely Pc;
    - 4 to 6 months for Reduced Pc
  - Based on time required between Stage 1 and 2 for embankment with berm width <11m, and the time required for stabilization of settlements, wick drains will be required to accommodate the proposed construction Schedules 1 and 3. Wick drains are not required for Schedule 2.
- *Location: McCarthy St. (Sta. > 10+240) - H=3m and 6m plus 1.0m and 1.5m surcharge - CPTUN4 - Figures A17 and A18, Tables B10 and B11:*
  - For H=6m plus 1.5 m surcharge:*
    - Time delay between Stages 1 and 2: N/A
    - Time after the end of construction required for stabilization of settlements due to primary consolidation for embankment with surcharge: >720 days
    - Minimum time required after the end of construction for removal of surcharge and stabilization of primary consolidation of embankment at final design elevation:
      - 8 to 10 months for Most Likely Pc;
      - 10 to 12 months for Reduced Pc
    - Based on time required for stabilization of settlements, wick drains will be required to accommodate construction Schedule 3. Wick drains are not required for Schedules 1 and 2.
- *EW-N (Sta. 13+160 to 13+270) and S-EW (Sta. 13+540 to 13+450) Ramps - H=7.5m and 4.5m plus 1.5 and 1.0 m surcharge - CPTUN2 - Figures A13 and A14, Tables B12 and B13:*
  - For H=7.5m plus 1.5 m surcharge:*
    - Time delay between Stages 1 and 2: N/A
    - Time after the end of construction required for stabilization of settlements due to primary consolidation for embankment with

- surcharge: 600 days
  - Minimum time required after the end of construction for removal of surcharge and stabilization of primary consolidation of embankment at final design elevation:
    - 4 to 6 months for Most Likely Pc;
    - 6 to 8 months for Reduced Pc
  - Based on time required for stabilization of settlements, wick drains will be required to accommodate construction Schedule 3. Wick drains are not required for Schedules 1 and 2.
- *EW-N (Sta. 13+270 to 13+350) and S-EW (Sta.13+450 to 13+380) Ramps - H=4.5m and 3.0m plus 1.0 m surcharge - CPTUN3 - Figures A15 and A16, Tables B14 and B15:*  
*For H=4.5m plus 1.0 m surcharge:*
  - Time delay between Stages 1 and 2: N/A
  - Time after the end of construction required for stabilization of settlements due to primary consolidation for embankment with surcharge: 360 days
  - Minimum time required after the end of construction for removal of surcharge and stabilization of primary consolidation of embankment at final design elevation:
    - 3 to 4 months for Most Likely Pc;
    - 6 to 8 months for Reduced Pc
  - Wick drains are not required for Schedules 1, 2 and 3.
- *EW-N (Sta. 13+350 to 13+458) and S-EW (Sta.13+380 to 13+125) Ramps and Hwy 11 south of the bridge - H=3.0m plus 1m surcharge - BH-11BP - Figure A21, Tables B16 and B17:*
  - Time after the end of construction required for stabilization of settlements due to primary consolidation for embankment with surcharge: >720 days
  - Minimum time required after the end of construction for removal of surcharge and stabilization of primary consolidation of embankment at final design elevation:
    - 2 to 3 months for Most Likely Pc;

10 to 12 months for Reduced  $P_c$

- Wick drains are not required for Schedules 1, 2 and 3.

### 8.3.3 Settlements due to Primary Consolidation - With Wick Drains

The one-dimensional consolidation analysis above identified that, depending on the construction schedule selected, the following areas will require wick drains to accelerate dissipation of EPP:

- Bridge East Approach - McCarthy St. (Sta. 10+040 to 10+120)
- McCarthy St. (Sta. 10+120 to 10+240) and W-N (Sta. 13+091 to 13+160) and S-EW (Sta. 13+630 to 13+540)
- McCarthy St. (Sta. 10+240 to Sta. 10+310)
- EW-N (Sta. 13+160 to 13+270) and S-EW (Sta. 13+540 to 13+450)

The presence of slight artesian pressures in the non-plastic silt deposit underlying the Silty Clay deposit poses a potential for loss of fines due to the continuous flow of water around the wick drains. In order to minimize this potential, the wick drains should be terminated within the Silty Clay, about 1 m above the underlying layer of silt.

The wick drain spacing was selected based on the percentage consolidation required within the Silty Clay layer, determined from the stability analysis, for the construction schedules presented in the preceding section.

Two methods were used for the wick drain design:

- Hansbo (1979, opt.cit.)
- Robertson, Campanella and Brown<sup>3</sup> (1988)

The former method includes well resistance and disturbance factors due to the wick drain installation. The latter method uses the original derivation by Hansbo<sup>4</sup> (1960) adjusted for wick drain design based on  $C_h$  values

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<sup>3</sup> Robertson, P.K., Campanella, R.G., and Brown, P.T. (1988). "Prediction of wick drain performance using piezometer cone data". Canadian Geotechnical Journal 25, 56-61 (1988)

<sup>4</sup> Hansbo, S. (1960). "Consolidation of clay, with special reference to influence of vertical sand drains. Swedish Geotechnical Institute, Proceedings No.18 (1960)

interpreted from the CPTU. The wick drain spacing has been selected as the smallest of the two spacings provided by the two methods above. EPP dissipation due to vertical drainage was coupled with EPP due to horizontal drainage into the wick drains according to the following equation:

$$U = 1 - (1 - U_v)(1 - U_h)$$

where U is the combined total percentage consolidation and  $U_v$  and  $U_h$  are the percentage consolidation values due to vertical and horizontal drainage only, respectively, divided by 100.

The design parameters and required percentage consolidation at specific times used in the analysis are summarized in Table B18. Since the wick drain design methods described above do not allow inclusion of variable  $C_h$  values for the horizontal drainage portion of the analysis, the lowest value of  $C_h$  has been assumed for a specific test hole location. It has been assumed that the wick drains will be installed in a triangular pattern. Since the wick drains will be terminated within the Silty Clay, the wick drain drainage length has been assumed equal to the entire length of the wick drain.

The results of the wick drain analysis are presented in Tables B19 to B30. Figures A11-C, A13-C, A17-C A19-C and A19-D present EPP dissipation at depth. These tables and figures present only the analyses carried out for Most Likely  $P_c$  values. The analyses carried out with Reduced  $P_c$  values yielded results very similar to the Most Likely  $P_c$  values due to the fact that the lowest  $C_h$  values were selected at the test hole locations, as discussed above.

A summary of the required wick drain spacing for each of the regions studied and different construction schedules is presented in Table 31.

#### 8.3.4 Settlements due to Secondary Consolidation

Settlements due to secondary consolidation have been assessed based on the following equation:

$$\Delta T_{cs} = C\alpha\epsilon * T * \text{Log } t_{sc}/t_p,$$

where:

$\Delta T_{cs}$  = settlement due to secondary consolidation

$C\alpha\epsilon$  = secondary compression ratio

$T$  = initial thickness of compressible layer

$t_{sc}$  = time over which secondary consolidation is to be calculated

$t_p$  = time to complete primary consolidation

As indicated in Table B2, a value of 0.002 has been selected for  $C\alpha\epsilon$ . This value reflects the fact that upon completion of primary consolidation, a minimum of 0.5 m to 1.0 m will be removed from the embankment top and the compressible soils will be slightly over-consolidated.

The settlements due to secondary consolidation anticipated at the interchange embankments are presented in the table in the following page.

The settlements due to secondary consolidation below indicate that the design requirement of maximum long term settlement of 25 mm, after removal of the surcharge, is met at all locations with the exception of McCarthy St at Stations greater than 10+240. In view of the extended period of time considered in the calculations (35 years) and the fact that the embankments where the design specifications, for on going secondary settlements, are exceeded are not in the proximity of the bridge, these anticipated long term settlements may be acceptable to MTO. This should be discussed with MTO.

### Secondary Consolidation Analysis

Location	T (m)	$t_{sc}$ (years)	$t_p$ (years)	$\Delta T_{cs}$ (mm)
McCarthy St. (10+040 - 10+120) CPTUN5	4.5	35	1	15
McCarthy (10+120 - 10+240); W-N Ramp (13+091 - 13+160) S-EW Ramp (13+630 - 13+540) CPTUN1	4.5	35	1	15
McCarthy (>10+240) CPTUN4	9	35	1	30
W-N Ramp (13+160 - 13+270) S-EW Ramp (13+540 - 13+450) CPTUN2	7.5	35	1	25
W-N Ramp (13+270 - 13+350) S-EW Ramp (13+450 - 13+380) CPTUN3	7	35	1	25
W-N Ramp (13+350 - 13+458) S-EW Ramp (13+380 - 13+125) Hwy 11 South of Bridge BH-11BP	8.5	35	1	25

#### 8.4 Lateral Displacement at Depth at the East Abutment

Provided that the abutment piles are installed after most of the settlements due to primary consolidation have taken place, relatively small time dependent lateral displacements are anticipated to occur along the piles. For monitoring purposes and verification of the structural capacity of the abutment piles, the maximum outstanding pile lateral deflection should be equal to 20% (Ladd, opt.cit.) of the maximum outstanding settlement of the embankment at the centre of the silty clay layer, at EL. 311. The lateral deflections can be assumed decreasing to zero above and below the point of maximum deflection, at ground surface and at the top of the silty sand & gravel layer, at El. 303, respectively.

## 9. Embankment Design Recommendations

### 9.1 Embankment Geometry and Construction Schedule

Based on the analysis presented in the preceding sections, the embankment design, wick drain location and spacing and construction sequence summarized in Table B31 is proposed. It should be noted that recommendations have been provided for a berm width of 9 m, since other berm sizes either required 100% consolidation between construction stages, which is not desirable, or resulted in minor gain in construction time.

### 9.2 Site Preparation

All organic soils should be removed within the footprint of the embankments, including side berms. Due to the relatively high groundwater table at this site, a NSSP should be included in the contract documents warning the contractor that the removal of organic soils will probably be carried out below water at most locations. Where unwatering of excavation is required, it shall comply with the requirements of OPSS 517

Following the removal of organic soils, at locations where wick drains will be installed, free draining material, complying with the NSSP included in Appendix D, should be placed to an elevation at least 0.5 m above the groundwater table with minimum thickness of 0.5 m.

### 9.3 Wick Drain Specifications

In order to satisfy the design requirements for discharge capacity, soil retention, permeability and clogging criteria, and installation, the wick drains should be supplied and installed according to the NSSP included in Appendix D.

## 9.4 Monitoring Program

### 9.4.1 Types of Instruments

The performance of selected areas of the embankments will be monitored using the following instruments:

- **Slope Indicator (SI):** to monitor horizontal displacements at depth at the abutment locations. Due to the potential for large settlements at the abutment locations telescopic casings should be used and selected to accommodate settlements of up to 1 m;
- **Vibrating Wire Settlement Cells with Pressurized Reservoir (SC):** for the remote monitoring of settlements of the embankment base at the abutment locations;
- **Settlement Rods (SR):** anchored on a steel plate at ground surface, at the base of the embankment, extended to the top of the embankment for monitoring of settlements of the embankment base with conventional survey methods. The rods should be protected by a PVC or ABS pipe of larger diameter, to minimize the development of friction along the rods, and by a 400 mm CMP, for protection against damage during the embankment construction. The rods and protection pipes should be erected in 3 m increments as the embankment increases in height.
- **Settlement Pins (SP):** standard steel pins anchored in a concrete block cast on top of the embankment surcharge.
- **Vibrating Wire Piezometers (VWP):** installed in the compressible clayey silt and silty clay deposits and underlying sand deposit. The VWPs should be installed as close as possible to the centre of the triangle defined by the nearby three wick drains.
- **Shallow Standpipe (SSP):** installed near each of the monitoring sections to monitor the near surface groundwater table
- **Read-out Unit:** depending on the economics of the monitoring program, the vibrating wire instruments may be read automatically at specified time increments by an automatic acquisition system



### 9.4.2 Monitoring Sections

The instruments will be installed in the following three typical monitoring sections:

#### ***Monitoring Section Type A***

- Location: at the East Abutment location, 3 m behind the line of piles: ~Sta. 10+043
- One SI: at the embankment centreline
- Two SC: at the centreline of the E/B and W/B lanes
- Two SR: at the centreline of the E/B and W/B lanes (1.0 m from the SC)
- Four SP: Two at the top of the surcharge: at the centreline of the E/B and W/B lanes (1.0 m from the SR);  
Two: one on each side berm, near the side slope of the main embankment
- Two strings of VWP: at the centreline of the E/B and W/B lanes.  
One string will include Two VWP installed at the following elevations: EL 311.5 (0.5 m above the bottom of the silty clay layer) and EL 310.5 (0.5 m below the top of the silty clay layer).  
The other string of VWPs should include the VWPs above plus one VWP installed in the Sand and Gravel layer, at EL. 302.
- One Standpipe: Installed to 3 m depth and slotted in the bottom 1 m

#### ***Monitoring Section Type B***

- Location: McCarthy St. Sta. 10+160
- Two SR: at the centreline of the E/B and W/B lanes
- Four SP: two at the top of the surcharge: at the centreline of the E/B and W/B lanes (1.0 m from the SR);  
Two: one on each side berm, near the side slope of the main embankment
- Two strings of VWP: at the centreline of the E/B and W/B lanes.  
Each string will include three VWP. One string will include VWs installed at the following elevations: EL 310, EL 308, EL 306. The other string of VWPs should

include VWPs installed at the same elevations above plus one VWP installed in the loose to compact silt layer, at EL. 302.

One Standpipe: Installed to 3 m depth and slotted in the bottom 1 m

***Monitoring Section Type C:***

Locations:

McCarthy St. Sta. 10+080

McCarthy St. Sta. 10+240

McCarthy St. Sta. 10+310

EW-N Ramp Sta. 13+160 and S-EW Ramp Sta. 13+540

EW-N Ramp Sta. 13+270 and S-EW Ramp Sta. 13+450

EW-N Ramp Sta. 13+350 and S-EW Ramp Sta. 13+380

EW-N Ramp Sta. 13+450 and S-EW Ramp Sta. 13+280

S-EW Ramp Sta. 13+180

Hwy 11 Sta. 13+300

Hwy 11 Sta. 13+400

Hwy 11 Sta. 13+492 (Bridge Centreline)

Hwy 11 Sta. 13+600

Hwy 11 Sta. 13+700

One SR: at the embankment centreline

Two SPs: each at 3 m from the embankment crest; on top of the surcharge

**9.4.3 Installation of Instruments**

With the exception of the Settlement Pins, all instruments should be installed after the site preparation, construction of the drainage blanket and installation of wick drains. It would be preferable to have the instruments installed before the wick drains but the potential for damaging the instruments during installation of the wick drains is too high.

The Settlement Pins should be installed immediately after the embankment target height (top of surcharge) is reached.

#### 9.4.4 Frequency of Readings

All instruments should be initialized and read at least three times in three different days before placement of any rock fill.

During construction the instruments should be read at least once immediately before the placement of 1 m high fill lifts and at least once a week between construction of 1 m lifts and between Stage 1 and Stage 2.

Upon completion of the embankment construction to the top of surcharge the instruments should be read:

- weekly for a period of 2 months
- monthly thereafter until the removal of the surcharge
- weekly for a period of 1 month after the removal of surcharge
- monthly for a period of one year following the removal of surcharge
- once every three months following the paving of the roads for a period of three years

#### 9.4.5 Monitoring Levels

There are basically three parameters that should be monitored closely during and after construction:

- Excess Pore Pressures (EPP)
- Embankment Base Settlement
- Lateral Displacements at Depth

The EPP requirements for stability purposes during construction are shown in Table B31. The EPP shown have priority over the estimated times shown in Table B31.

The monitoring of settlements after the end of construction of the embankment to top of surcharge, allows the assessment of long term settlements due to primary consolidation and when the surcharge can be removed for the pavement construction. It is recommended that the

Rectangular Hyperbola Method<sup>5</sup> be used for reduction of and prediction of long term settlements due to primary consolidation.

Lateral deflections at the abutment pile locations should also be monitored in order to confirm that the lateral displacements due to primary consolidation have mostly stabilized prior to installation of piles.

### 9.5 Trial Embankment

Although the CPTUs provided a significant increase in confidence about the foundation material properties and expected performance of the embankment, some issues regarding the pre-consolidation pressures and time required for stabilization of settlements in areas where wick drains are not required remain unanswered.

In order to confirm the design assumptions and possibly further optimize the wick drain and surcharge design, it is recommended that portion of the proposed embankment be constructed in an advance contract.

In our opinion, the trial embankment is a prudent investment that should minimize the potential for construction schedule delays.

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<sup>5</sup>

Sridharan, A., Murthy, N.S. and Prakash, K (1987). "Rectangular hyperbola method of consolidation analysis". Geotechnique 37, No. 3, 355-368 and,  
Tan, S.A., (1993). "Ultimate Settlement by Hyperbolic Plot for Clays with Vertical Drains". ASCE Journal of Geotechnical Engineering, Vol. 119, No.5, May, 1993, 950-956



## APPENDIX A

### FIGURES

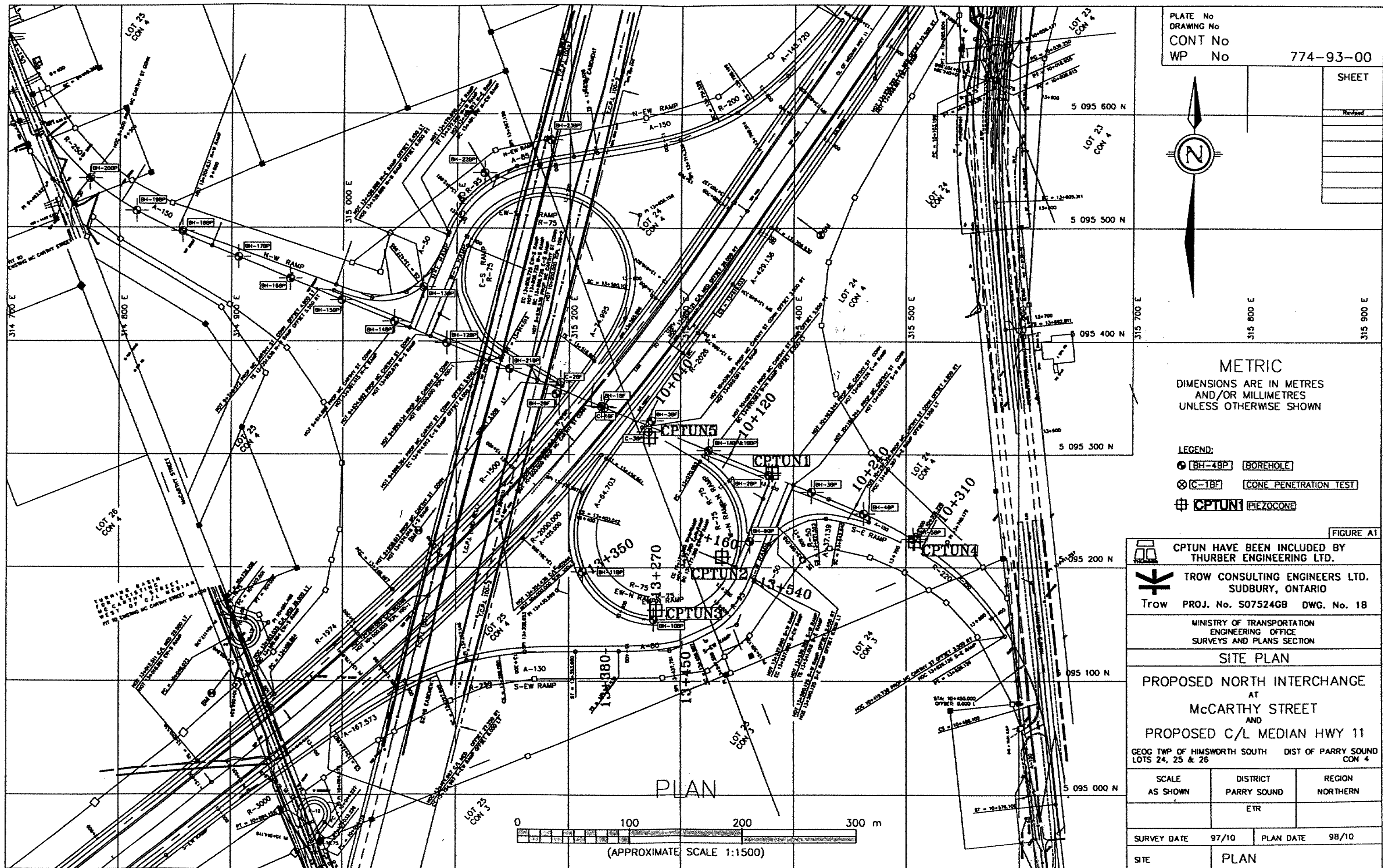


PLATE No  
 DRAWING No  
 CONT No  
 WP No

774-93-00

SHEET



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

- LEGEND:
- BH-4BP BOREHOLE
  - ⊗ C-1BP CONE PENETRATION TEST
  - ⊕ CPTUN1 PIEZOCON

FIGURE A1

CPTUN HAVE BEEN INCLUDED BY THURBER ENGINEERING LTD.		
TROW CONSULTING ENGINEERS LTD. SUDBURY, ONTARIO Trow PROJ. No. S07524GB DWG. No. 18		
MINISTRY OF TRANSPORTATION ENGINEERING OFFICE SURVEYS AND PLANS SECTION		
SITE PLAN		
PROPOSED NORTH INTERCHANGE AT MCCARTHY STREET AND PROPOSED C/L MEDIAN HWY 11		
GEOG TWP OF HIMS WORTH SOUTH DIST OF PARRY SOUND LOTS 24, 25 & 26 CON 4		
SCALE AS SHOWN	DISTRICT PARRY SOUND	REGION NORTHERN
SURVEY DATE 97/10		PLAN DATE 98/10
SITE		PLAN

HIGHWAY 11 - TROUT CREEK BY-PASS - NORTH INTERCHANGE  
SUMMARY OF SUBSURFACE CONDITIONS  
MCCARTHY STREET - APPROXIMATE STATION 10+161

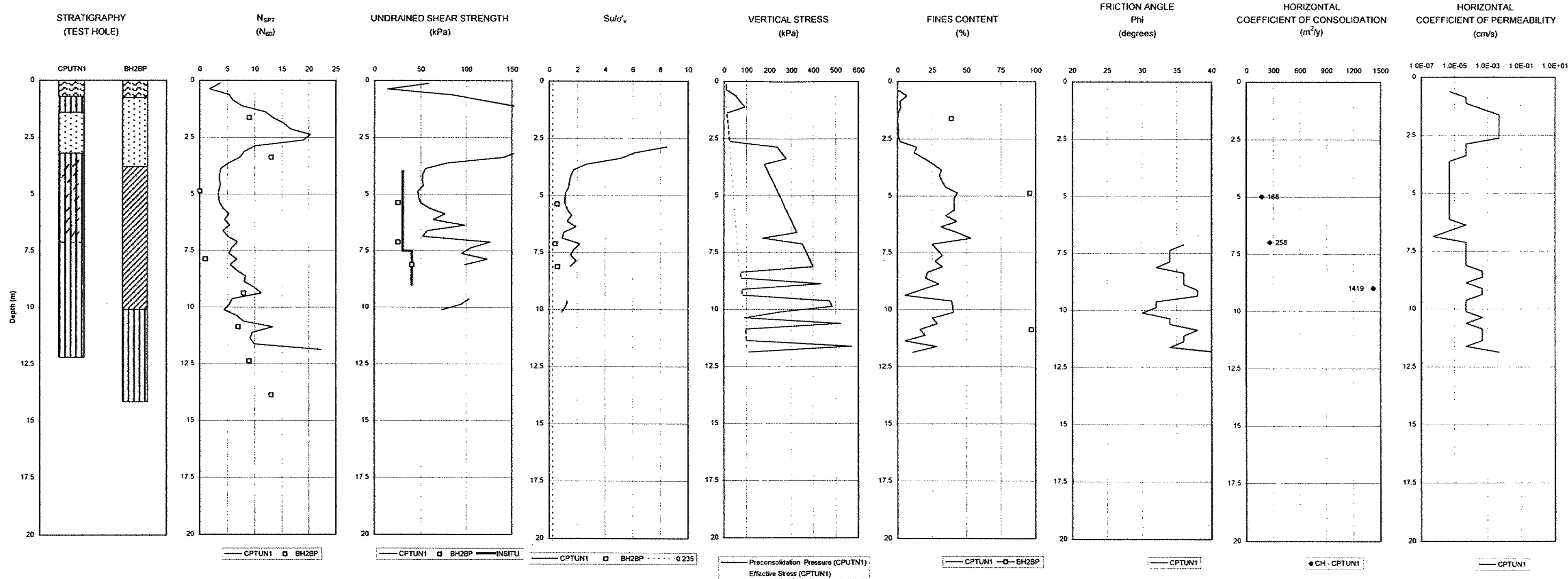


FIGURE A2



**HIGHWAY 11 - TROUT CREEK BY-PASS - NORTH INTERCHANGE  
SUMMARY OF SUBSURFACE CONDITIONS**

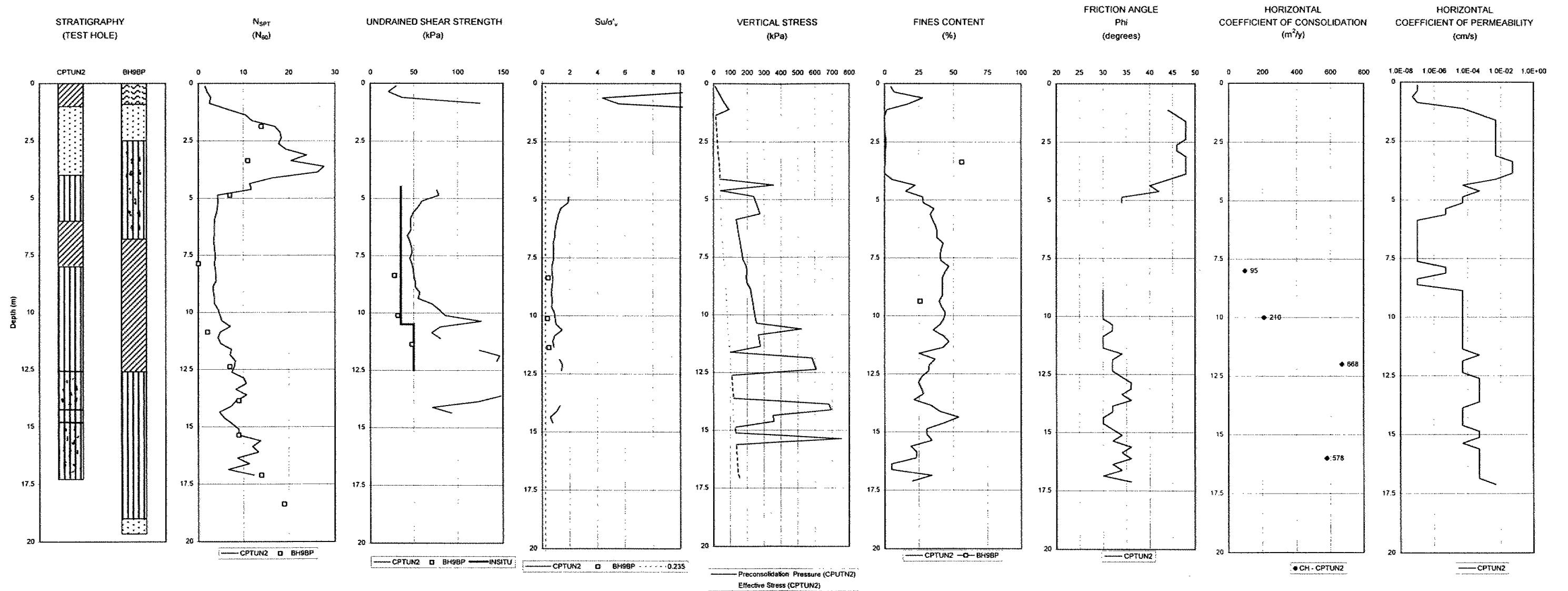


FIGURE A3

**HIGHWAY 11 - TROUT CREEK BY-PASS - NORTH INTERCHANGE  
SUMMARY OF SUBSURFACE CONDITIONS**

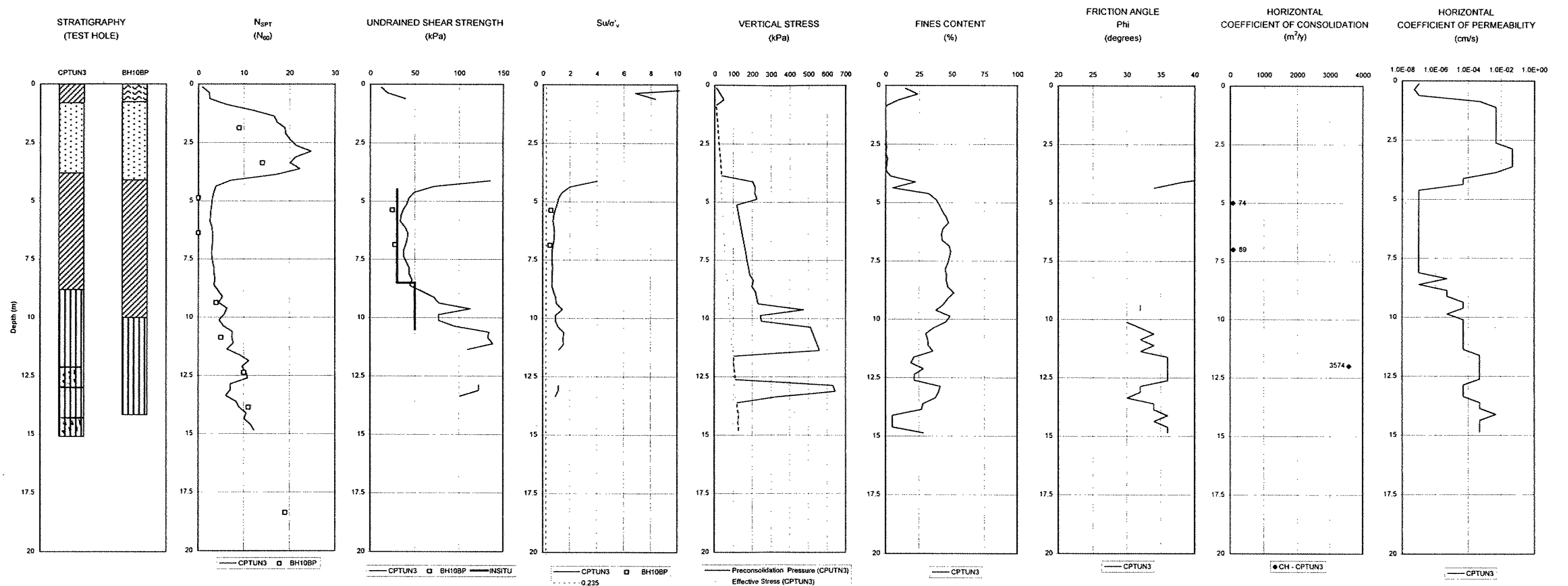


FIGURE A4

**HIGHWAY 11 - TROUT CREEK BY-PASS - NORTH INTERCHANGE  
SUMMARY OF SUBSURFACE CONDITIONS**

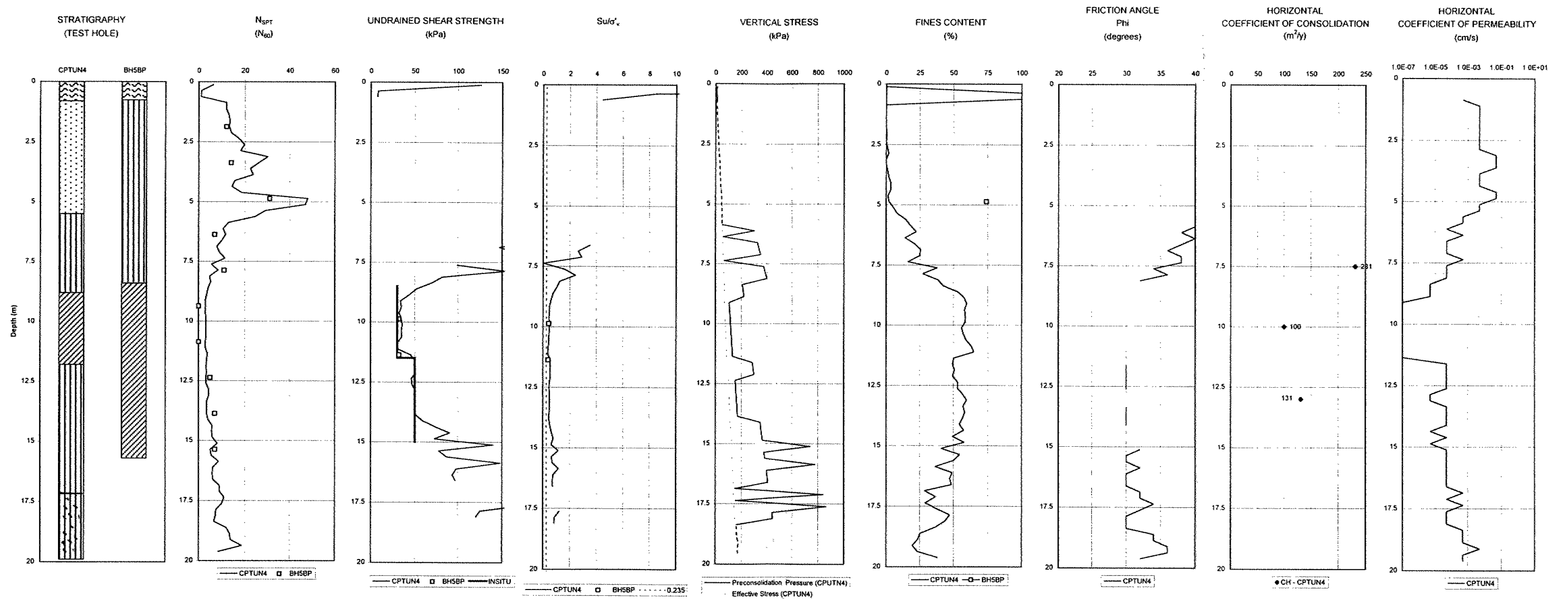


FIGURE A5

**HIGHWAY 11 - TROUT CREEK BY-PASS - NORTH INTERCHANGE  
SUMMARY OF SUBSURFACE CONDITIONS**

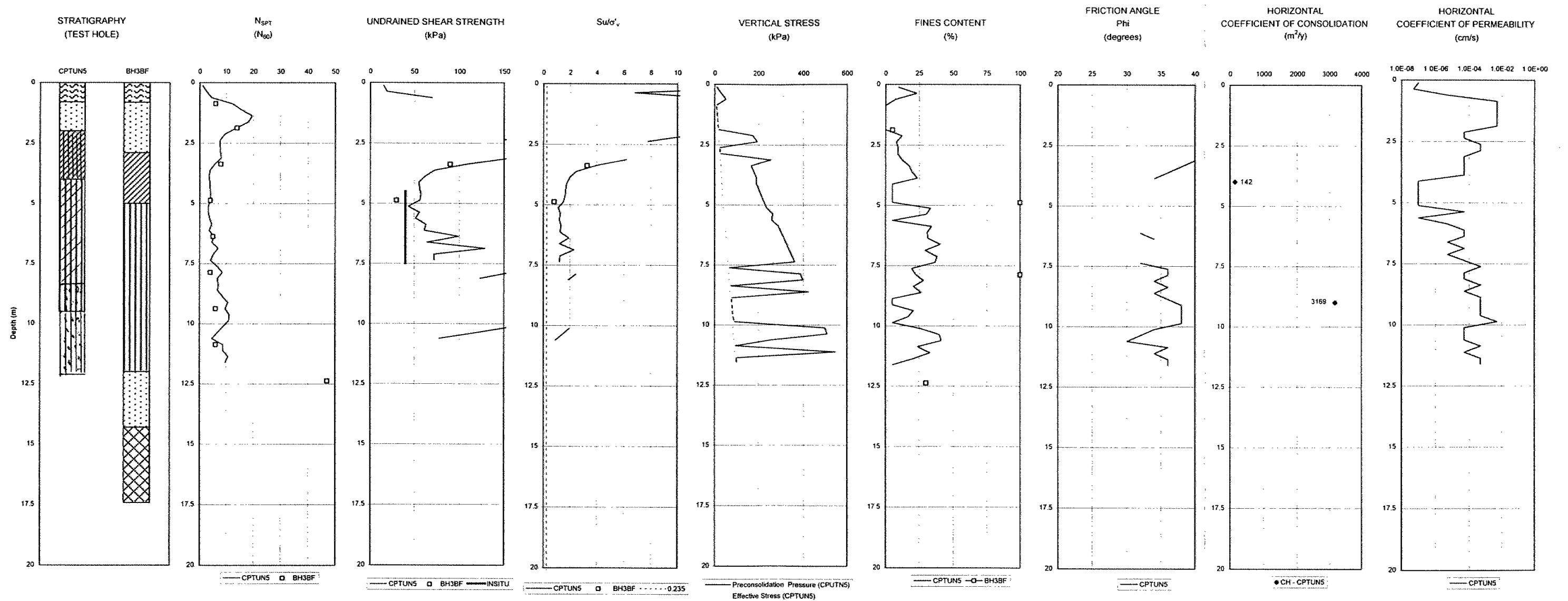
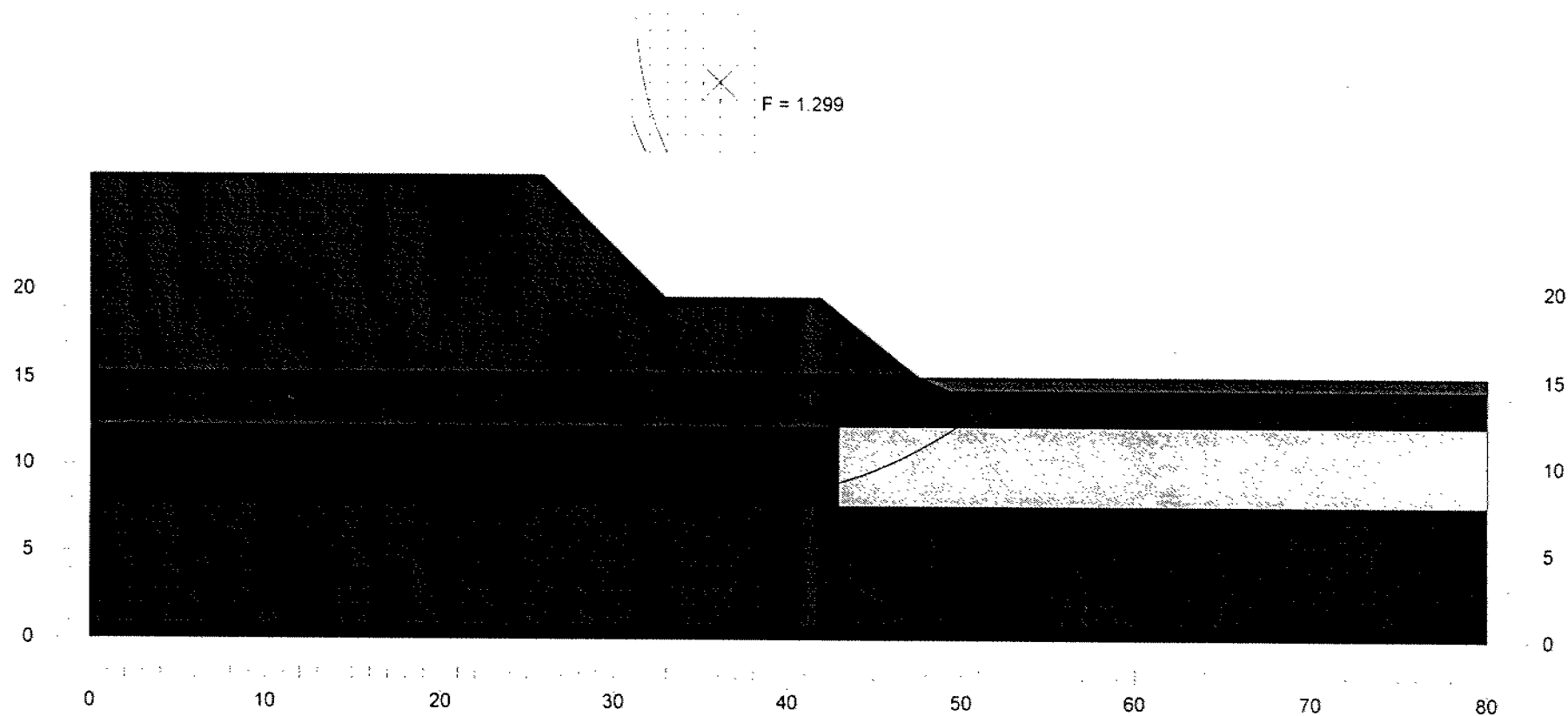


FIGURE A6

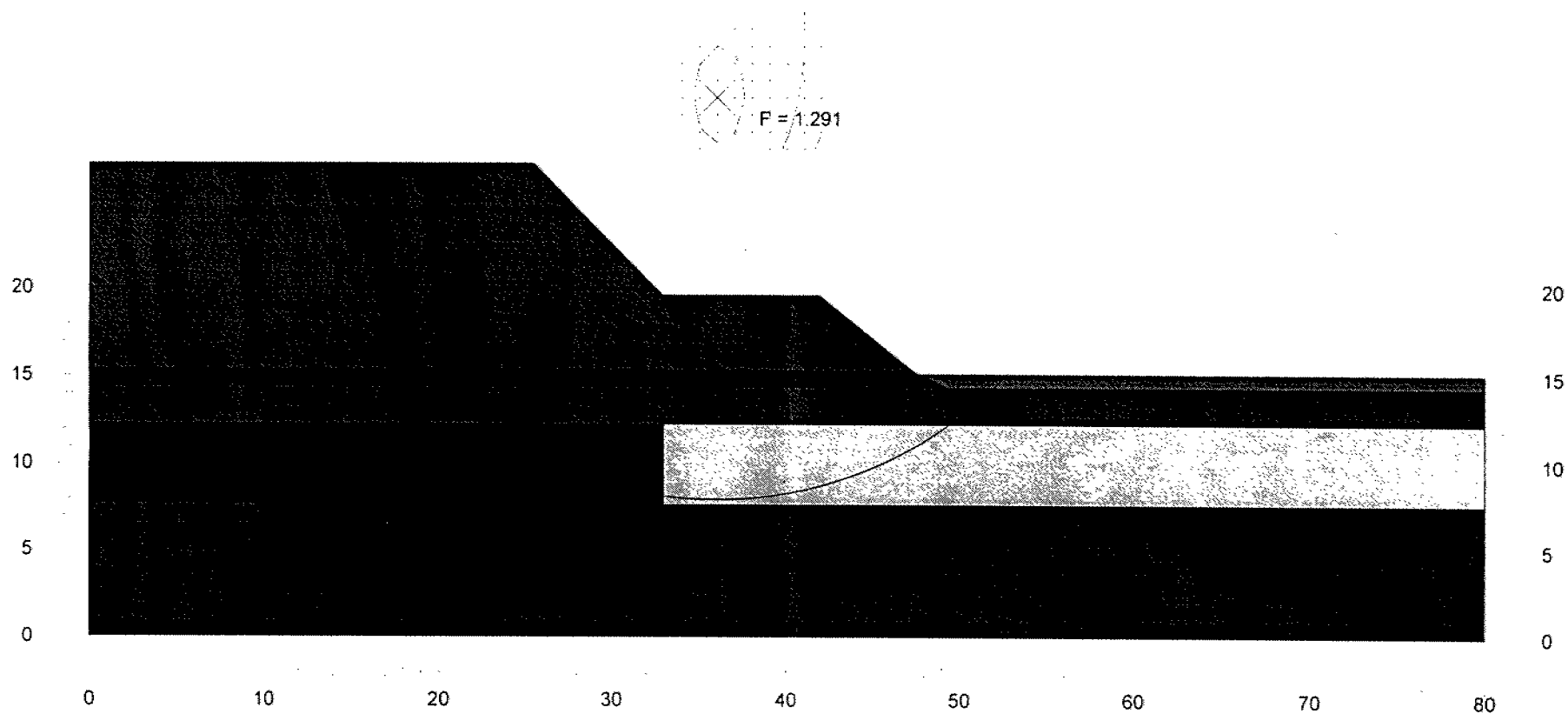
	Gamma kN/m <sup>3</sup>	C kPa	Phi deg	Piezo Surf.	Ru
Rock Fill	20	0	42	1	0
Peat	16	10	0	1	0
Granular B Fill	20	0	32	1	0
Sand	19	0	30	1	0
Silty Clay A-1	18	40	0	1	0
Silty Clay A-2	18	40	0	1	0
Silty Clay A-3	18	40	0	1	0
Silt	18	0	30	1	0
Sand and Gravel	22	0	35	1	0
Bedrock	22	500	35	1	0

Thurber Engineering Ltd. - Toronto  
 19-1104-4  
 Trout Creek North I.C. BH 3BF East Abutment  
 April 19, 1999  
 11.5 m single stage, 9.0 m berm



	Gamma kN/m <sup>3</sup>	C kPa	Phi deg	Piezo Surf.	Ru
Rock Fill	20	0	42	1	0
Peat	16	10	0	1	0
Granular B Fill	20	0	32	1	0
Sand	19	0	30	1	0
Silty Clay A-1	18	40	0	1	0
Silty Clay A-2	18	40	0	1	0
Silty Clay A-3	18	51.2	0	1	0
Silt	18	0	30	1	0
Sand and Gravel	22	0	35	1	0
Bedrock	22	500	35	1	0

Thurber Engineering Ltd. - Toronto  
19-1104-4  
Trout Creek North I.C. BH 3BF East Abutment  
April 19, 1999  
12.0 m (surcharge height - El 327.5), 75% consol.  
9.0 m berm at 19.7



	Gamma kN/m <sup>3</sup>	C kPa	Phi deg	Piezo Surf.	Ru
Rock Fill	20	0	42	0	0
Peat	16	10	0	1	0
Granular B Fill	20	0	32	1	0
Silty Sand	19	0	29	1	0
Sand	19	0	30	1	0
Silty Clay A-1	18	30	0	1	0
Silty Clay A-2	18	30	0	1	0
Silty Clay A-3	18	30	0	1	0
Silty Clay B-1	18	40	0	1	0
Silty Clay B-2	18	40	0	1	0
Silty Clay B-3	18	40	0	1	0
Silt	19	0	30	1	0

Thurber Engineering Ltd. - Toronto  
 19-1104-4  
 Trout Creek North I.C. BH 2BP  
 May 4, 1999  
 8.8 m above original (EI 323.5). Single stage  
 9.0 m wide berm at EI 316.7

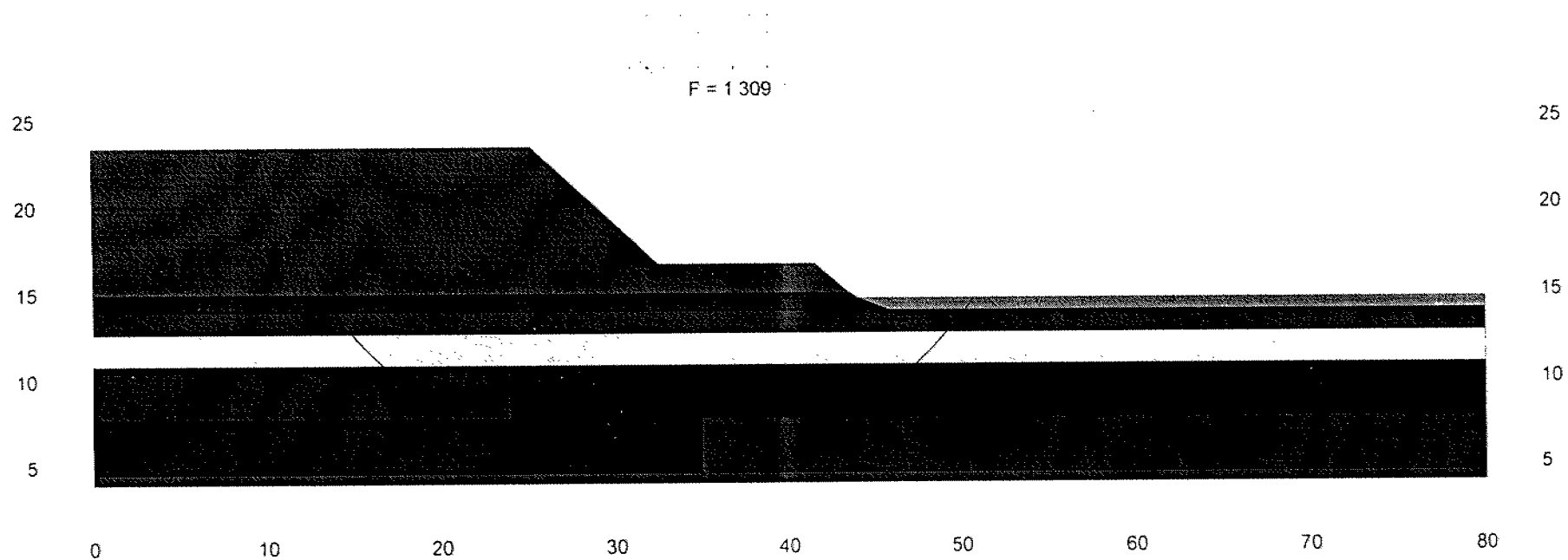
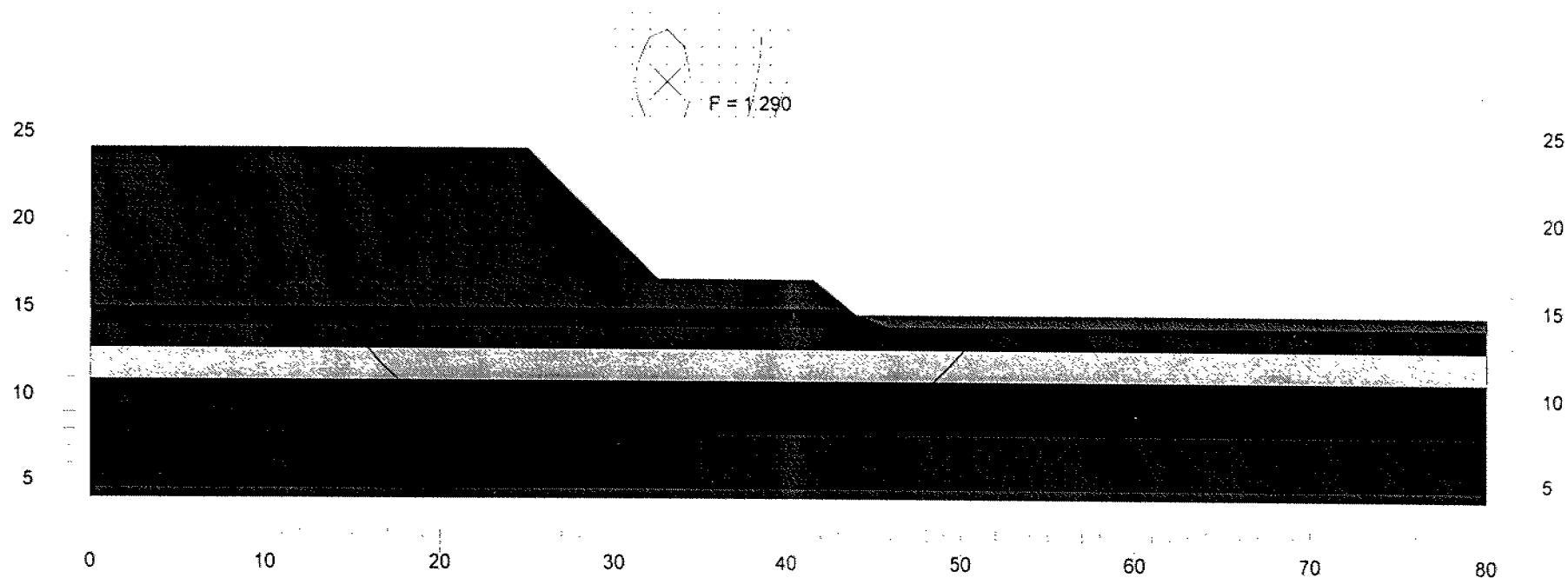


FIGURE A9

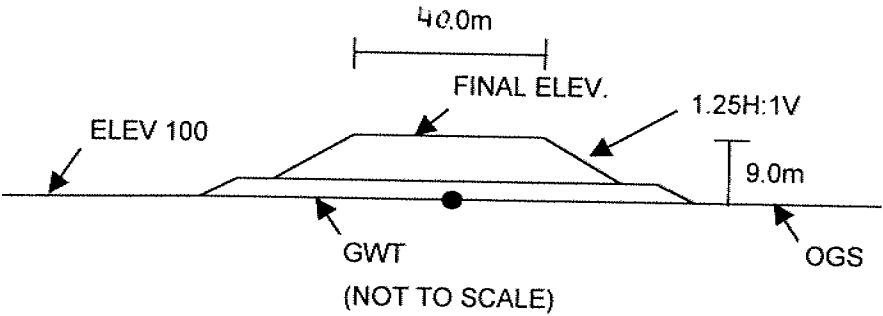
	Gamma kN/m <sup>3</sup>	C kPa	Phi deg	Piezo Surf.	Ru
Rock Fill	20	0	42	0	0
Peat	16	10	0	1	0
Granular B Fill	20	0	32	1	0
Silty Sand	19	0	29	1	0
Sand	19	0	30	1	0
Silty Clay A-1	18	30	0	1	0
Silty Clay A-2	18	30.2	0	1	0
Silty Clay A-3	18	42.2	0	1	0
Silty Clay B-1	18	40	0	1	0
Silty Clay B-2	18	40	0	1	0
Silty Clay B-3	18	48.3	0	1	0
Silt	19	0	30	1	0

Thurber Engineering Ltd. - Toronto  
 19-1104-4  
 Trout Creek North I.C. BH 2BP  
 May 4, 1999  
 9.5 m above original (EI 324.2) 75% under 8.8m  
 9.0 m wide berm at EI 316.7



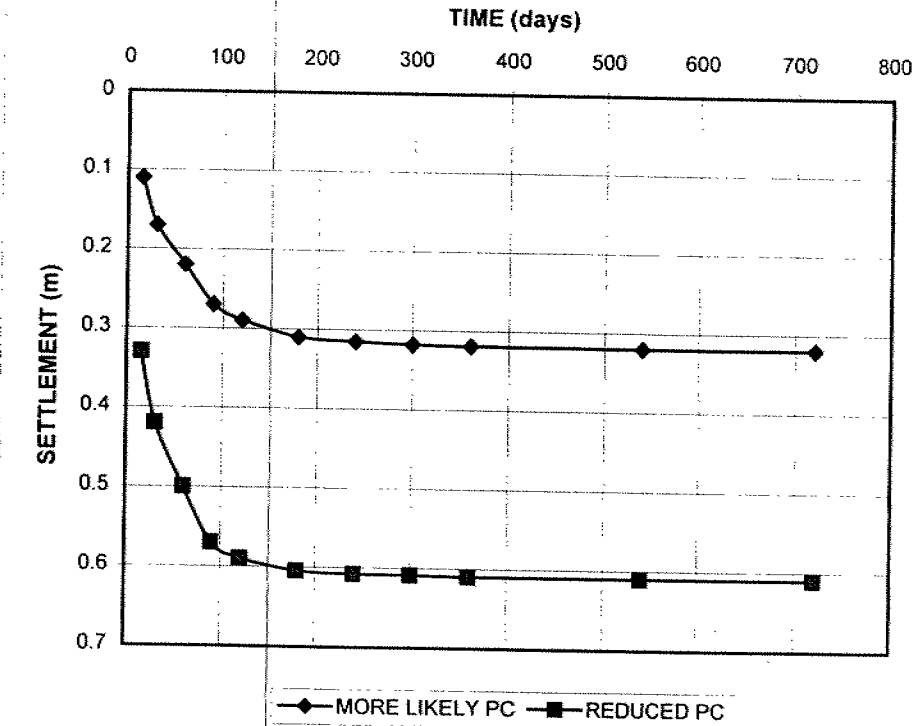
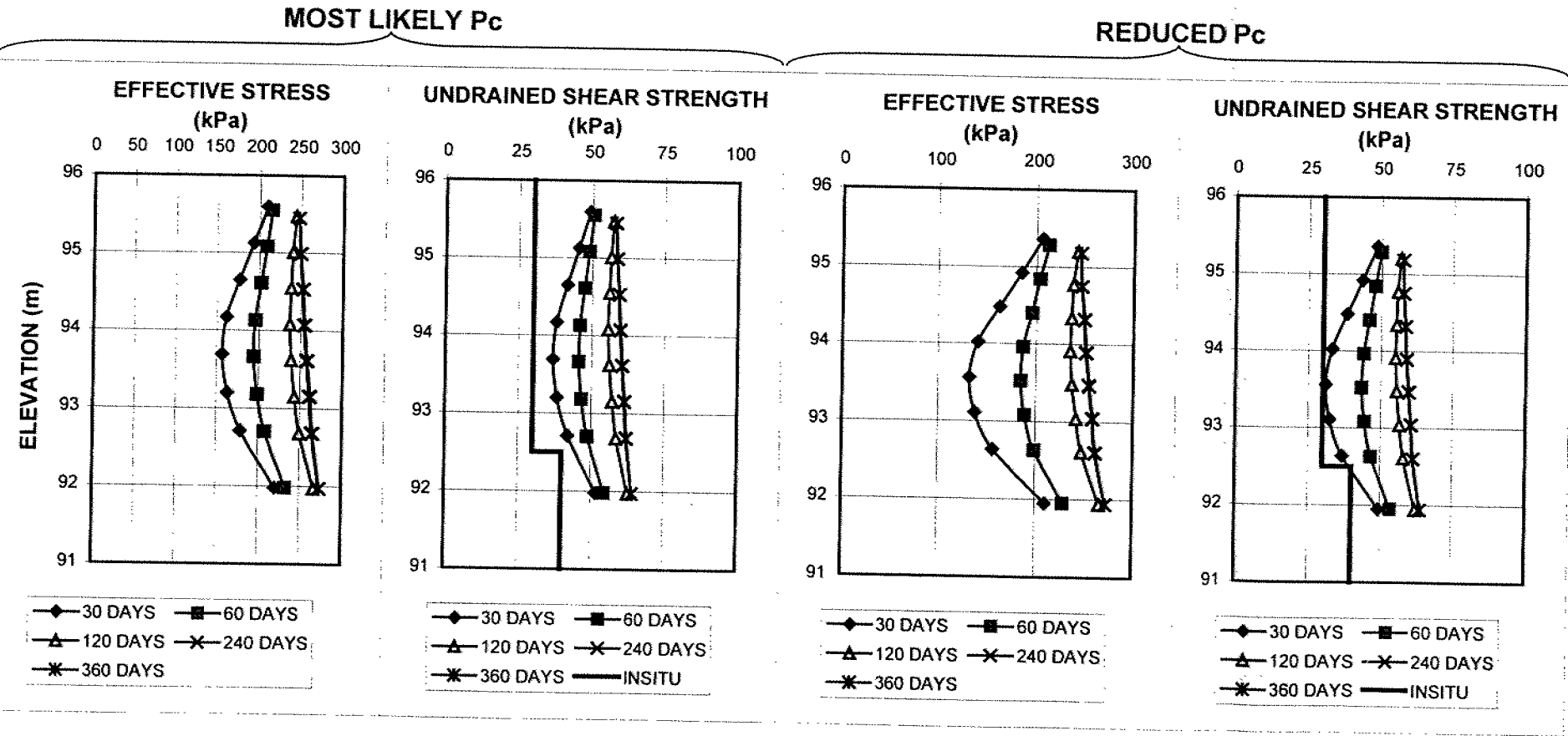


HIGHWAY 11 - TROUT CREEK BY-PASS  
 NORTH INTERCHANGE - APPROX. STATION 10+161, MCCARHTY STREET (CPTUN1)  
 SETTLEMENTS DUE TO PRIMARY CONSOLIDATION - NO WICK DRAINS  
 (AT THE CENTRELINE OF THE EMBANKMENT)



CONSTRUCTION STAGES FOR MOST LIKELY $P_c$		
STAGE	EMBANKMENT HEIGHT (m)	TIME (DAYS)
1	9	0
2	10.5	60

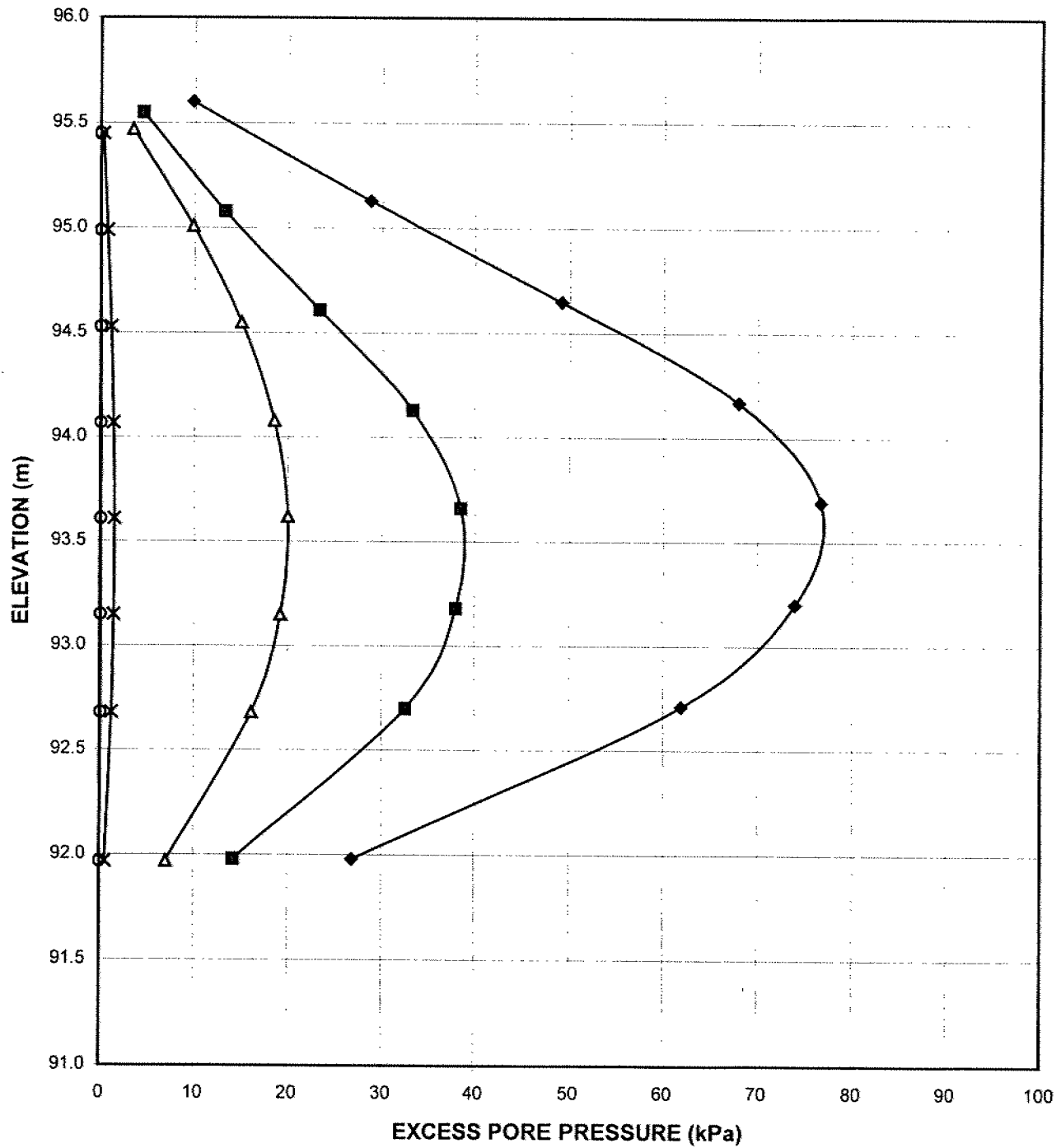
CONSTRUCTION STAGES FOR REDUCED $P_c$		
STAGE	EMBANKMENT HEIGHT (m)	TIME (DAYS)
1	9	0
2	10.5	60



MASTER PLOT

FIGURE A11  
 23/04/99

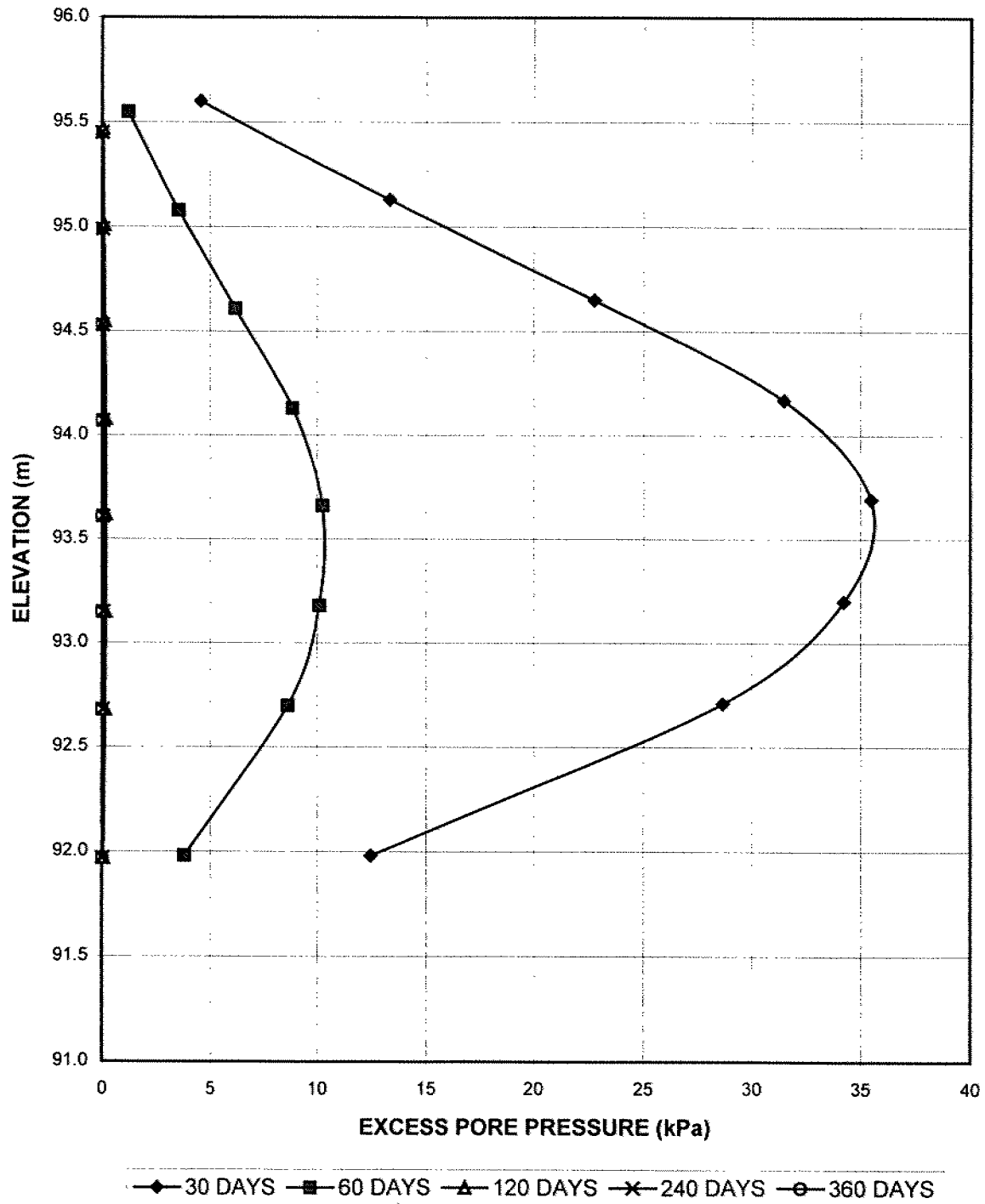
HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - APPROX. STATION 10+161 MCCARTHY ST (CPTUN1)  
EXCESS PORE PRESSURES - NO WICK DRAINS - MOST LIKELY  $P_c$   
(AT THE CENTRELINE OF THE EMBANKMENT)



EPP - CHART

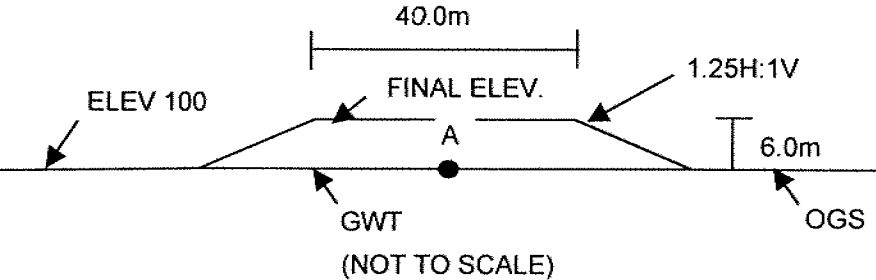
FIGURE A11-B

HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - APPROX. STATION 10+161 MCCARTHY ST (CPTUN1)  
EXCESS PORE PRESSURES - WICK DRAIN  $s=4.0\text{m}$   
MOST LIKELY  $P_c$   
(AT THE CENTRELINE OF THE EMBANKMENT)



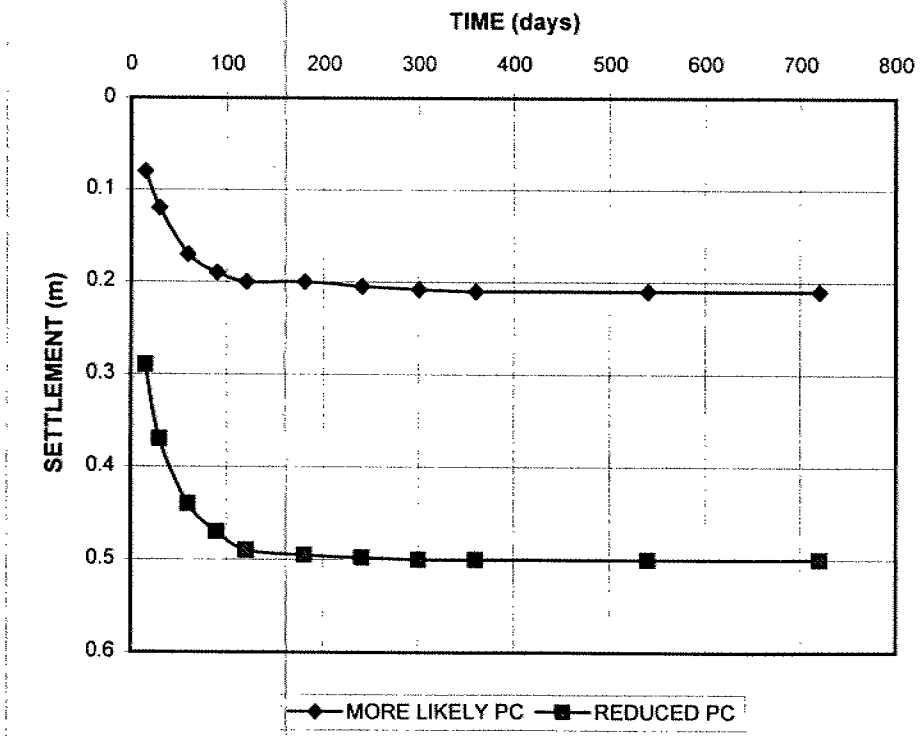
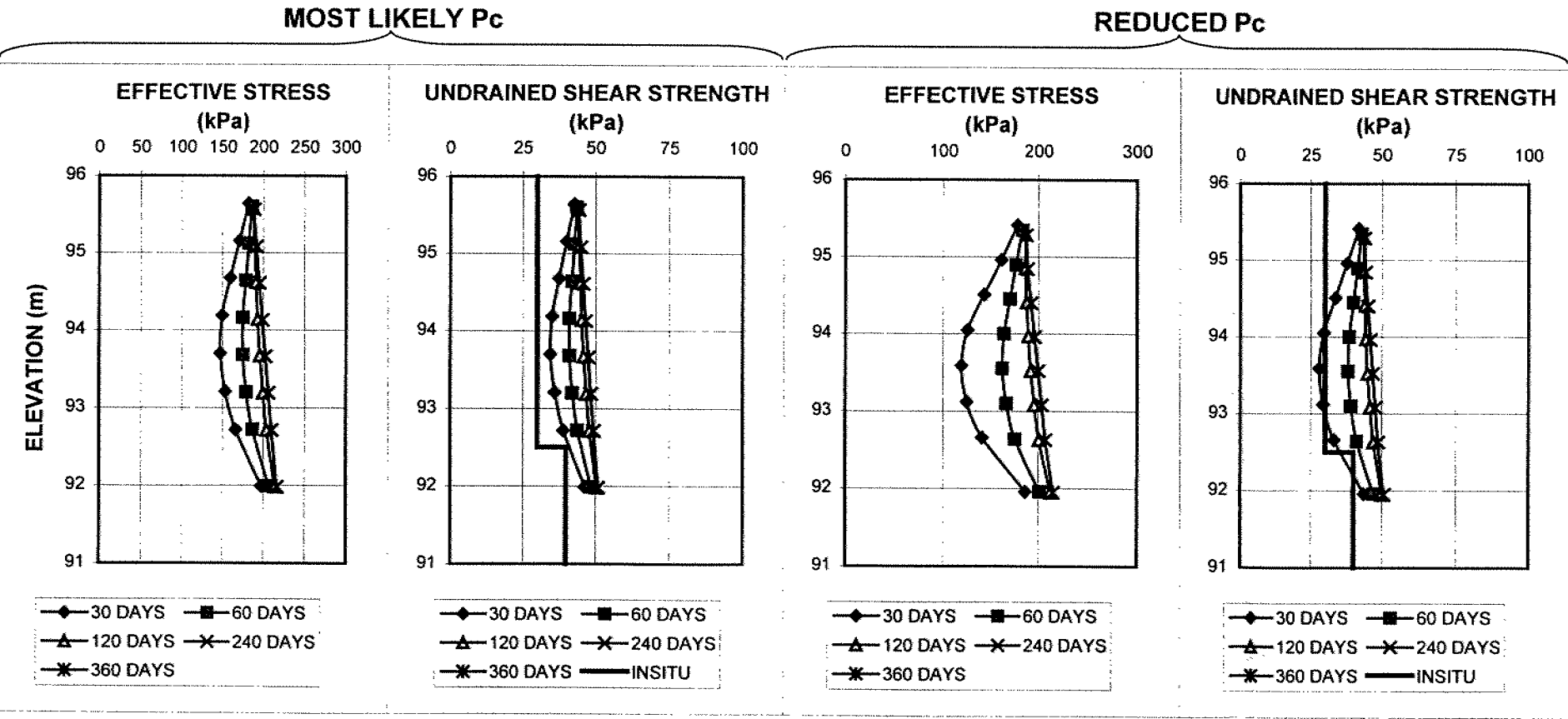
EPP - CHART (2)

HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - APPROX. STATION 10+161, MCCARTHY STREET (CPTUN1)  
SETTLEMENTS DUE TO PRIMARY CONSOLIDATION - NO WICK DRAINS  
(AT THE CENTRELINE OF THE EMBANKMENT)



CONSTRUCTION STAGES FOR MOST LIKELY $P_c$		
STAGE	EMBANKMENT HEIGHT (m)	TIME (DAYS)
1	7.5	0

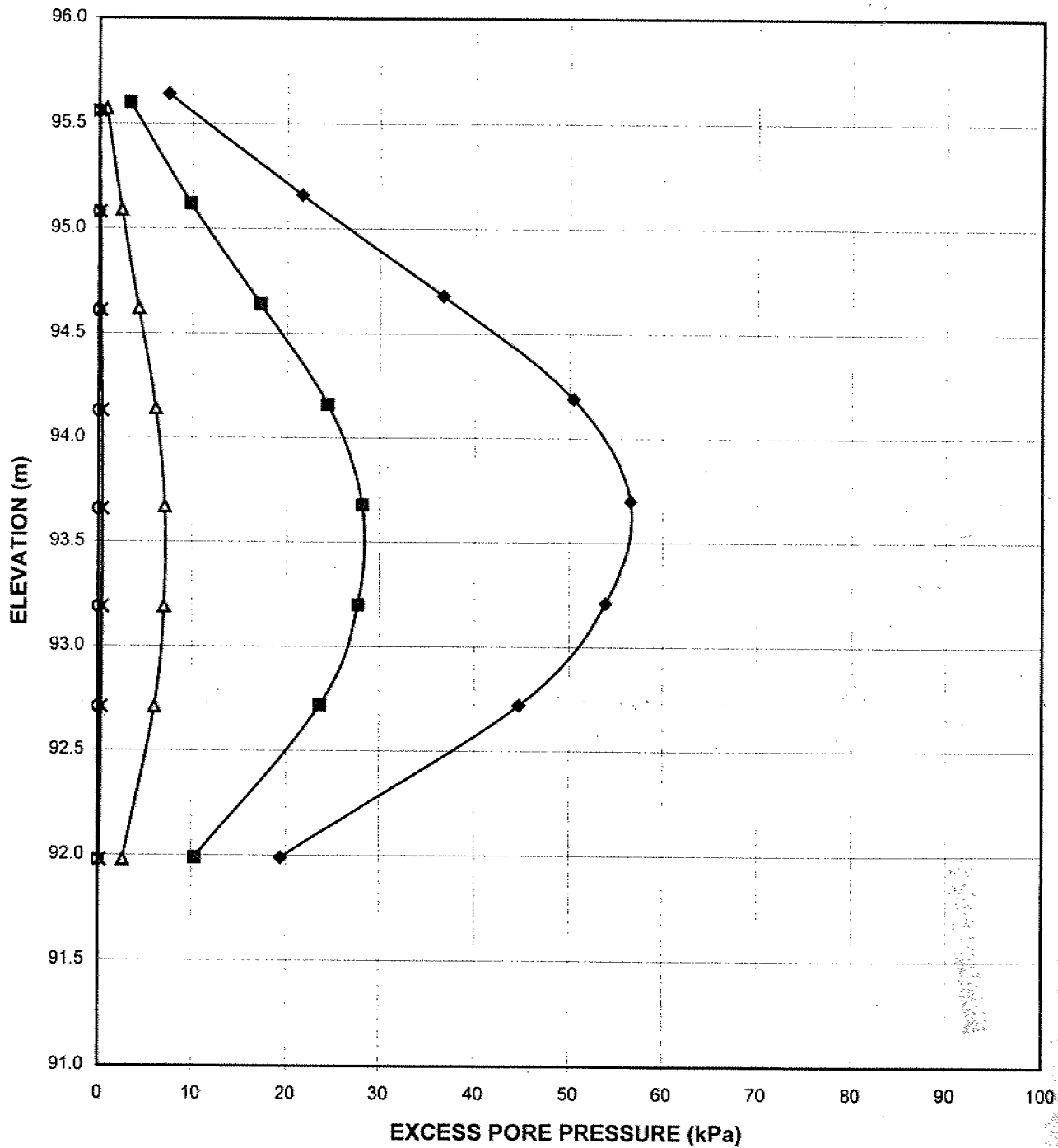
CONSTRUCTION STAGES FOR REDUCED $P_c$		
STAGE	EMBANKMENT HEIGHT (m)	TIME (DAYS)
1	7.5	0



MASTER PLOT

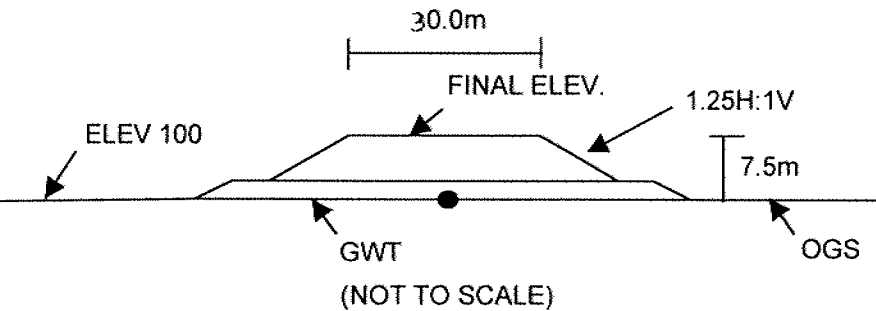
FIGURE A12  
23/04/99

HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - APPROX. STATION 10+161, MCCARTHY ST (CPTUN1)  
EXCESS PORE PRESSURES - NO WICK DRAINS - MOST LIKELY  $P_c$   
(AT THE CENTRELINE OF THE EMBANKMENT)



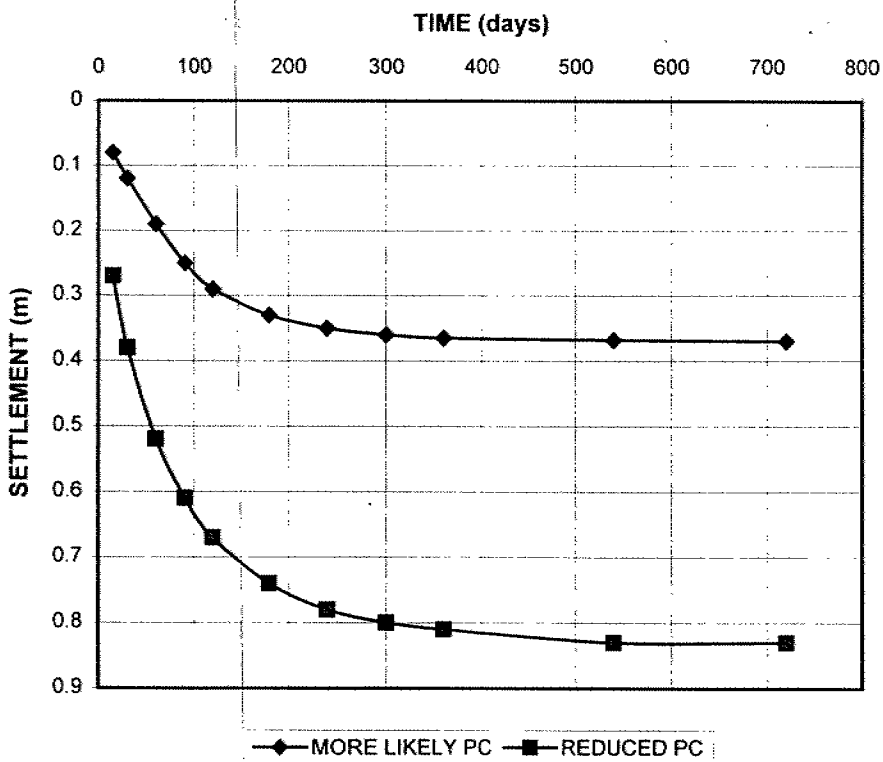
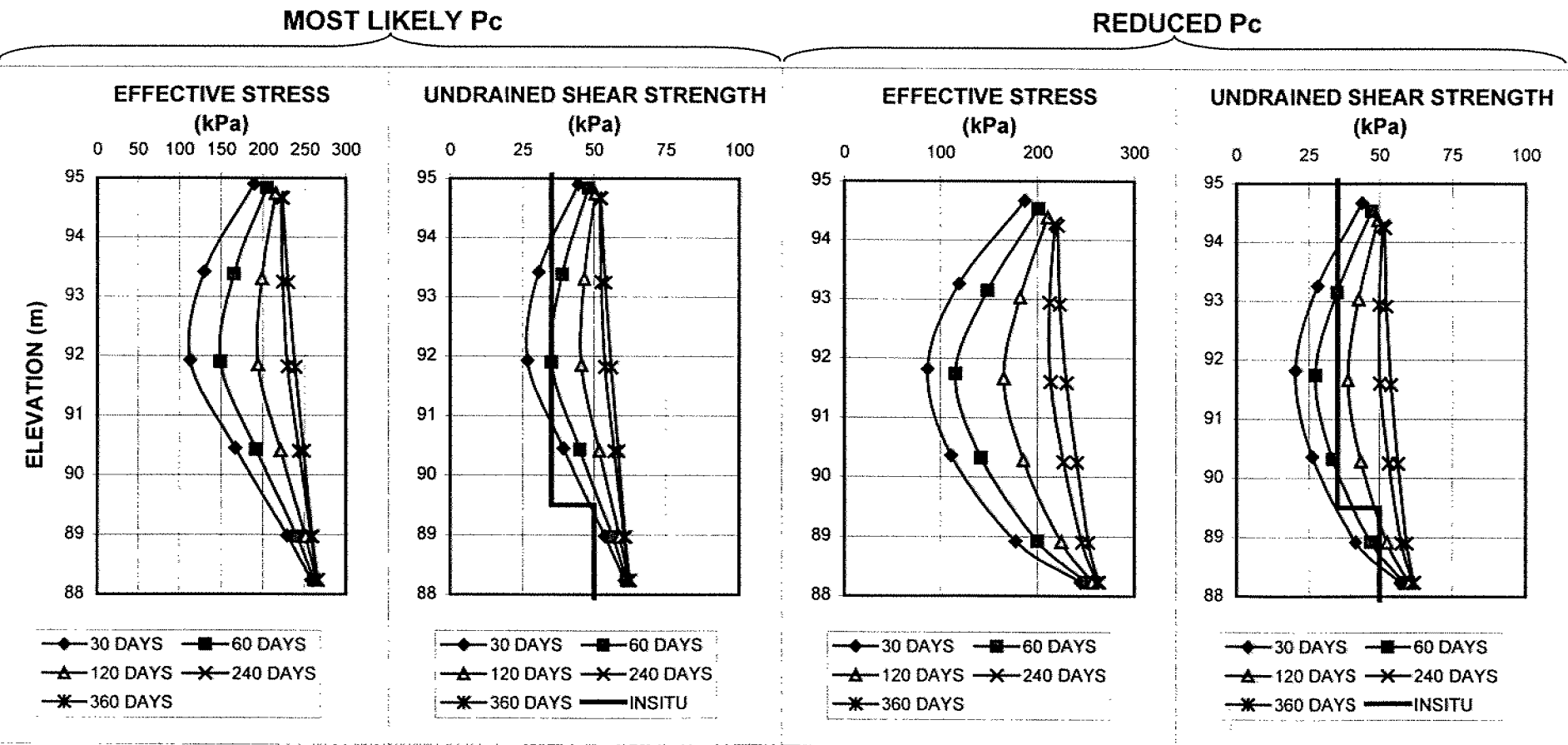
—◆— 30 DAYS —■— 60 DAYS —▲— 120 DAYS —×— 240 DAYS —○— 360 DAYS

HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - APPROX. STATION 13+180, E-N RAMP (CPTUN2)  
SETTLEMENTS DUE TO PRIMARY CONSOLIDATION -NO WICK DRAINS  
(AT THE CENTRELINE OF THE EMBANKMENT)



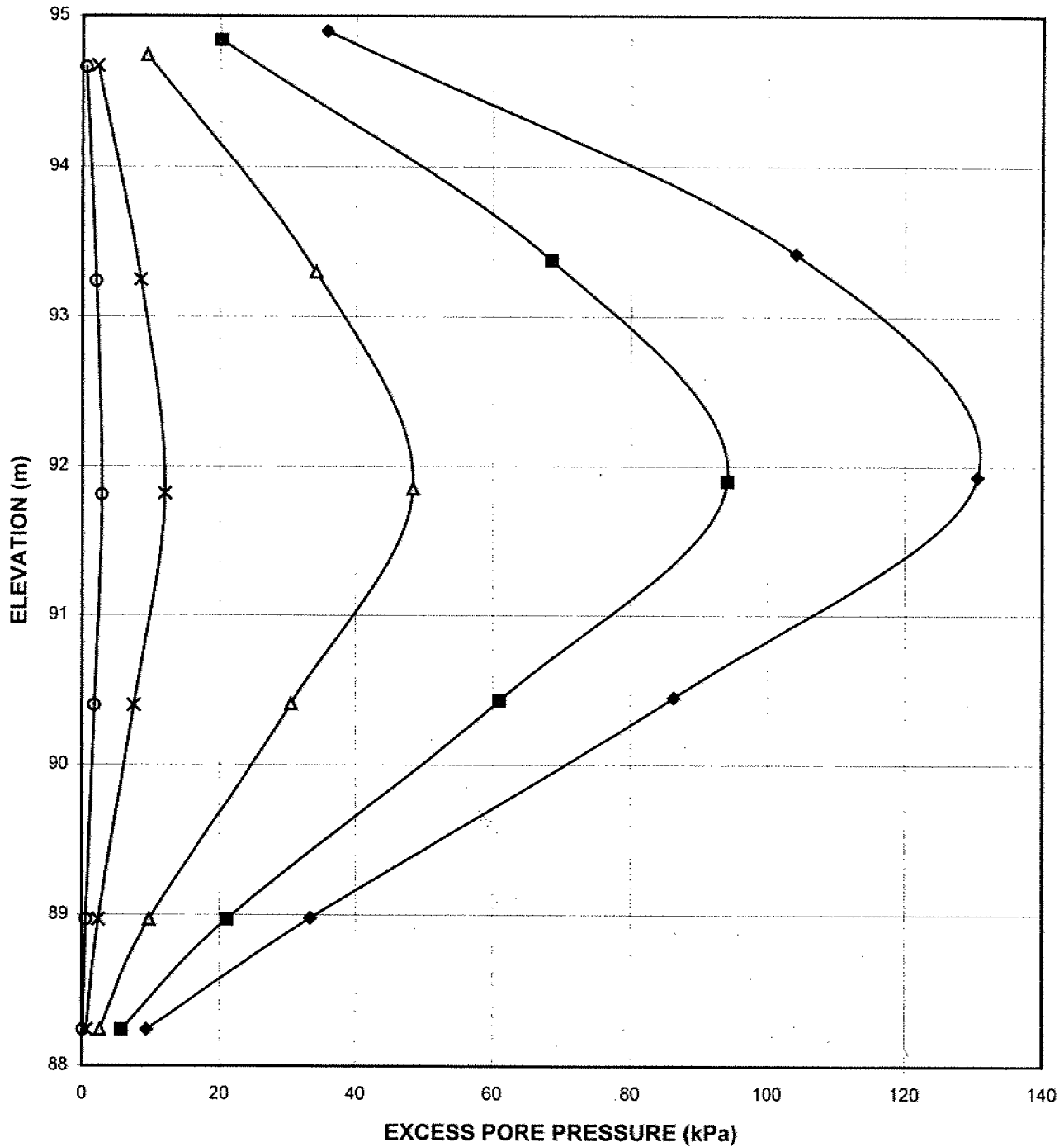
CONSTRUCTION STAGES FOR MOST LIKELY $P_c$		
STAGE	EMBANKMENT HEIGHT (m)	TIME (DAYS)
1	9.0	0

CONSTRUCTION STAGES FOR REDUCED $P_c$		
STAGE	EMBANKMENT HEIGHT (m)	TIME (DAYS)
1	9.0	0



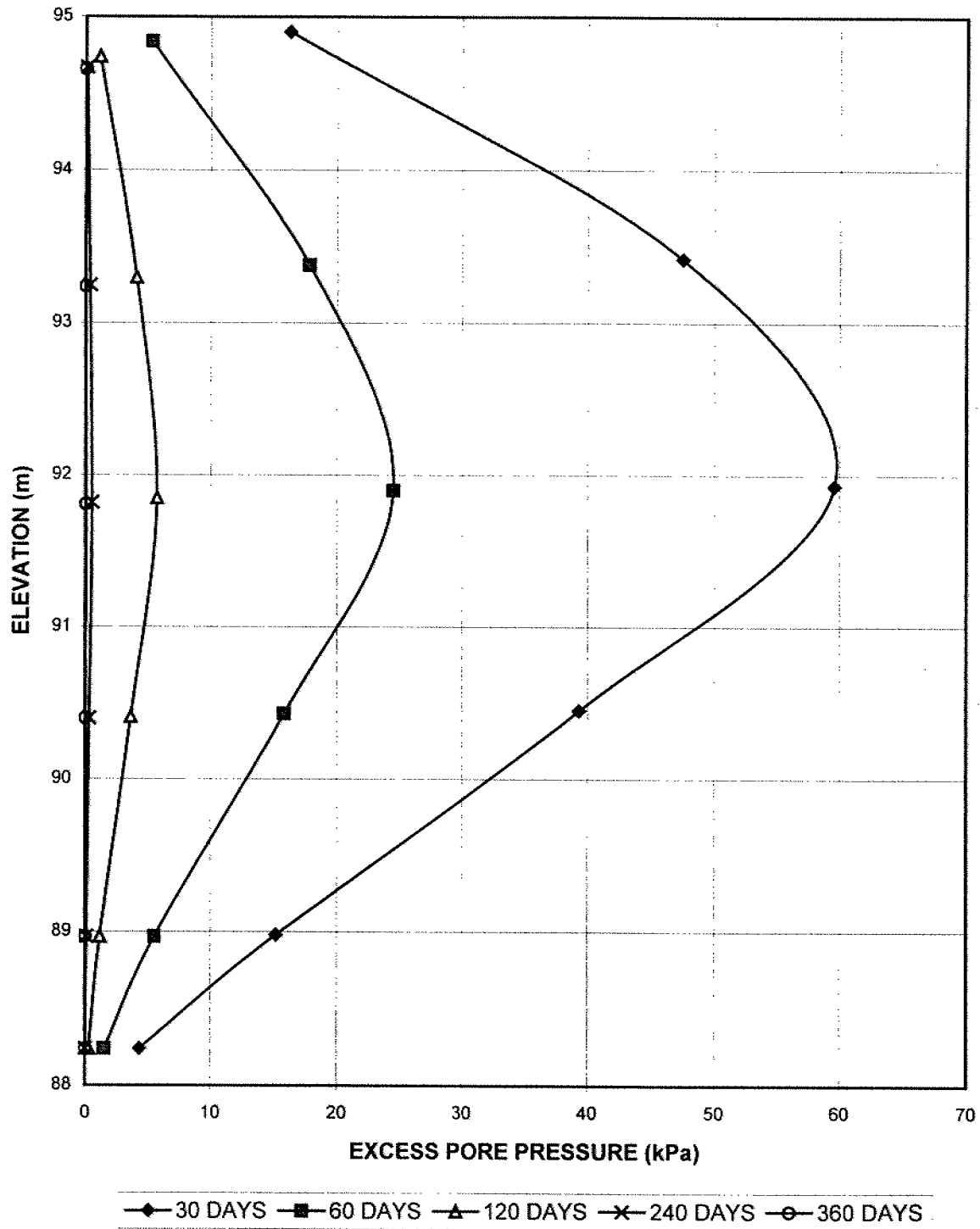
MASTER PLOT

HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - APPROX. STATION 13+180, E-N RAMP (CPTUN2)  
EXCESS PORE PRESSURES - NO WICK DRAINS - MOST LIKELY  $P_c$   
(AT THE CENTRELINE OF THE EMBANKMENT)



—◆— 30 DAYS —■— 60 DAYS —▲— 120 DAYS —×— 240 DAYS —○— 360 DAYS

HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - APPROX. STATION 13+180, E-N RAMP (CPTUN2)  
EXCESS PORE PRESSURES - WICK DRAINS  $s=3.1\text{m}$  - MOST LIKELY  $P_c$   
(AT THE CENTRELINE OF THE EMBANKMENT)

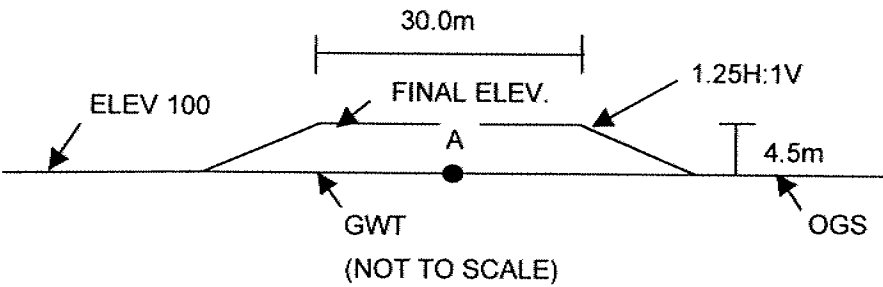


EPP - CHART (2)

FIGURE A13-C

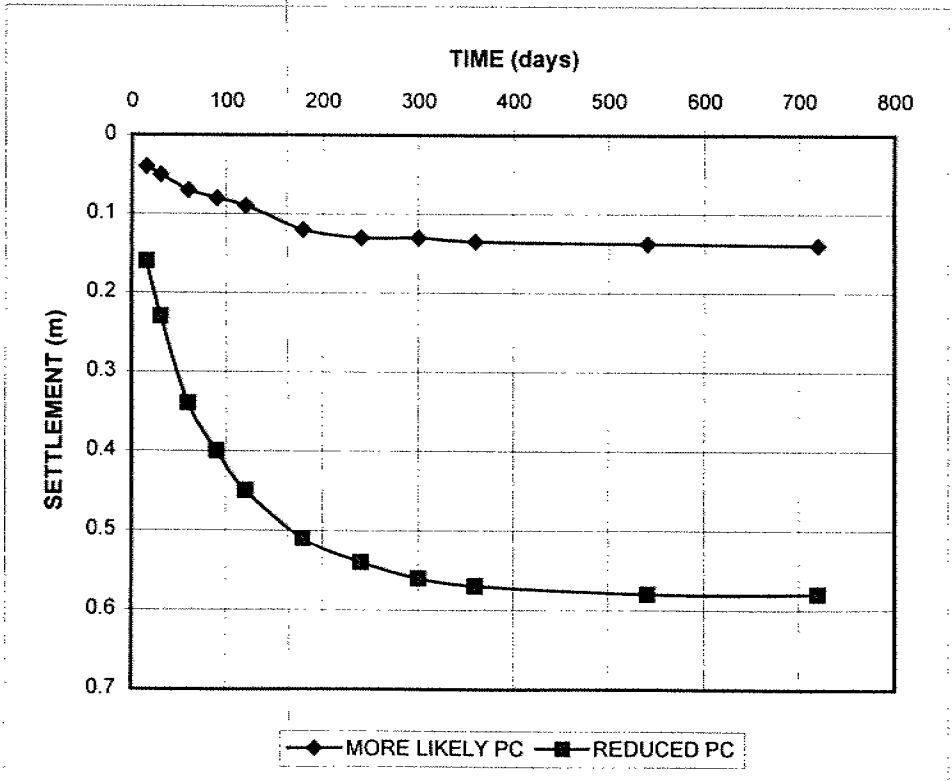
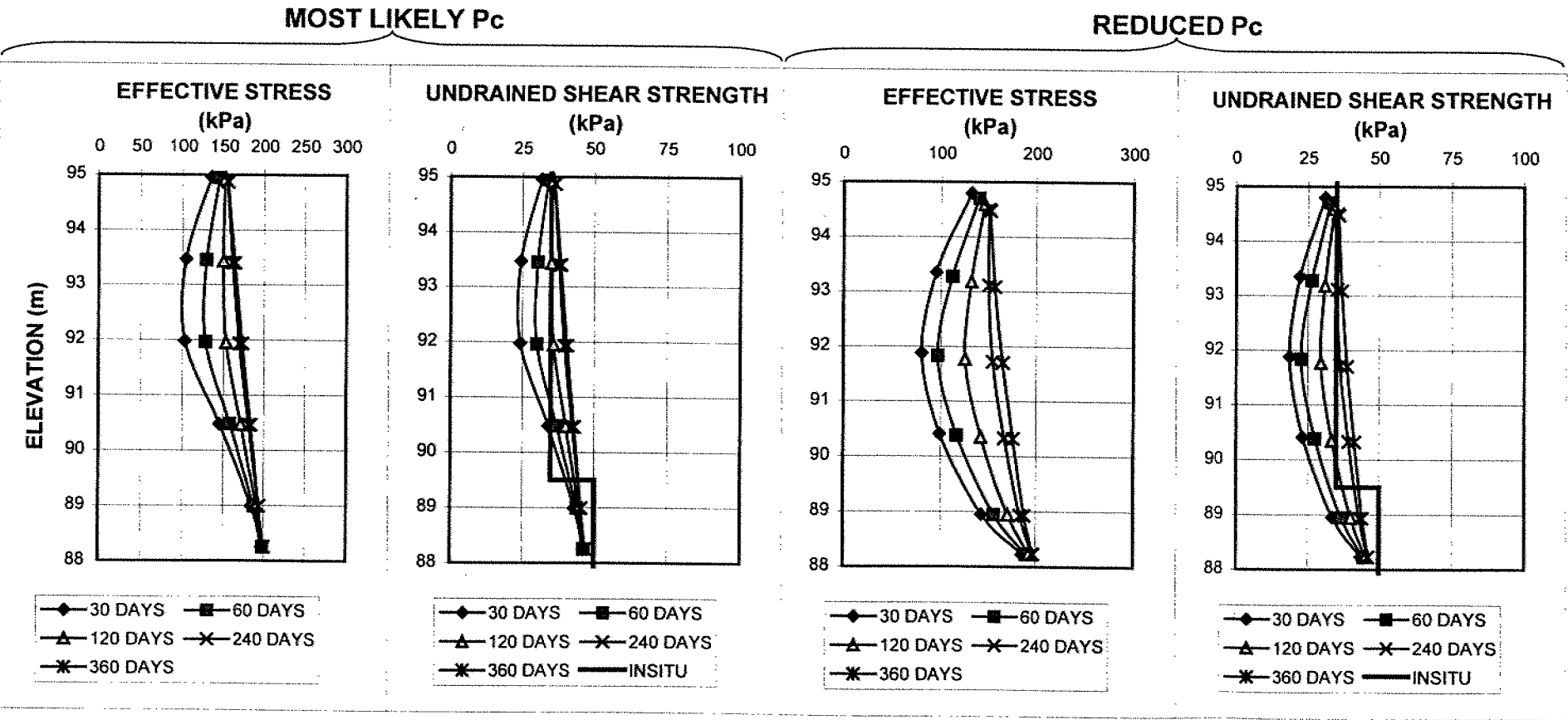


HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - APPROX. STATION 13+180, E-N RAMP (CPTUN2)  
SETTLEMENTS DUE TO PRIMARY CONSOLIDATION - NO WICK DRAINS  
(AT THE CENTRELINE OF THE EMBANKMENT)



CONSTRUCTION STAGES FOR MOST LIKELY $P_c$		
STAGE	EMBANKMENT HEIGHT (m)	TIME (DAYS)
1	5.5	0

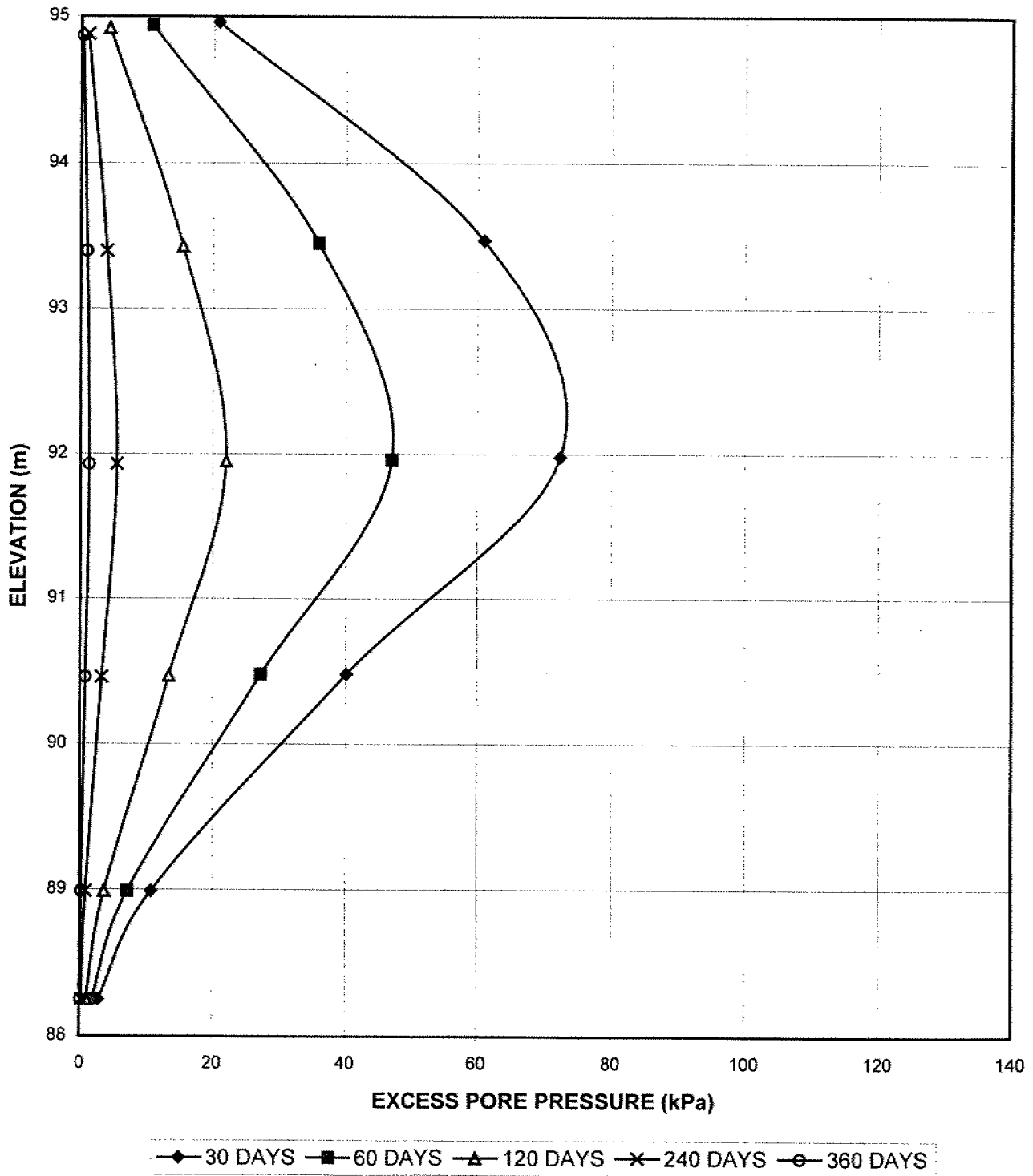
CONSTRUCTION STAGES FOR REDUCED $P_c$		
STAGE	EMBANKMENT HEIGHT (m)	TIME (DAYS)
1	5.5	0



MASTER PLOT

FIGURE A14

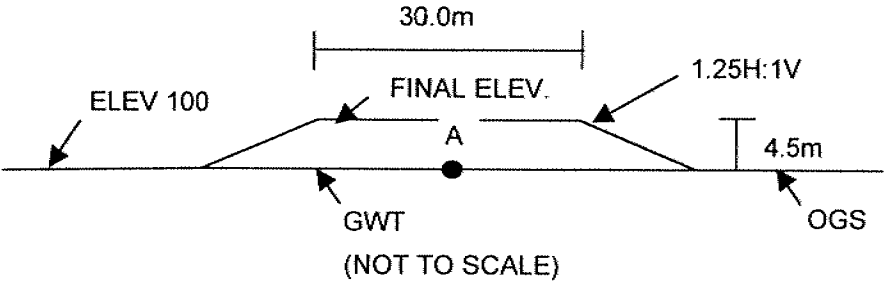
HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - APPROX. STATION 13+180, E-N RAMP (CPTUN2)  
EXCESS PORE PRESSURES - NO WICK DRAINS - MOST LIKELY  $P_c$   
(AT THE CENTRELINE OF THE EMBANKMENT)



EPP - CHART

FIGURE A14-B

HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - APPROX. STATION 13+270, EW-N RAMP (CPTUN3)  
SETTLEMENTS DUE TO PRIMARY CONSOLIDATION - NO WICK DRAINS  
(AT THE CENTRELINE OF THE EMBANKMENT)



CONSTRUCTION STAGES FOR MOST LIKELY $P_c$		
STAGE	EMBANKMENT HEIGHT (m)	TIME (DAYS)
1	5.5	0

CONSTRUCTION STAGES FOR REDUCED $P_c$		
STAGE	EMBANKMENT HEIGHT (m)	TIME (DAYS)
1	5.5	0

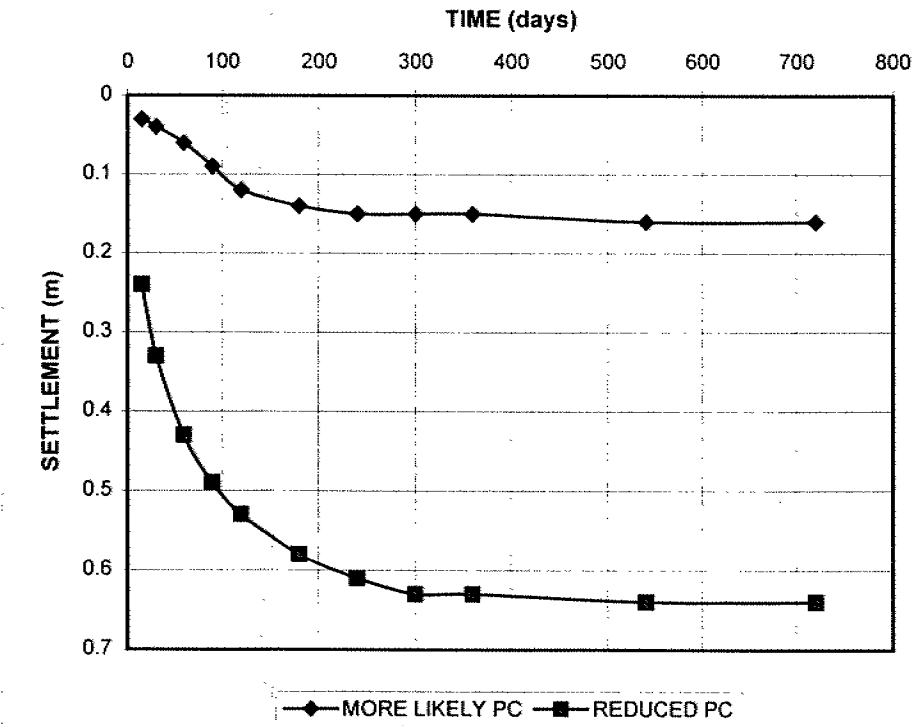
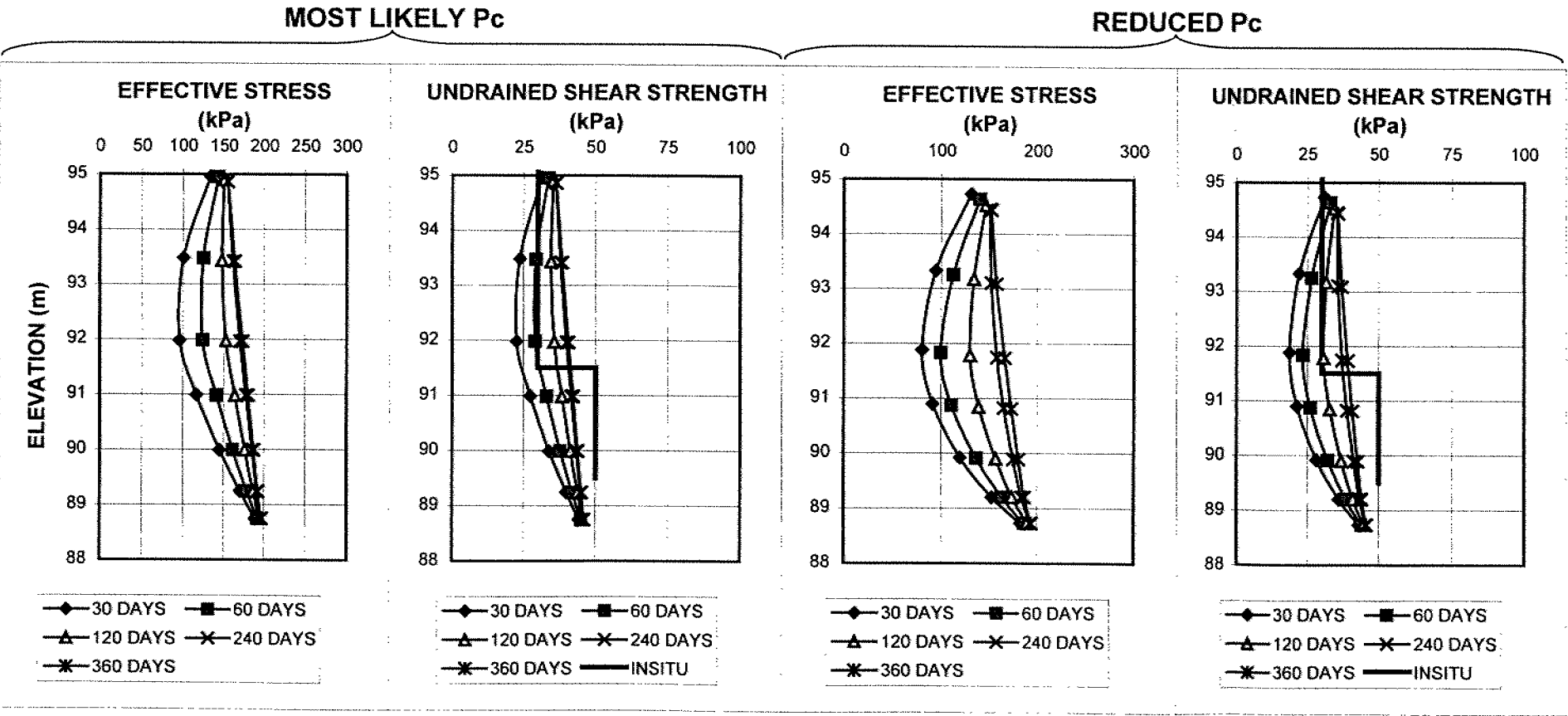
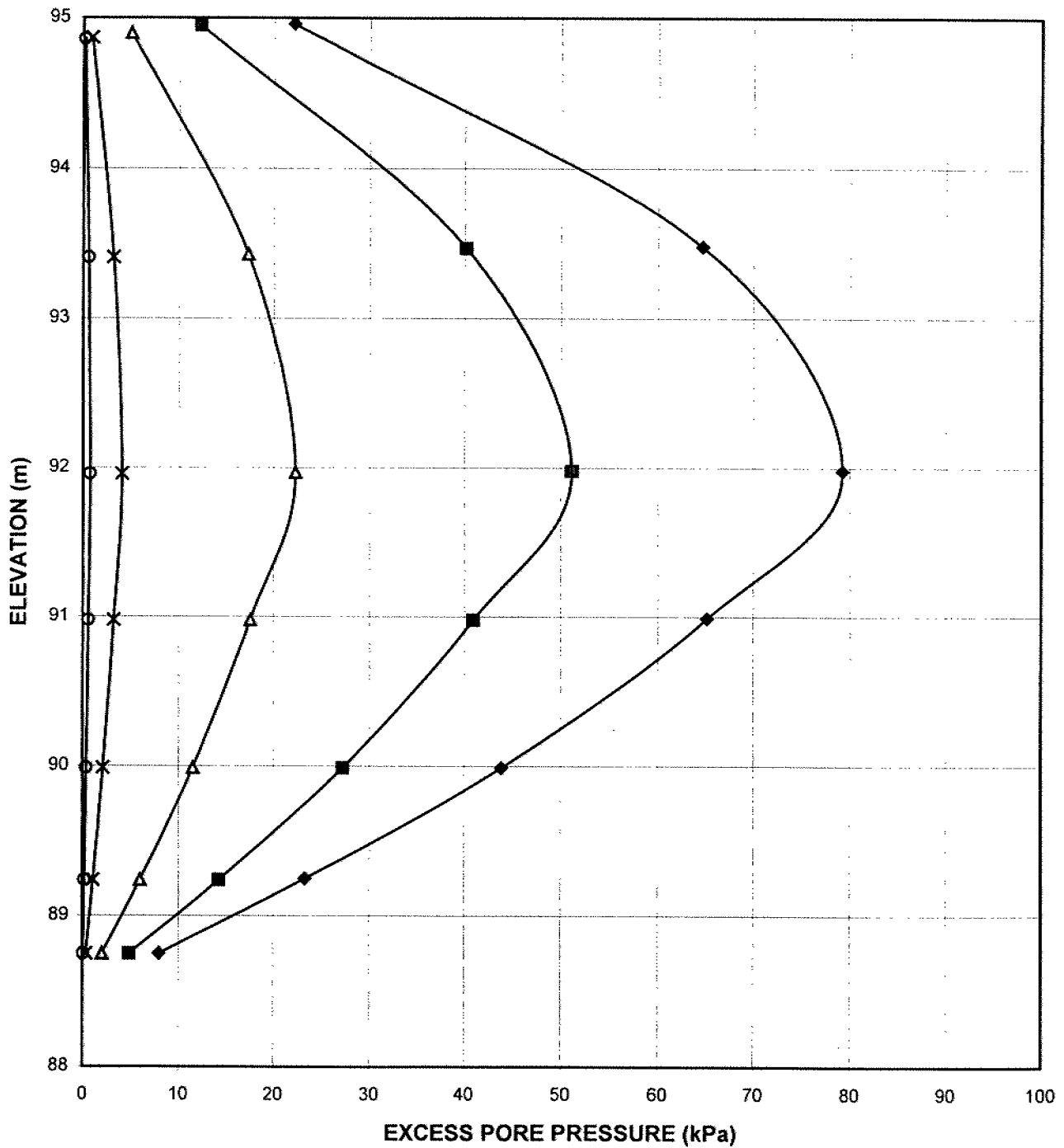


FIGURE A15

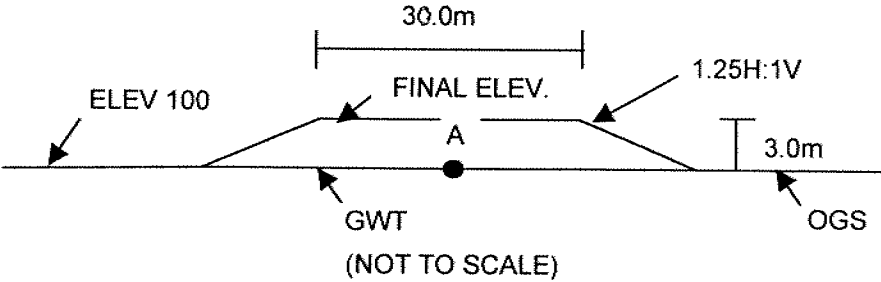
HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - APPROX. STATION 13+270, EW-N RAMP (CPTUN3)  
EXCESS PORE PRESSURES - NO WICK DRAINS - MOST LIKELY  $P_c$   
(AT THE CENTRELINE OF THE EMBANKMENT)



EPP - CHART

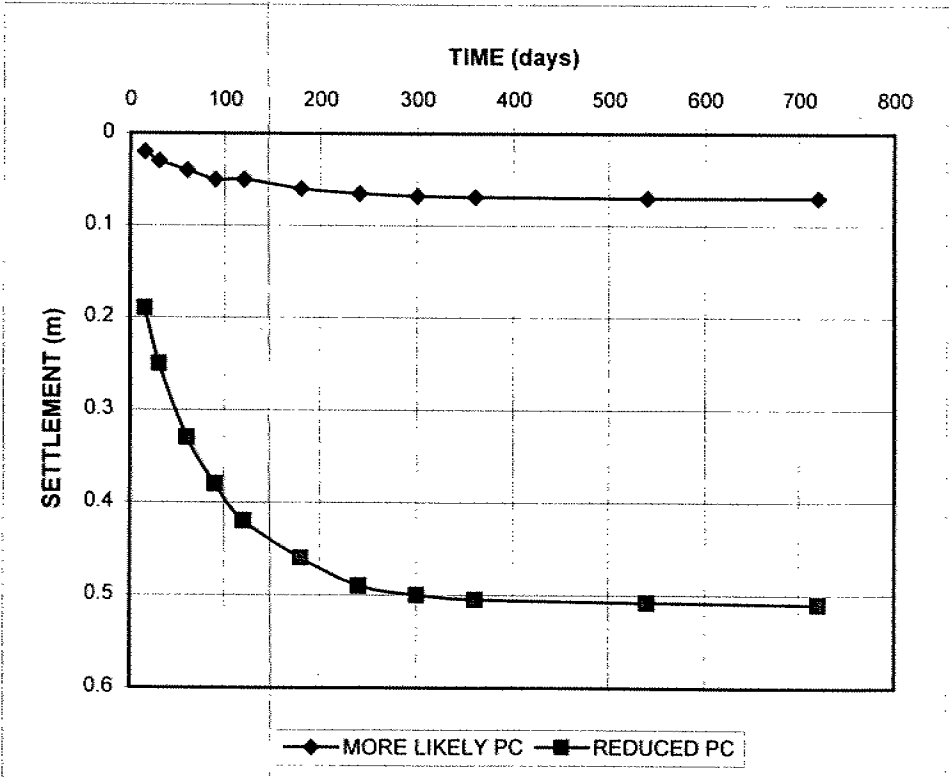
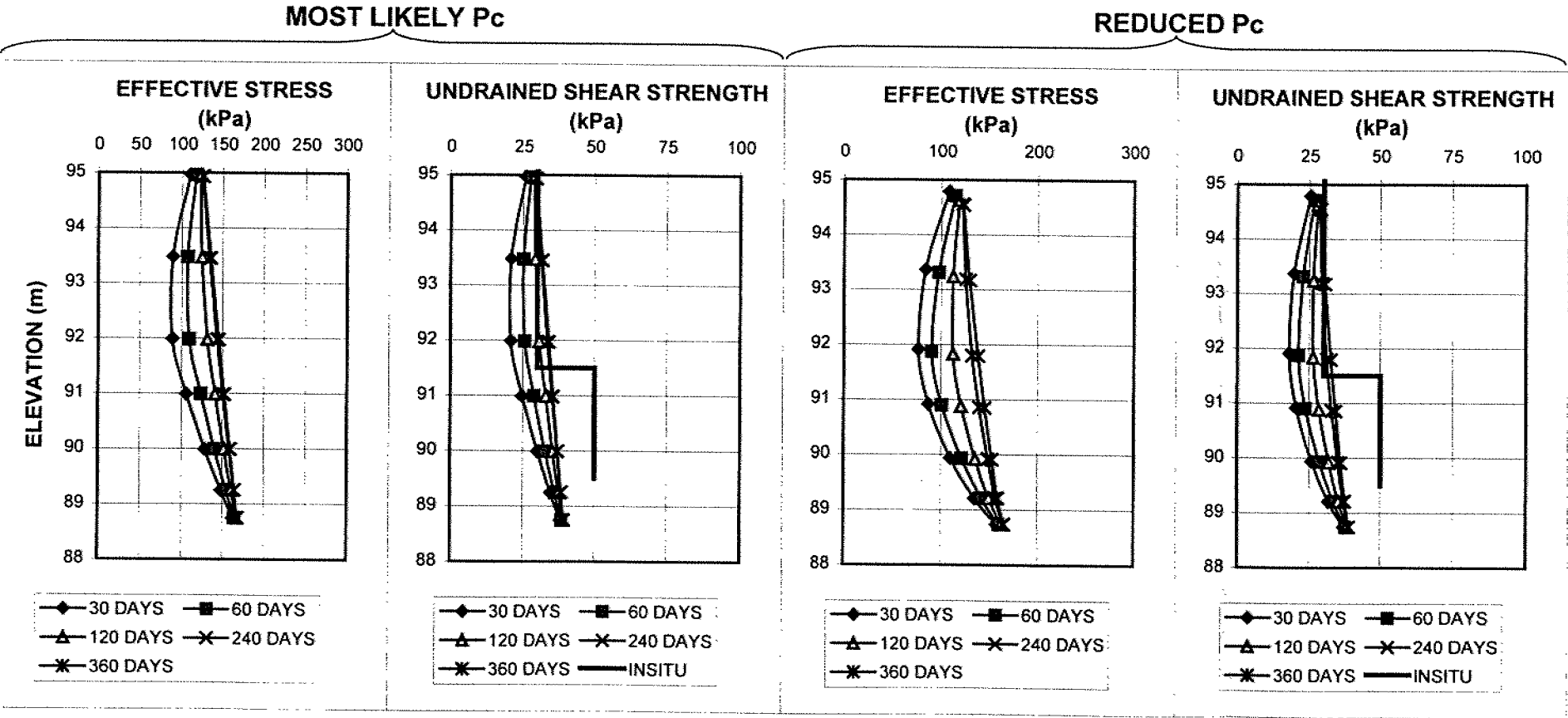
FIGURE A15-B

HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - APPROX. STATION 13+270, EW-N RAMP (CPTUN3)  
SETTLEMENTS DUE TO PRIMARY CONSOLIDATION - NO WICK DRAINS  
(AT THE CENTRELINE OF THE EMBANKMENT)



CONSTRUCTION STAGES FOR MOST LIKELY $P_c$		
STAGE	EMBANKMENT HEIGHT (m)	TIME (DAYS)
1	4	0

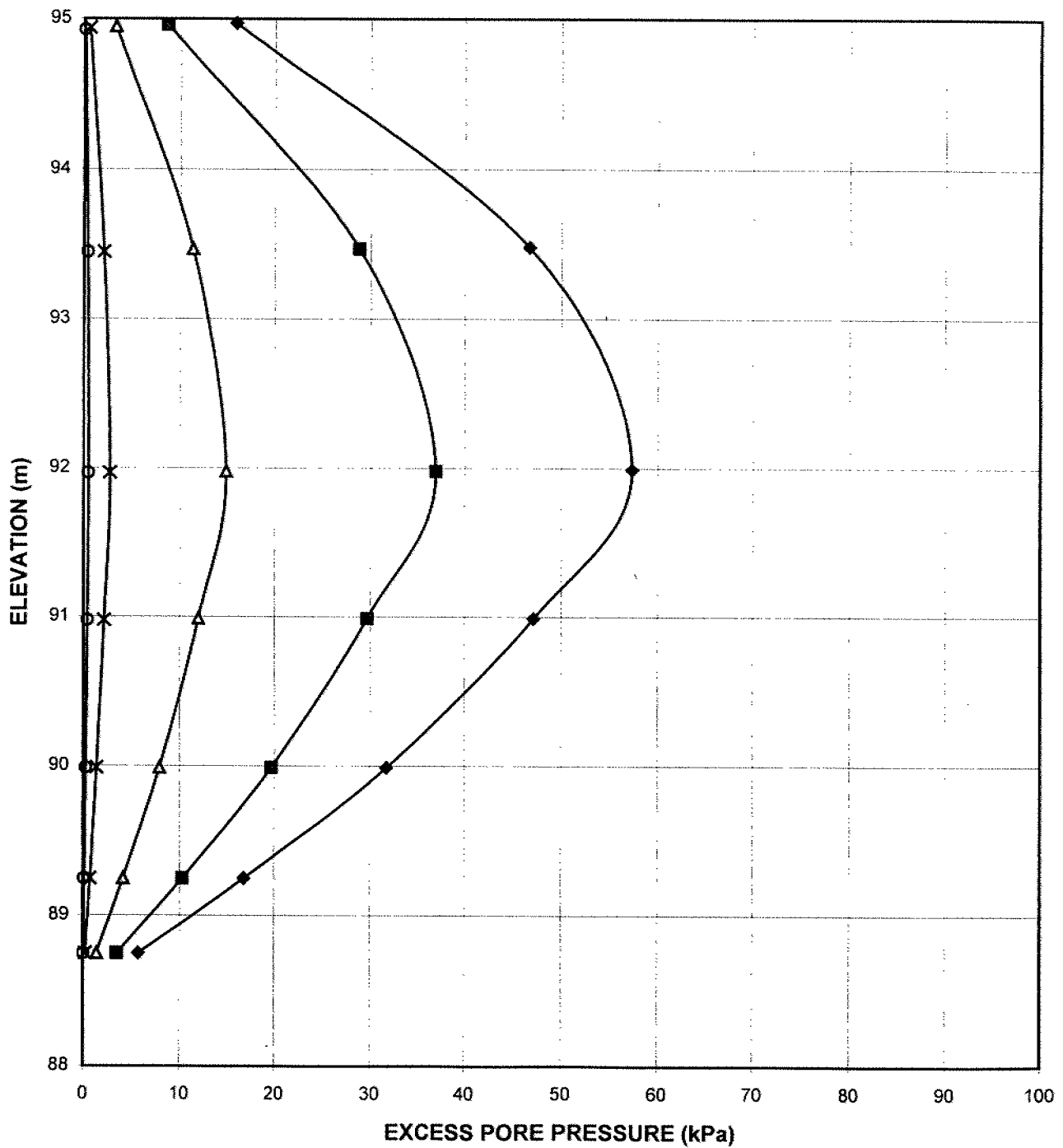
CONSTRUCTION STAGES FOR REDUCED $P_c$		
STAGE	EMBANKMENT HEIGHT (m)	TIME (DAYS)
1	4	0



MASTER PLOT

FIGURE A16

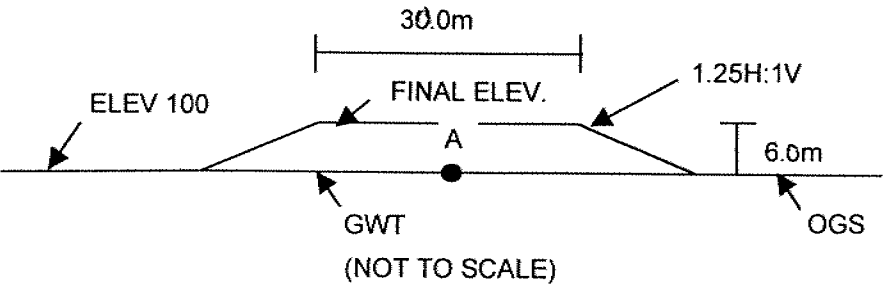
HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - APPROX. STATION 13+270, EW-N RAMP (CPTUN3)  
EXCESS PORE PRESSURE - NO WICK DRAINS - MOST LIKELY  $P_c$   
(AT THE EMBANKMENT CENTRELINE)



EPP - CHART

FIGURE A16-B

**HIGHWAY 11 - TROUT CREEK BY-PASS**  
**NORTH INTERCHANGE - APPROX. STATION 10+ 305 MCCARTHY STREET (CPTUN4)**  
**SETTLEMENTS DUE TO PRIMARY CONSOLIDATION - NO WICK DRAINS**  
**(AT THE CENTRELINE OF THE EMBANKMENT)**

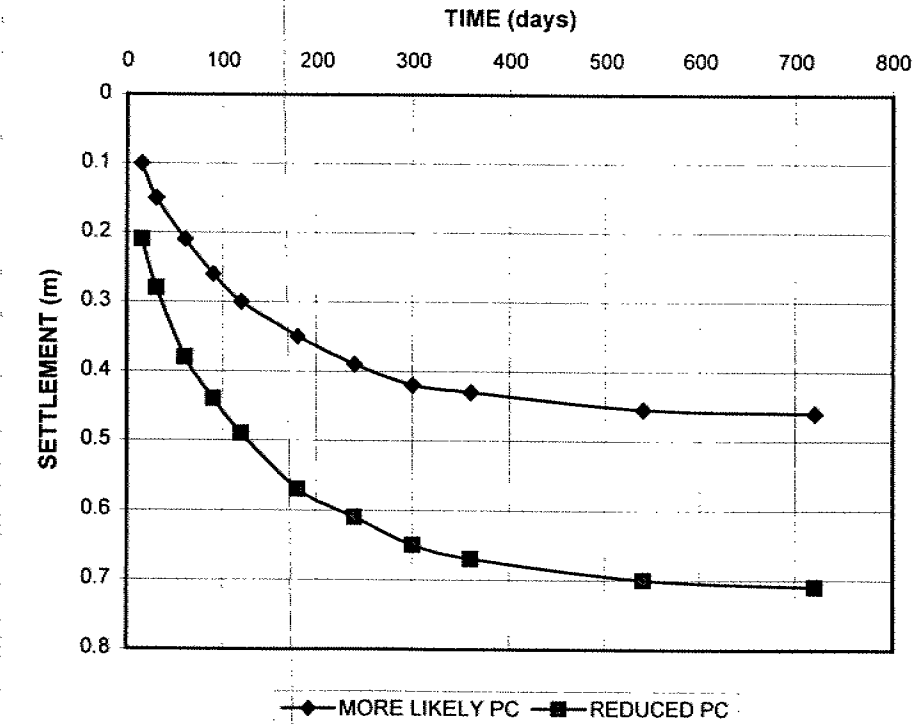
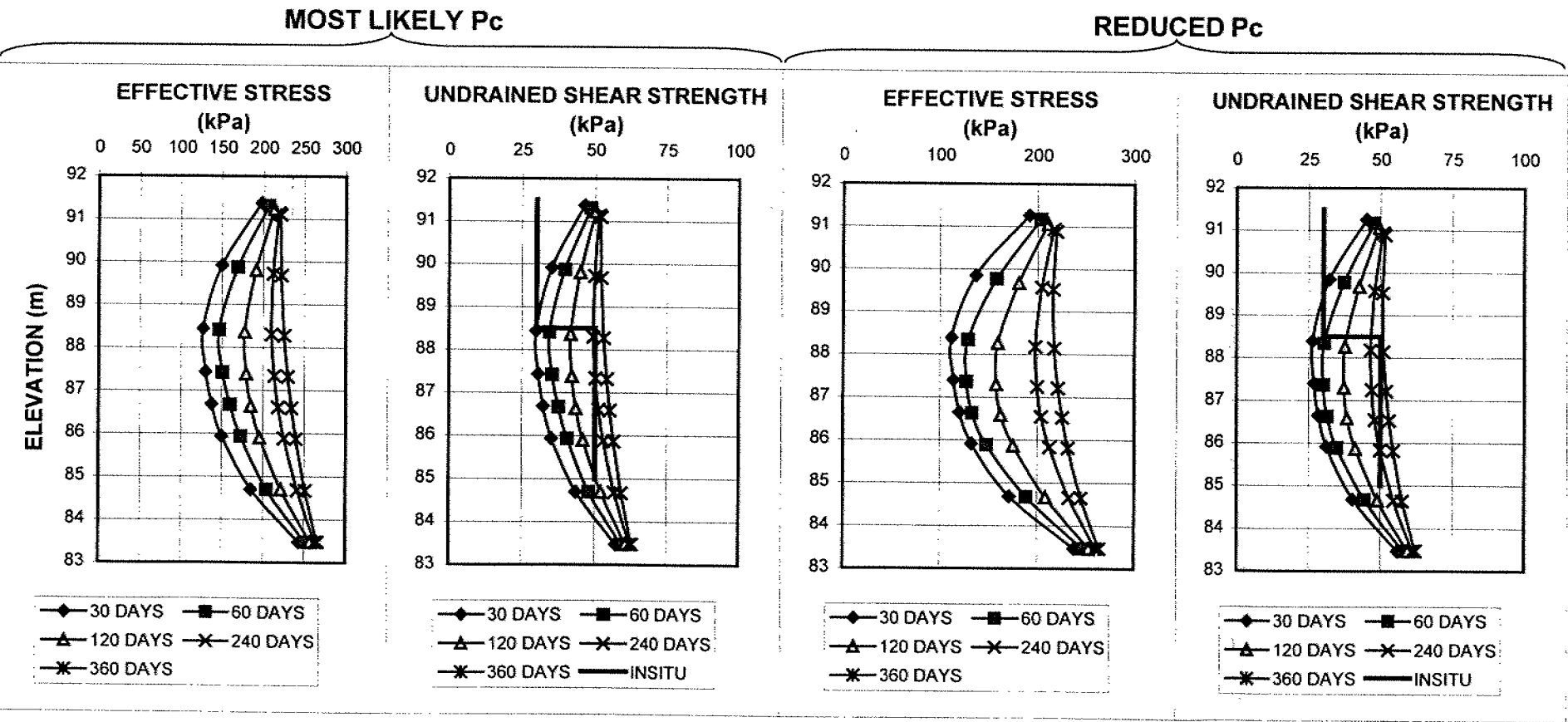


CONSTRUCTION STAGES FOR MOST LIKELY  $P_c$

STAGE	EMBANKMENT HEIGHT (m)	TIME (DAYS)
1	7.5	0

CONSTRUCTION STAGES FOR REDUCED  $P_c$

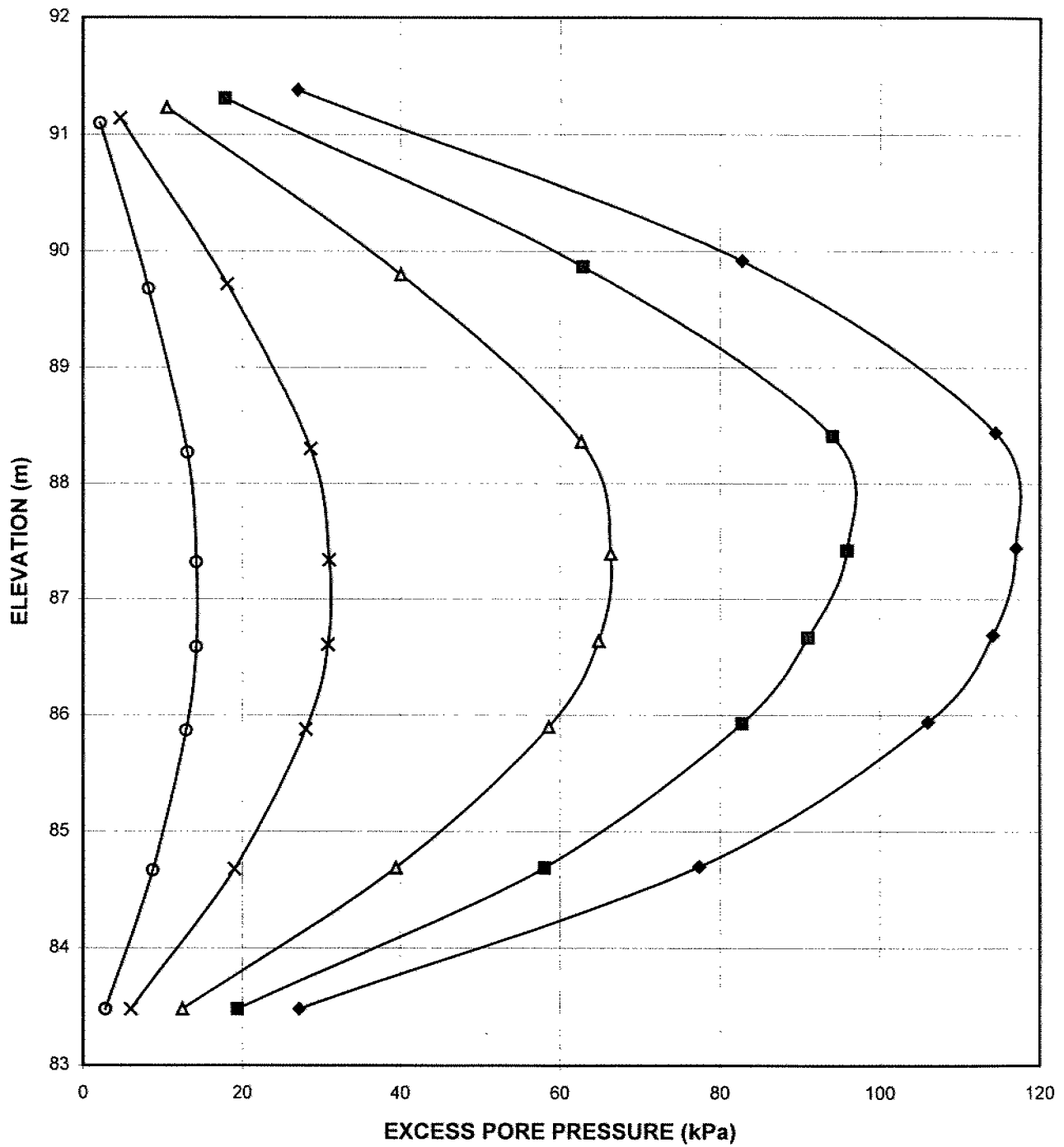
STAGE	EMBANKMENT HEIGHT (m)	TIME (DAYS)
1	7.5	0



MASTER PLOT

FIGURE A17

HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - APPROX. STATION 10+305 MCCARTHY ST (CPTUN4)  
EXCESS PORE PRESSURES - NO WICK DRAINS - MOST LIKELY  $P_c$   
(AT THE CENTRELINE OF THE EMBANKMENT)

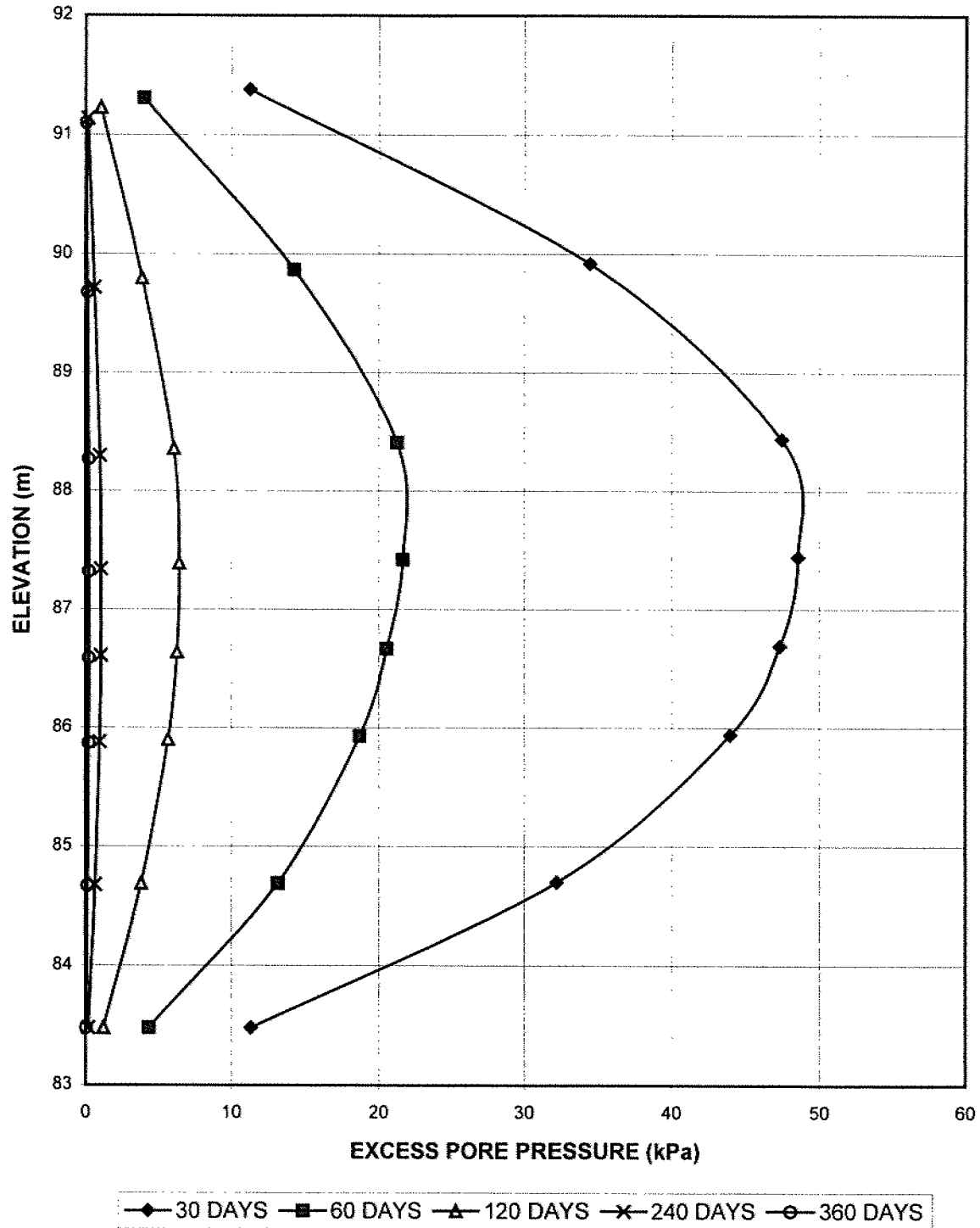


EPP - CHART

FIGURE A17-B

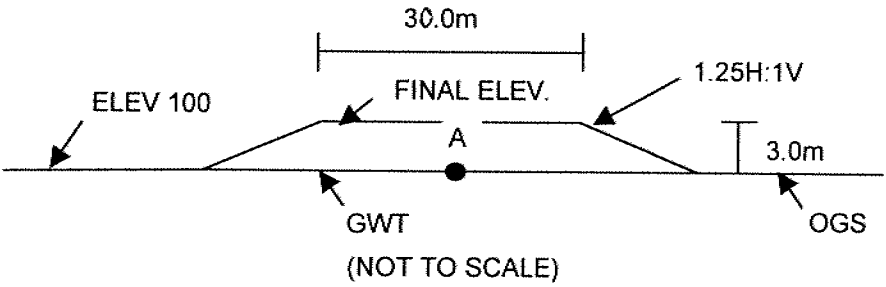


HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - APPROX. STATION 10+305 MCCARTHY ST (CPTUN4)  
EXCESS PORE PRESSURES - WICK DRAINS  $s=2.9\text{m}$ - MOST LIKELY  $P_c$   
(AT THE CENTRELINE OF THE EMBANKMENT)



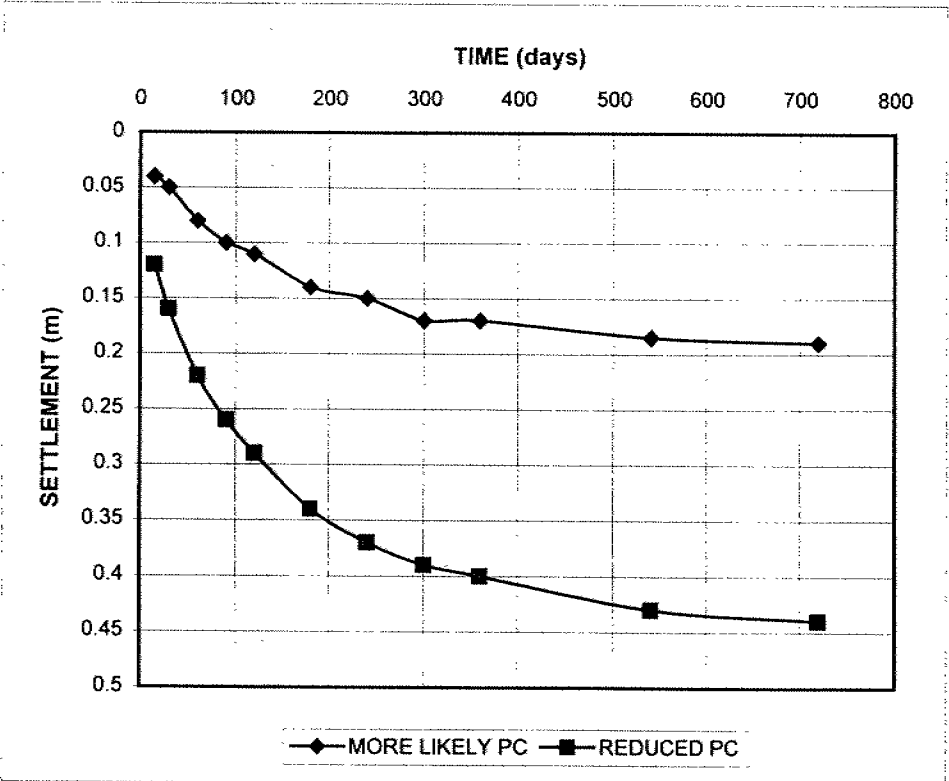
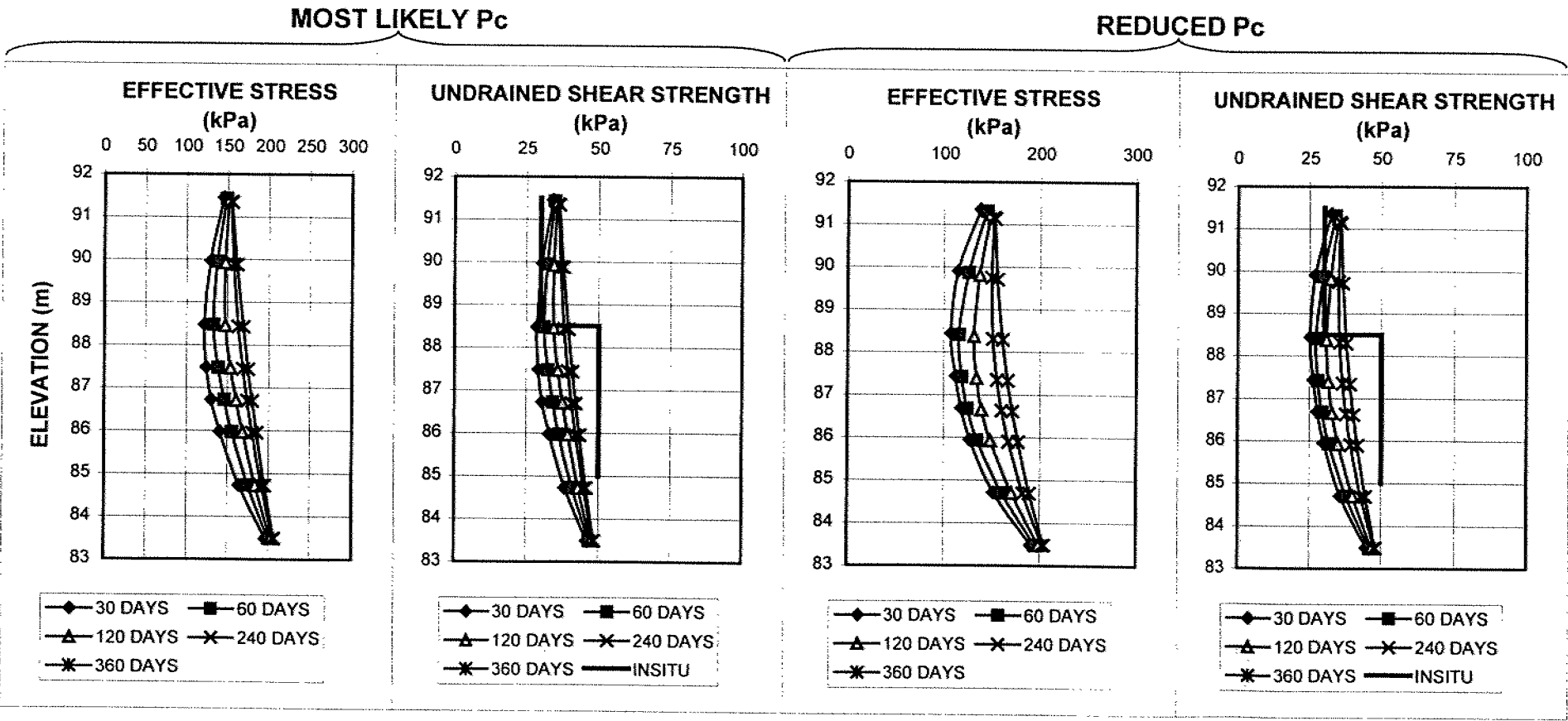
EPP - CHART (2)

HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - APPROX. STATION 10+305 MCCARTHY STREET (CPTUN4)  
SETTLEMENTS DUE TO PRIMARY CONSOLIDATION - NO WICK DRAINS  
(AT THE CENTRELINE OF THE EMBANKMENT)



CONSTRUCTION STAGES FOR MOST LIKELY $P_c$		
STAGE	EMBANKMENT HEIGHT (m)	TIME (DAYS)
1	4	0

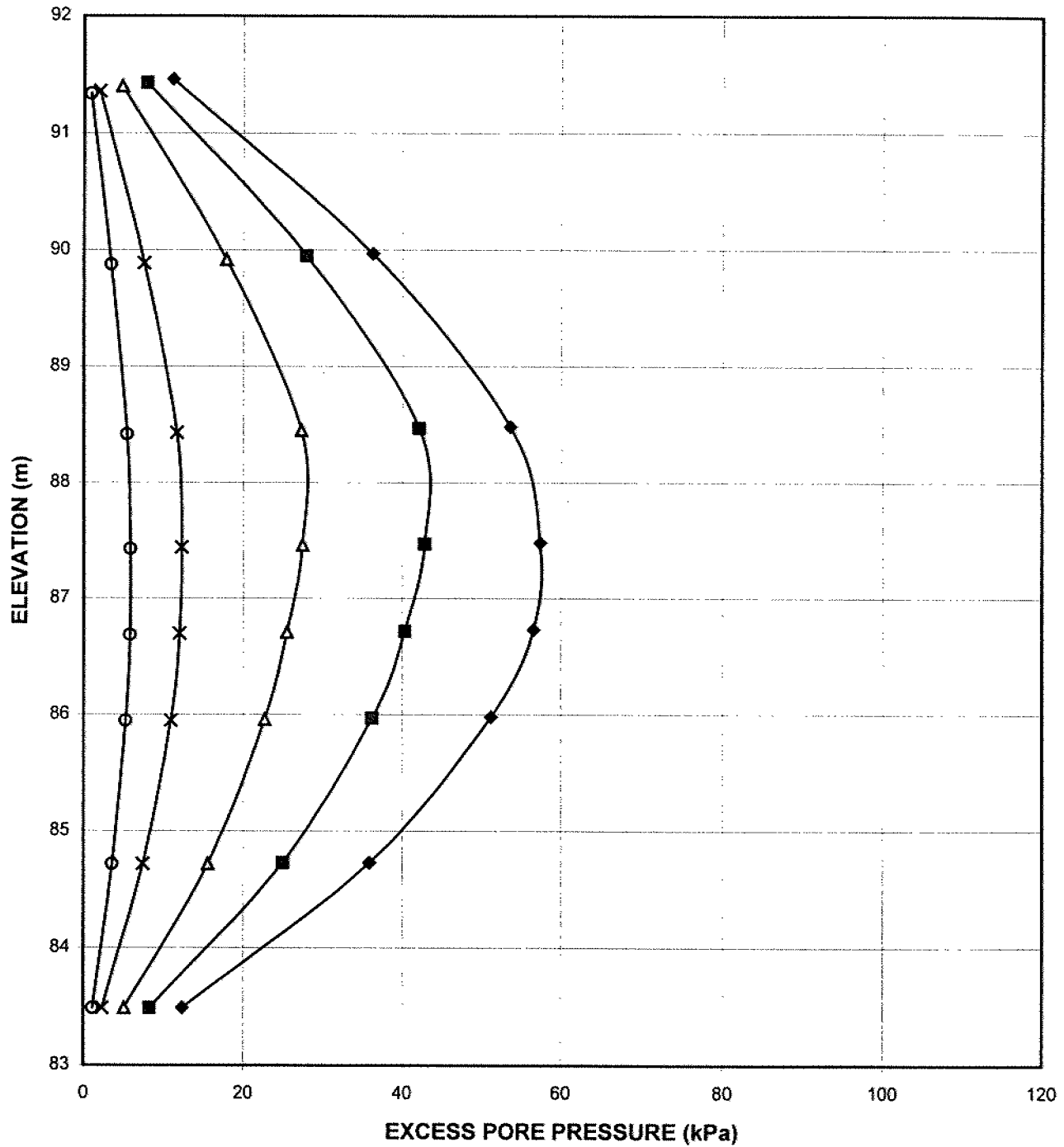
CONSTRUCTION STAGES FOR REDUCED $P_c$		
STAGE	EMBANKMENT HEIGHT (m)	TIME (DAYS)
1	4	0



MASTER PLOT

FIGURE A18

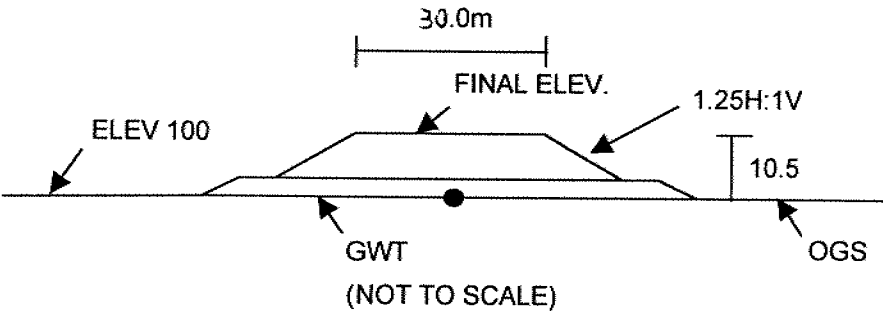
HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - APPROX. STATION 10+305 MCCARTHY ST (CPTUN4)  
EXCESS PORE PRESSURES - NO WICK DRAINS - MOST LIKELY  $P_c$   
(AT THE CENTRELINE OF THE EMBANKMENT)



EPP - GRAPH

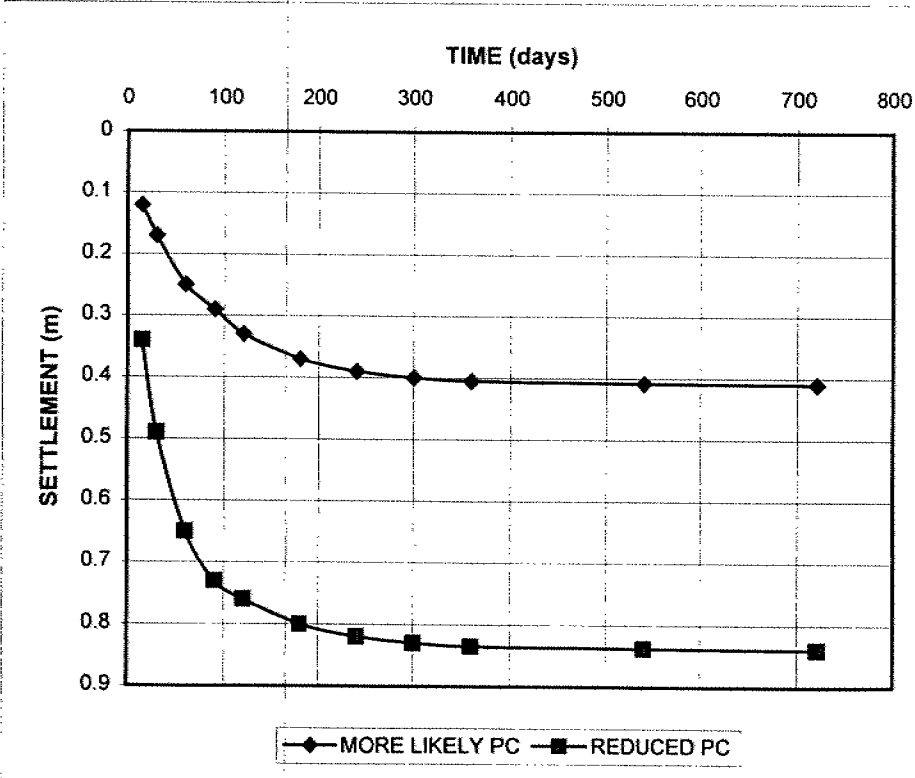
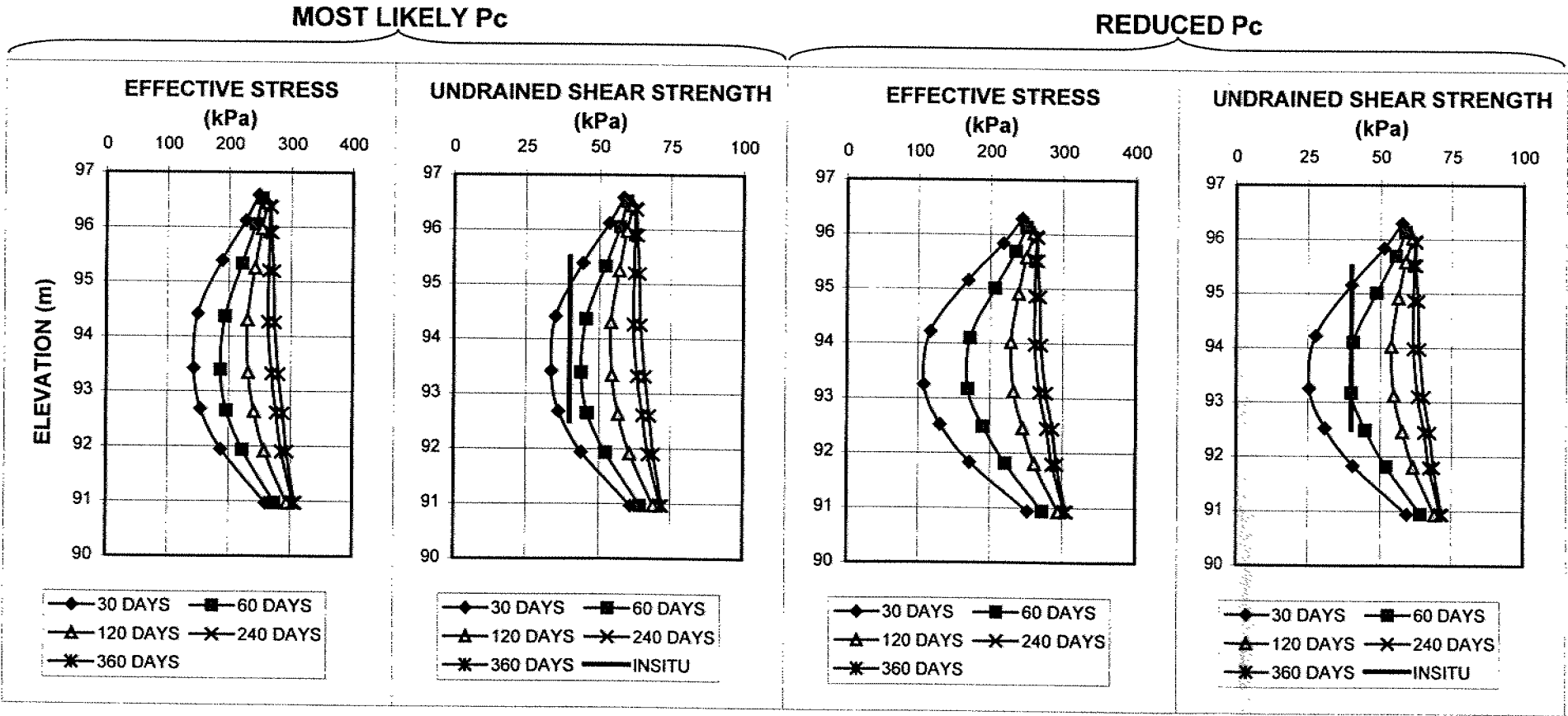
FIGURE A18-B

HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - APPROX. STATION 10+045 MCCARTHY STREET (CPTUN5)  
SETTLEMENTS DUE TO PRIMARY CONSOLIDATION - NO WICK DRAINS  
(AT THE CENTRELINE OF THE EMBANKMENT)



CONSTRUCTION STAGES FOR MOST LIKELY $P_c$		
STAGE	EMBANKMENT HEIGHT (m)	TIME (DAYS)
1	11.5	0
2	0.5	90

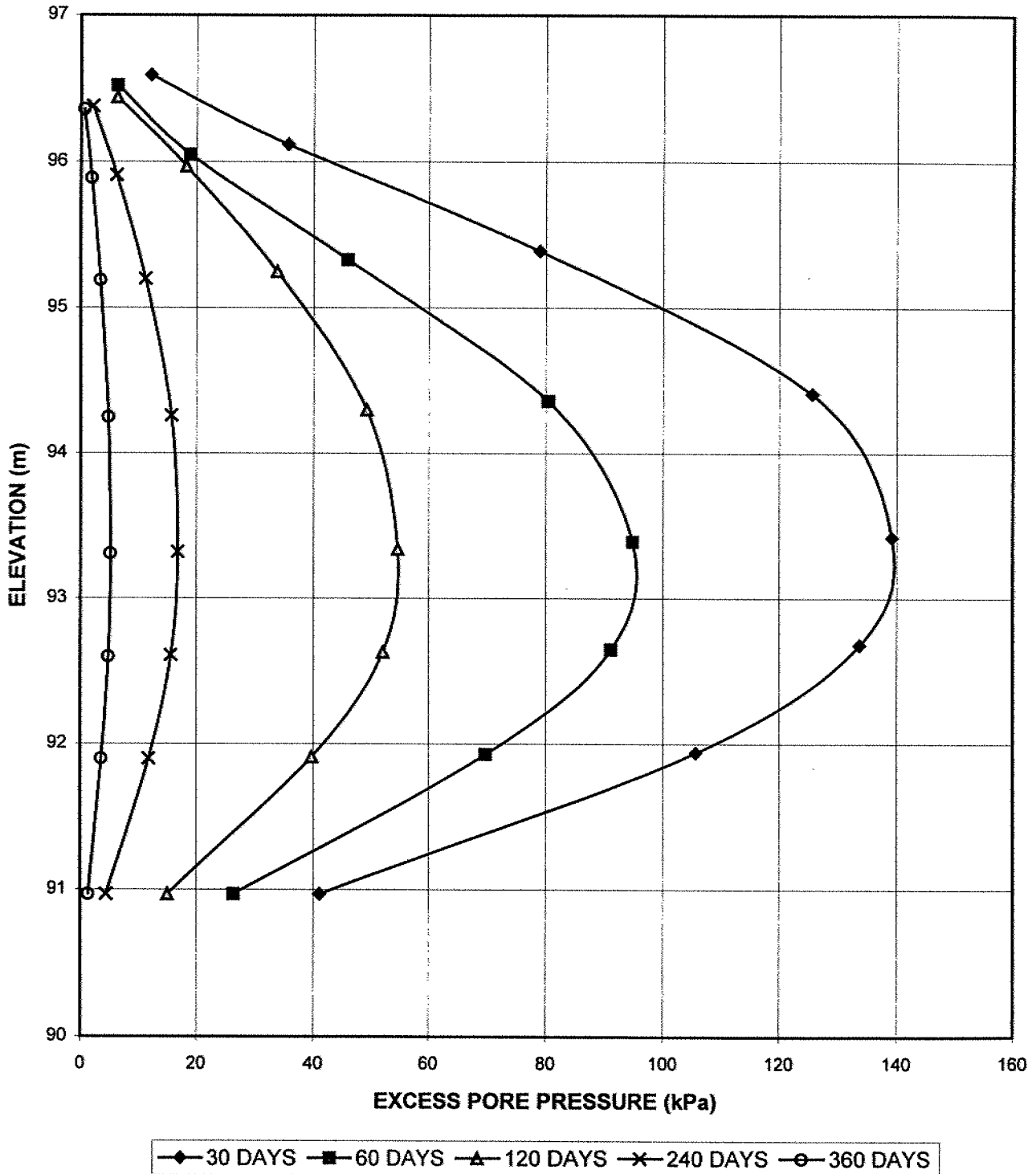
CONSTRUCTION STAGES FOR REDUCED $P_c$		
STAGE	EMBANKMENT HEIGHT (m)	TIME (DAYS)
1	11.5	0
2	0.5	90



MASTER PLOT

FIGURE A19  
23/04/99

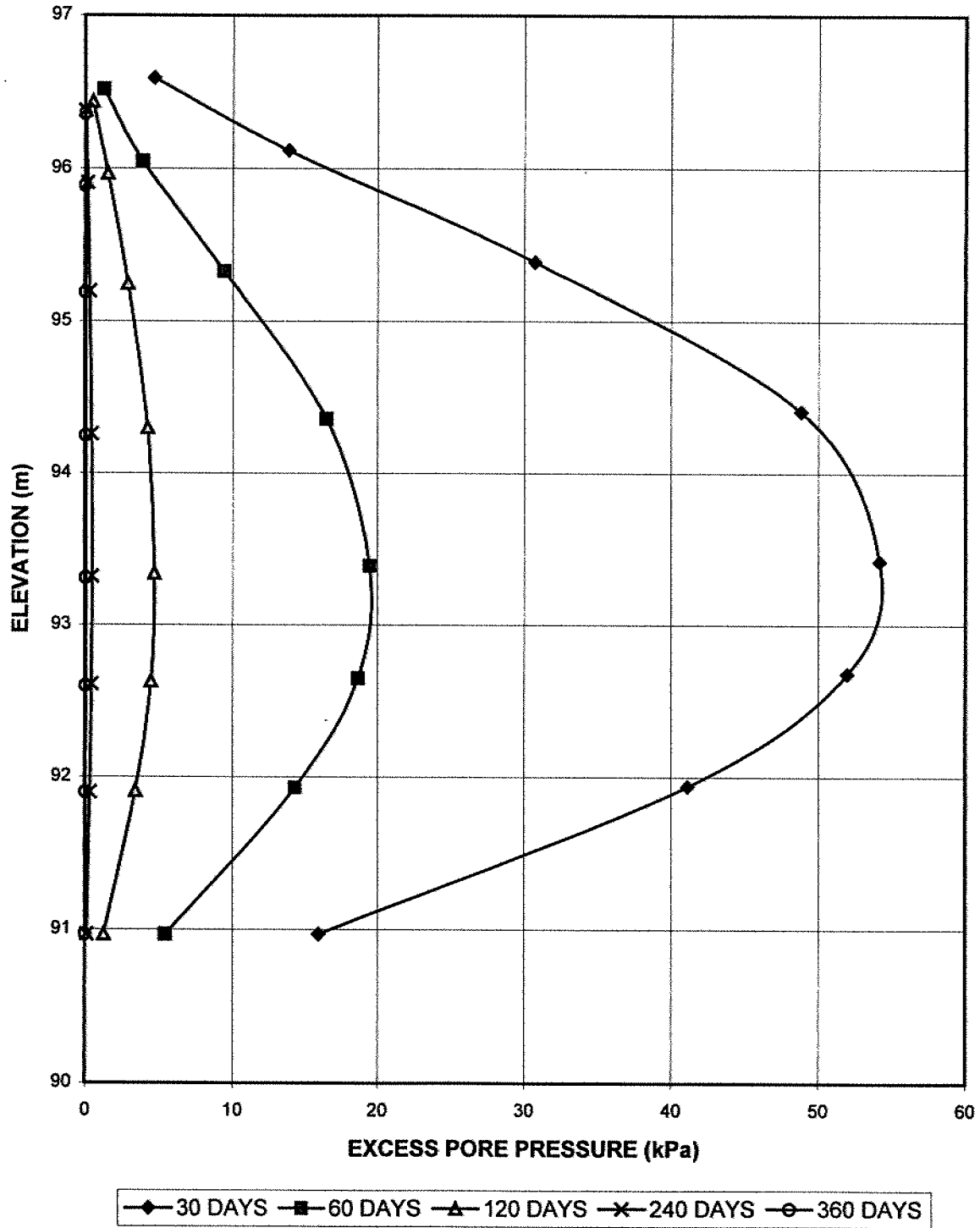
**HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - APPROX. STATION 10+045 MCCARTHY ST (CPTUN5)  
EXCESS PORE PRESSURES - NO WICK DRAINS - MOST LIKELY  $P_c$   
(AT THE CENTRELINE OF THE EMBANKMENT)**



EPP - CHART

FIGURE A19-B

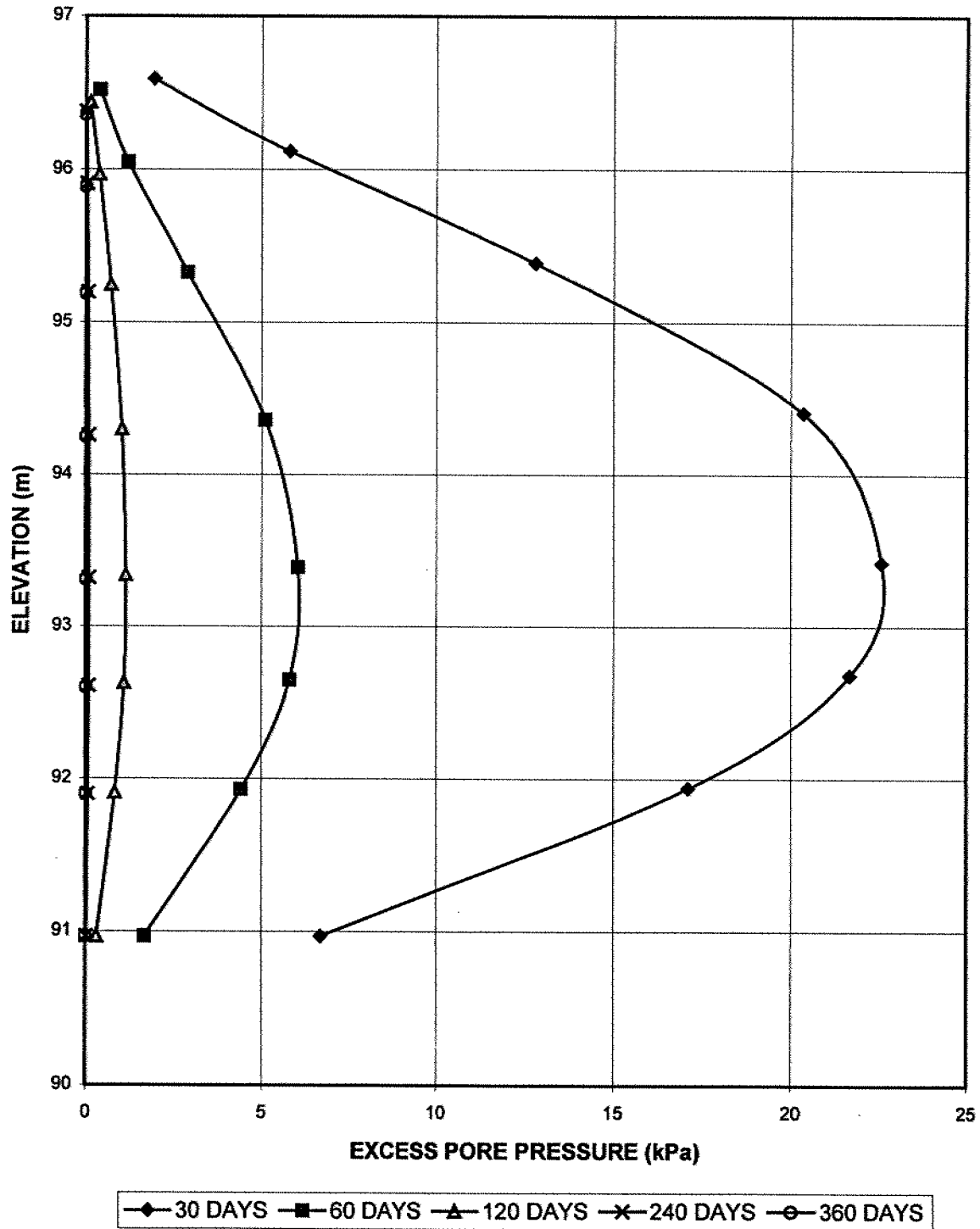
HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - APPROX. STATION 10+045 MCCARTHY ST (CPTUN5)  
EXCESS PORE PRESSURES - WICK DRAINS  $s=3.5\text{m}$  - MOST LIKELY  $P_c$   
(AT THE CENTRELINE OF THE EMBANKMENT)



EPP - CHART (2)

FIGURE A19-C

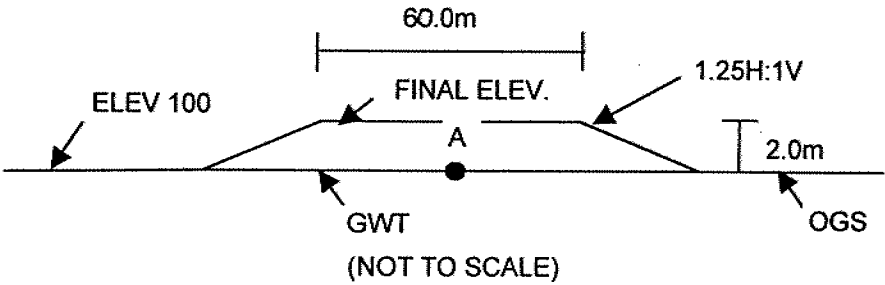
HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - APPROX. STATION 10+045 MCCARTHY ST (CPTUN5)  
EXCESS PORE PRESSURES - WICK DRAINS  $s=2.5\text{m}$  - MOST LIKELY  $P_c$   
(AT THE CENTRELINE OF THE EMBANKMENT)



EPP - CHART (3)

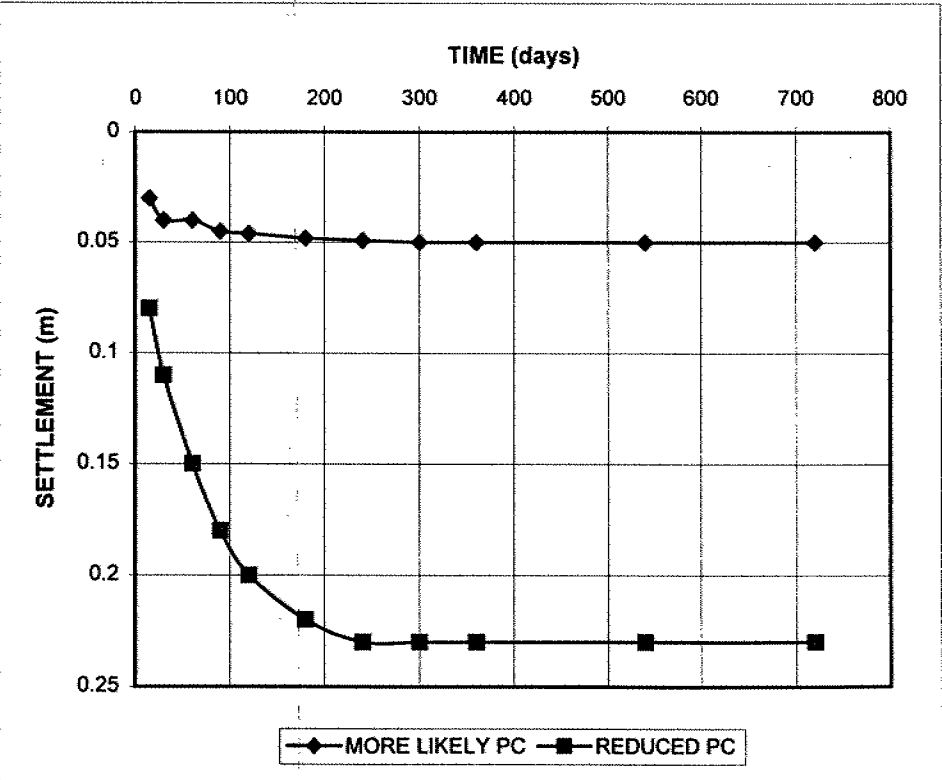
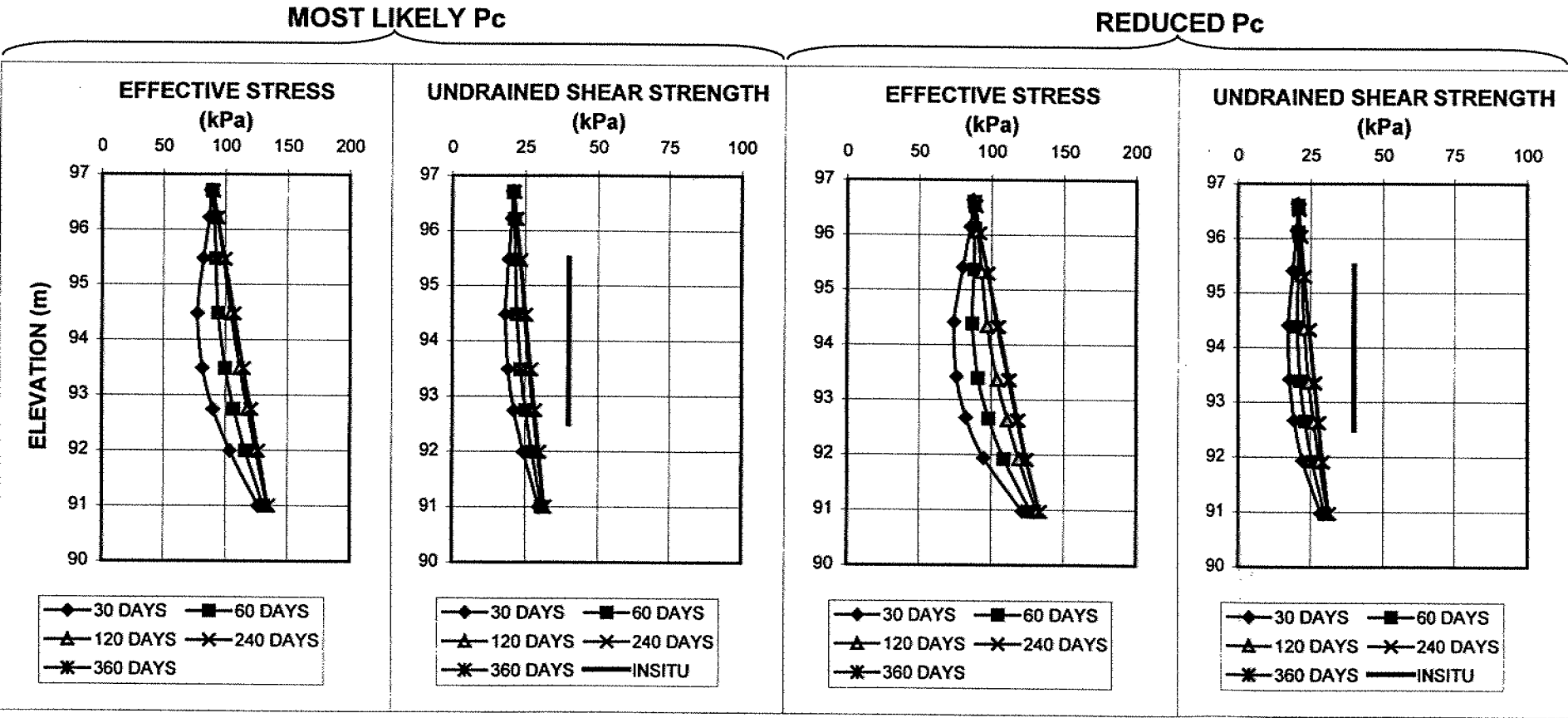
FIGURE A19-D

HIGHWAY 11 - TROUT CREEK BY-PASS  
 NORTH INTERCHANGE - APPROX. STATION 10+045 MCCARTHY STREET (CPTUN5)  
 SETTLEMENTS DUE TO PRIMARY CONSOLIDATION - NO WICK DRAINS  
 (AT THE CENTRELINE OF THE EMBANKMENT)



CONSTRUCTION STAGES FOR MOST LIKELY $P_c$		
STAGE	EMBANKMENT HEIGHT (m)	TIME (DAYS)
1	3	0

CONSTRUCTION STAGES FOR REDUCED $P_c$		
STAGE	EMBANKMENT HEIGHT (m)	TIME (DAYS)
1	3	0

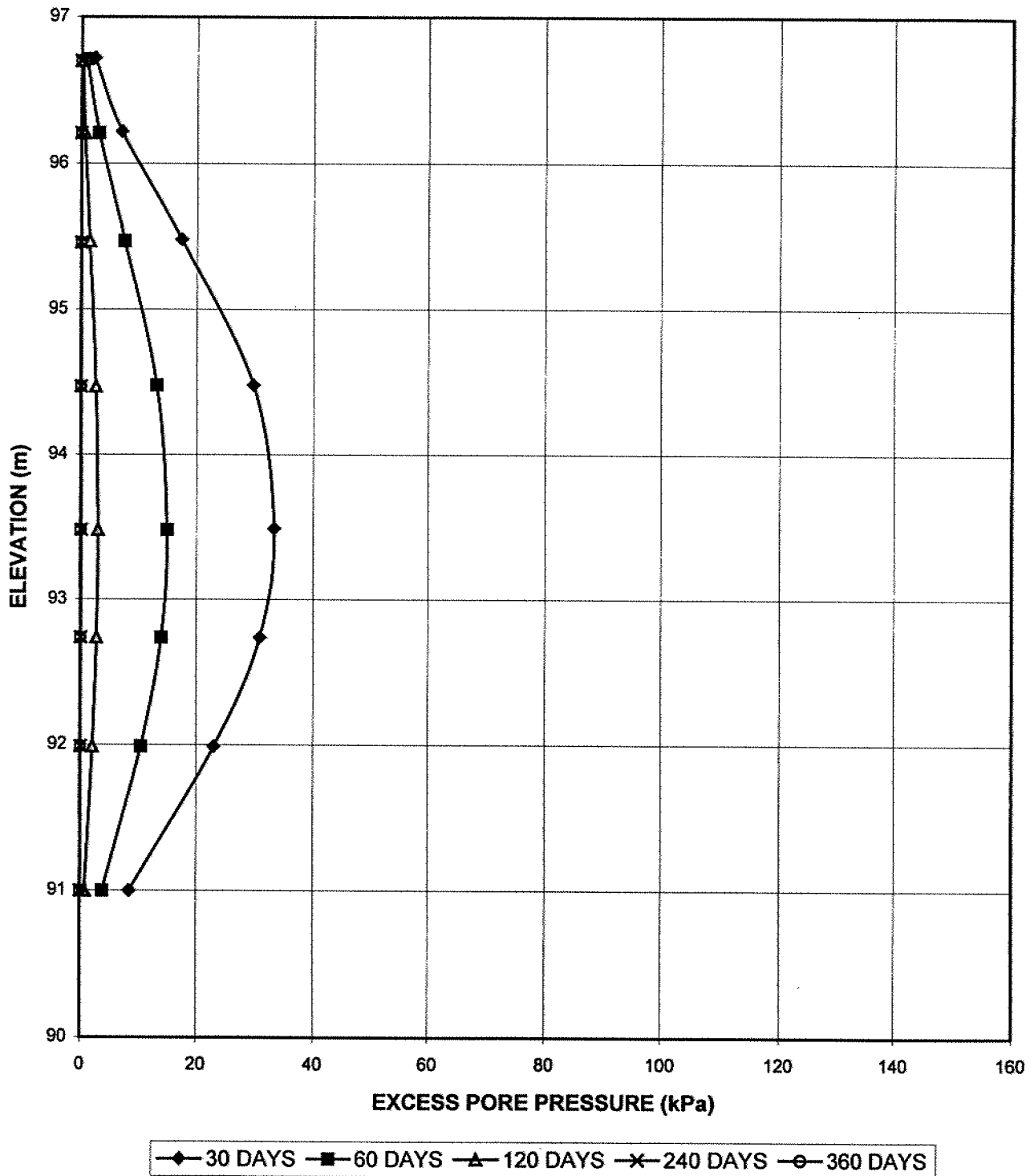


MASTER PLOT

FIGURE A20



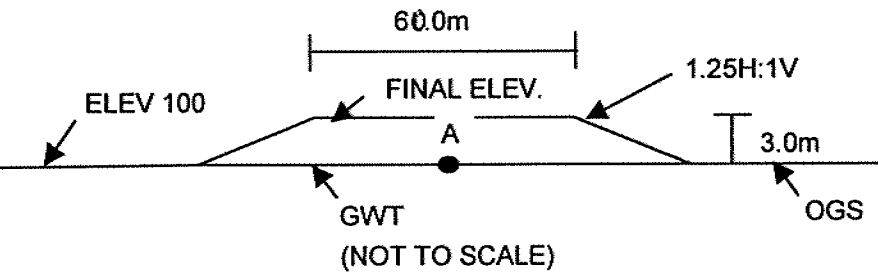
HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - APPROX. STATION 10+045 MCCARTHY ST (CPTUN5)  
EXCESS PORE PRESSURES - NO WICK DRAINS - MOST LIKELY  $P_c$   
(AT THE CENTRELINE OF THE EMBANKMENT)



EPP - CHART

FIGURE A20-B

HIGHWAY 11 - TROUT CREEK BY-PASS  
 NORTH INTERCHANGE - APPROX. STATION 10+335, EW-N RAMP (BH11BP)  
 SETTLEMENTS DUE TO PRIMARY CONSOLIDATION - NO WICK DRAINS  
 (AT THE CENTRELINE OF THE EMBANKMENT)



CONSTRUCTION STAGES FOR MOST LIKELY $P_c$		
STAGE	EMBANKMENT HEIGHT (m)	TIME (DAYS)
1	4	0

CONSTRUCTION STAGES FOR REDUCED $P_c$		
STAGE	EMBANKMENT HEIGHT (m)	TIME (DAYS)
1	4	0

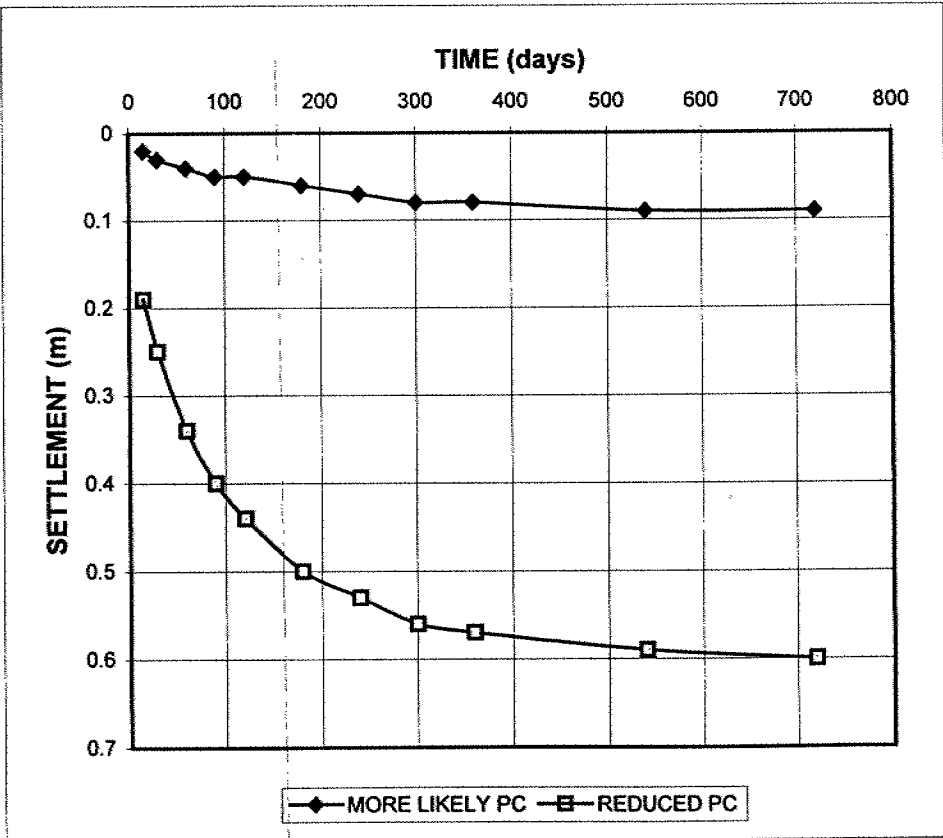
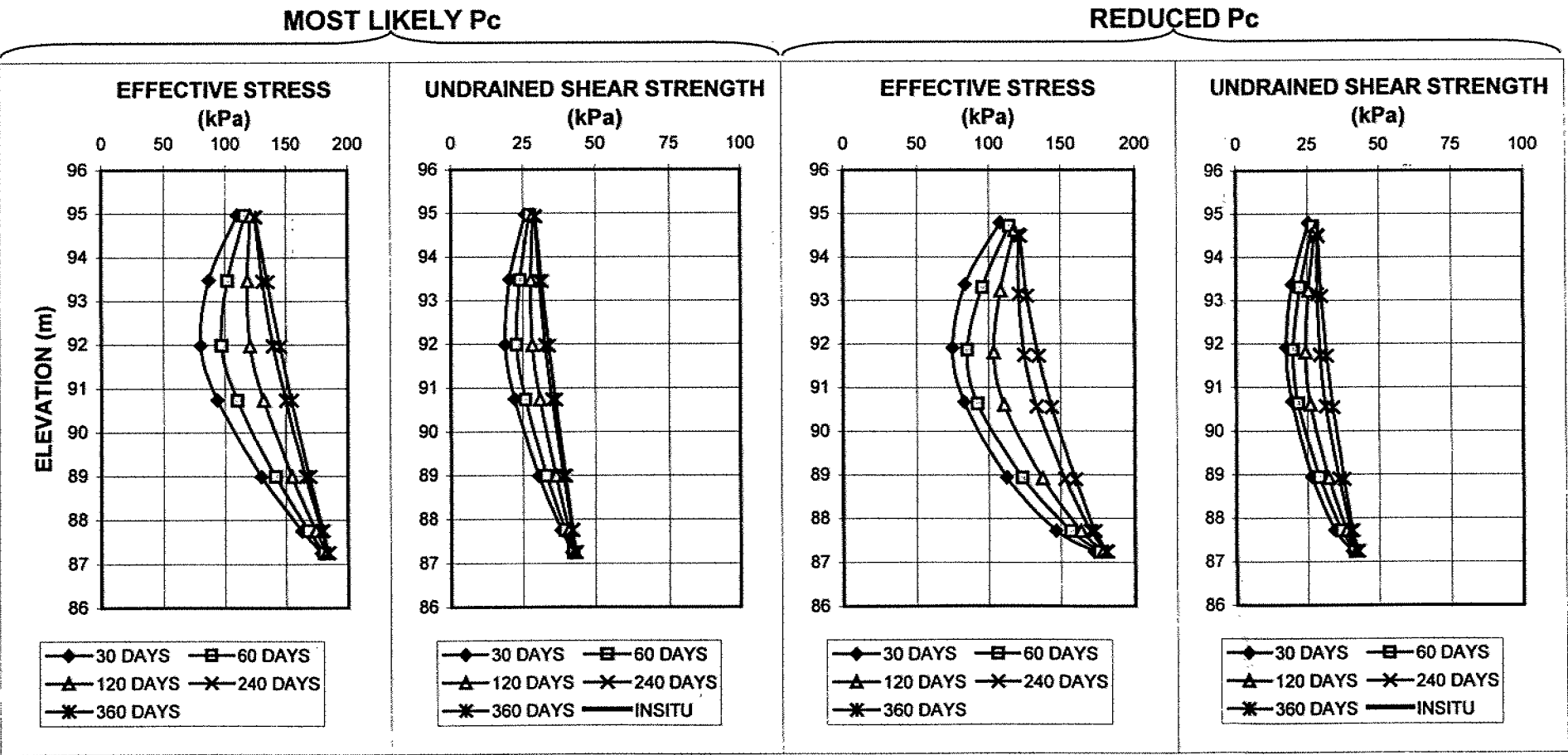
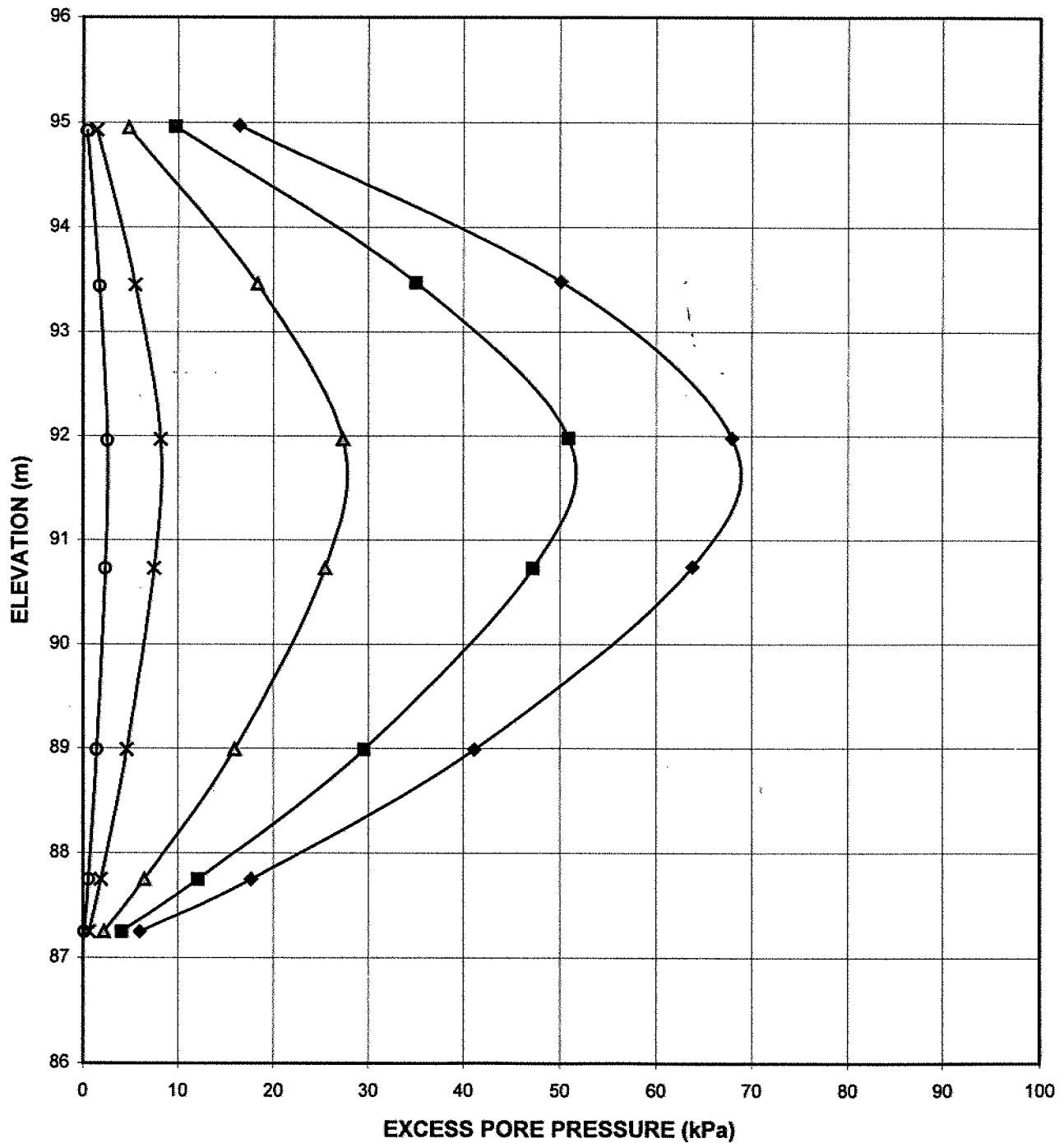


FIGURE A21

HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - APPROX. STATION 10+335, EW-N RAMP (BH11FP)  
EXCESS PORE PRESSURES - NO WICK DRAINS - MOST LIKELY  $P_c$   
(AT THE CENTRELINE OF THE EMBANKMENT)



EPP - CHART

FIGURE A21-B



## APPENDIX B

### TABLES

**Table B1 - Piezocone Test Locations and Depths**

Piezocone No.	Coordinates		Ground Surface Elevation (m)	Maximum Testing Depth (m)
	N	E		
CPTUN1	5095283.0	315381.7	314.7	12.1
CPTUN2	5095204.2	315333.8	314.7	17.3
CPTUN3	5095204.5	315214.2	315.0	15.1
CPTUN4	5095222.5	315507.7	314.6	20.6
CPTUN5	5095315.0	315272.0	315.2	12.6

HIGHWAY 11 - TROUT CREEK BY PASS - NORTH INTERCHANGE  
SOIL PROPERTIES FOR STABILITY AND SETTLEMENT ANALYSIS

Location	Soil Layer	Depth Interval		Unit Weight (kN/m3)	Undrained Shear Strength (kPa)	Friction Angle (deg)	Poisson's Ratio	Young's Modulus (MPa)	Compression Ratio		Pre-Consolidation Pressure		Coeff. Of Consolidation (m2/y)				Secondary Compression Ratio
		From (m)	To (m)						Cc(1-ec)	Cr(1-ec)	Most Likely (kPa)	Reduced (kPa)	O.C.		N.C.		
East Abutment and Hwy11 at and north of the bridge Sta.10+040 to10+120 CPTUN5	Rock Fill	top of fill	-0.2	20	---	42	0.3	150	N/A	---	---	---	---	---	---	---	---
	Peat	0	0.8	16	10	---	N/A	---	---	---	---	---	---	---	---	---	---
	Granular B	-0.2	0.8	20	---	32	0.3	20	---	---	---	---	---	---	---	---	---
	Sand	0.8	2.9	19	---	30	0.3	25	N/A	---	---	---	---	---	---	---	---
	Silty Clay	2.9	4	18	40	---	0.4	12	0.23	0.023	170	85	46	15	231	74	0.002
		4	6	17.5	40	---	0.45	12	0.23	0.023	170	85	28	15	142	74	0.002
		6	7.5	18	40	---	0.4	12	0.23	0.023	170	85	46	15	231	74	0.002
	Lower Silt	7.5	9.5	18	---	30	0.35	15	0.23	0.023	170	N.C.	46	15	231	74	0.002
		9.5	12	18	---	30	0.35	15	0.23	0.023	170	N.C.	46	15	231	74	0.002
	Sand & Gravel	12	14.3	21	---	35	0.3	30	---	---	170	N.C.	634	15	3169	74	0.002
Bedrock	14.3	>14.3	---	---	---	---	---	---	---	---	---	---	---	---	---	---	
East Approach: Sta.10+120-10+240 W-N Ramp(13+091-13+160) S-EW Ramp(13+630-13+540) CPTUN1	Rock Fill	top of fill	-0.3	20	---	42	0.3	150	---	---	---	---	---	---	---	---	---
	Peat	0	0.7	16	10	---	---	---	---	---	---	---	---	---	---	---	---
	Granular B	-0.3	0.7	20	---	32	0.3	20	---	---	---	---	---	---	---	---	---
	Silty Sand	0.7	2	19	---	29	0.3	25	---	---	---	---	---	---	---	---	---
	Sand	2	4	19	---	30	0.3	25	---	---	---	---	---	---	---	---	---
	Silty Clay	4	5	18	30	---	0.4	9	0.23	0.023	128	60	52	15	258	74	0.002
		5	6	17.5	30	---	0.45	9	0.23	0.023	128	60	34	15	168	74	0.002
		6	7	18	30	---	0.4	9	0.23	0.023	128	60	52	15	258	74	0.002
	Silty Clay	7	8.5	18	40	---	0.4	12	0.23	0.023	170	85	52	15	258	74	0.002
		8.5	10	18	40	---	0.4	12	0.23	0.023	170	85	284	284	1419	1419	---
Lower Silt	10	16	19	---	30	0.35	15	---	---	---	---	---	---	---	---	---	
Sand & Gravel	14	16	21	---	35	0.3	30	---	---	---	---	---	---	---	---	---	
Bedrock	16	>16	---	---	---	---	---	---	---	---	---	---	---	---	---	---	
East Approach Sta.>10+240 CPTUN4	Rock Fill	top of fill	-0.2	20	---	42	0.3	150	---	---	---	---	---	---	---	---	---
	Granular B	-0.2	0.8	20	---	32	0.3	20	---	---	---	---	---	---	---	---	---
	Peat	0	0.8	16	10	---	---	---	---	---	---	---	---	---	---	---	---
	Sandy Silt/Sand	0.8	5.5	19	---	29	0.3	25	---	---	---	---	---	---	---	---	---
	Upper Silt	5.5	8	18	---	28	0.35	22	---	---	---	---	46	15	231	74	0.002
	Silty Clay	8	12	17.5	30	---	0.45	9	0.23	0.023	128	N.C.	20	15	100	74	0.002
	Silty Clay	12	13.5	18	40	---	0.4	12	0.23	0.023	170	N.C.	26	15	131	74	0.002
	Lower Silt	13.5	17	18	---	30	0.35	15	0.23	0.023	170	N.C.	46	15	231	74	0.002
	Sandy Silt	17	20	18.5	---	30	0.35	20	---	---	---	---	---	---	---	---	---
	Silty Sand/Sand	20	23	21	---	35	0.3	30	---	---	---	---	---	---	---	---	---
Bedrock	23	>23	---	---	---	---	---	---	---	---	---	---	---	---	---	---	
EW-N Ramp (Sta.13+160-13+270) S-EW Ramp (13+540-13+450) CPTUN2	Rock Fill	top of fill	-0.1	20	---	42	0.3	150	---	---	---	---	---	---	---	---	---
	Peat	0	0.9	16	10	---	---	---	---	---	---	---	---	---	---	---	---
	Granular B	-0.1	0.9	20	---	32	0.3	20	---	---	---	---	---	---	---	---	---
	Sand	0.9	2.5	19	---	29	0.3	25	---	---	---	---	---	---	---	---	---
	Upper Silty Sand	2.5	4.5	19	---	30	0.3	25	---	---	---	---	---	---	---	---	---
	Silty Clay	4.5	8.5	17.5	35	---	0.45	10.5	0.23	0.023	150	75	19	15	95	74	0.002
		8.5	10.5	18	35	---	0.4	10.5	0.23	0.023	150	N.C.	42	15	210	74	0.002
		10.5	12	18	50	---	0.4	15	0.23	0.023	210	N.C.	134	15	668	74	0.002
	Sandy Silt	12	19	19	---	30	0.3	20	---	---	---	---	---	---	---	---	---
	Sand & Gravel	19	20	21	---	35	0.3	30	---	---	---	---	---	---	---	---	---
Bedrock	20	>20	---	---	---	---	---	---	---	---	---	---	---	---	---	---	
EW-N Ramp (Sta.13+270-13+350) S-EW Ramp (Sta.13+450-13+380) CPTUN3	Rock Fill	top of fill	-0.2	20	---	42	0.3	150	---	---	---	---	---	---	---	---	---
	Peat	0	0.8	16	10	---	---	---	---	---	---	---	---	---	---	---	---
	Granular B	-0.2	0.8	20	---	32	0.3	20	---	---	---	---	---	---	---	---	---
	Silty Sand & Sand	0.8	4.5	18	---	32	0.3	25	---	---	---	---	---	---	---	---	---
	Silty Clay	4.5	8.5	17.5	30	---	0.45	9	0.23	0.023	128	N.C.	18	15	89	74	0.002
	Silty Clay	8.5	10.5	18	50	---	0.4	15	0.23	0.023	210	N.C.	42	15	210	74	0.002
	Lower Silt	10.5	11.5	19	---	30	0.35	20	0.23	0.023	210	N.C.	46	15	231	74	0.002
	Sandy Silt	11.5	15 (?)	19	---	32	0.3	20	---	---	---	---	715	715	3574	3574	---
	Sand & Gravel	15 (?)	18 (?)	21	---	35	0.3	30	---	---	---	---	---	---	---	---	---
	Bedrock	18 (?)	>18 (?)	---	---	---	---	---	---	---	---	---	---	---	---	---	---
EW-N Ramp (Sta.13+350-13+458) S-EW Ramp (Sta.13+380-13+125) Hwy 11 (south of bridge) BH-11BP	Rock Fill	top of fill	-0.2	20	---	42	0.3	150	---	---	---	---	---	---	---	---	---
	Peat	0	0.8	16	10	---	---	---	---	---	---	---	---	---	---	---	---
	Granular B	-0.2	0.8	20	---	32	0.3	20	---	---	---	---	---	---	---	---	---
	Silty Sand & Sand	0.8	4.5	18	---	32	0.3	25	---	---	---	---	---	---	---	---	---
	Silty Clay	4.5	8.5	17.5	30	---	0.45	9	0.23	0.023	128	N.C.	18	15	89	74	0.002
	Silty Clay	8.5	12	18	30	---	0.4	15	0.23	0.023	210	N.C.	42	15	210	74	0.002
	Lower Silt	12	13	19	---	30	0.35	20	0.23	0.023	210	N.C.	46	15	231	74	0.002
	Sandy Silt	13	14	19	---	32	0.3	20	---	---	---	---	715	715	3574	3574	---
	Sand & Gravel	14	16 (?)	21	---	35	0.3	30	---	---	---	---	---	---	---	---	---
Bedrock	16 (?)	>16 (?)	---	---	---	---	---	---	---	---	---	---	---	---	---	---	

Notes: O.C.: Over Consolidated Soil  
N.C.: Normally Consolidated Soil

**HIGHWAY 11 - TROUT CREEK BY PASS - NORTH INTERCHANGE  
SUMMARY OF STABILITY ANALYSIS**

Location	Design Height (m)	Target Height (m)	Berm Height (m)	Berm Width (m)	Height at this Stage (m)	Height at Previous Stage (m)	EPP dissipation before this stage	Factor of Safety	Reference Figure
East Abutment McCarthy St. CPTUN5 BH-3BF	10.5	12	4.5	11	12	0	0%	1.33	
				9	11.5	0	0%	1.30	A7
					12	11.5	100%	1.36	
					12	11.5	75%	1.29	A8
Head Slope @ 1.25H:1V	10.5	12	0	0	12	0	0%	<1.3	
East Approach to Bridge McCarthy St. E-W and S-EW Ramps CPTUN1	8	9.5	2	12	9.5	0	0%	1.28	
				11	9	0	0%	1.33	
					9.5	9	100%	1.41	
					9.5	9	75%	1.33	
					9.5	9	50%	1.28	
				9	8.8	0	0%	1.31	A9
					9.5	8.8	100%	1.36	
					9.5	8.8	75%	1.29	A10
					9.5	8.8	50%	1.24	
				8	8	0	0%	1.33	
					9.5	8	100%	1.33	
McCarthy St. transition to high emb./BH-3BP	7	8.5	1	11	8.5	0	0%	1.35	
				8	8.5	0	0%	1.32	
McC.H<6m/BH-3BP/4BP	6	7.5	0	0	7.5	0	0%	1.30	
EW-N and S-EW Ramps CPTUN2	7	8.5	1	11	8.5	0	0%	1.31	
				9	8.5	0	0%	1.28	

Note: EPP - Excess Pore Pressure



**HIGHWAY 11 - TROUT CREEK BY-PASS**  
**NORTH INTERCHANGE - SURCHARGE ANALYSIS**  
**East Abutment Sta. 10+040 to 10+120 - CPTUN5**  
**Most Likely Pre-Consolidation Pressures -30 m wide embankment top**

Initial Fill Height (m)	9 m	10 m	11 m	12 m	13 m	14 m
Immediate Settlement (mm)	84 mm	94 mm	103 mm	113 mm	122 mm	131 mm
Primary Consol. Settl. (mm)	279 mm	333 mm	384 mm	431 mm	475 mm	516 mm
Total Settlement (mm) (*)	363 mm	427 mm	487 mm	544 mm	597 mm	647 mm
Final Height above O.G.S. (m)	8.637 m	9.573 m	10.513 m	11.456 m	12.403 m	13.353 m

Time		% Consolidation	Settlement	Height Above	Settlement	Height Above	Settlement	Height Above	Settlement	Height Above	Settlement	Height Above	Settlement	Height Above
(days)	(months)	U%	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)
15	0.50	28.9	165	8.835	190	9.810	214	10.786	238	11.762	259	12.741	280	13.720
30	1.00	41	198	8.802	231	9.769	260	10.740	290	11.710	317	12.683	343	13.657
60	2.00	60.2	252	8.748	294	9.706	334	10.666	372	11.628	408	12.592	442	13.558
90	3.00	69.9	279	8.721	327	9.673	371	10.629	414	11.586	454	12.546	492	13.508
120	4.00	79.5	306	8.694	359	9.641	408	10.592	456	11.544	500	12.500	541	13.459
180	6.00	89.2	333	8.667	391	9.609	446	10.554	497	11.503	546	12.454	591	13.409
240	8.00	94	346	8.654	407	9.593	464	10.536	518	11.482	569	12.432	616	13.384
300	10.00	96.4	353	8.647	415	9.585	473	10.527	528	11.472	580	12.420	628	13.372
360	12.00	97.6	356	8.644	419	9.581	478	10.522	534	11.466	586	12.414	635	13.365
540	18.00	98.8	360	8.640	423	9.577	482	10.518	539	11.461	591	12.409	641	13.359
720	24.00	100	363	8.637	427	9.573	487	10.513	544	11.456	597	12.403	647	13.353

Note: (\*) Does not include settlements due to secondary consolidation

**Summary of Surcharge Requirements**

Initial Fill Height (m)	9	10	11	12	13	14
Final Height above O.G.S. (m)						
8.637	> 24.00	> 4.00	> 2.00	> 1.00	> 1.00	> 1.00
9.573		> 24.00	> 4.00	> 3.00	> 2.00	> 1.00
10.513			> 24.00	> 4.00	> 3.00	> 2.00
11.456				> 24.00	> 4.00	> 4.00
12.403					> 24.00	> 6.00
13.353						> 24.00

**HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - SURCHARGE ANALYSIS  
East Abutment Sta. 10+040 to 10+120 - CPTUN5  
Reduced Pre-Consolidation Pressures -30 m wide embankment top**

Initial Fill Height (m)	9 m	10 m	11 m	12 m	13 m	14 m
Immediate Settlement (mm)	84 mm	94 mm	103 mm	113 mm	122 mm	131 mm
Primary Consol. Settl. (mm)	690 mm	745 mm	795 mm	842 mm	886 mm	927 mm
Total Settlement (mm) (*)	774 mm	839 mm	898 mm	955 mm	1008 mm	1058 mm
Final Height above O.G.S. (m)	8.226 m	9.161 m	10.102 m	11.045 m	11.992 m	12.942 m

Time (days)	Time (months)	% Consolidation U%	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)
15	0.50	28.9	283	8.717	309	9.691	333	10.667	356	11.644	378	12.622	399	13.601
30	1.00	41	367	8.633	399	9.601	429	10.571	458	11.542	485	12.515	511	13.489
60	2.00	60.2	499	8.501	542	9.458	582	10.418	620	11.380	655	12.345	689	13.311
90	3.00	69.9	566	8.434	615	9.385	659	10.341	702	11.298	741	12.259	779	13.221
120	4.00	79.5	633	8.367	686	9.314	735	10.265	782	11.218	826	12.174	868	13.132
180	6.00	89.2	699	8.301	759	9.241	812	10.188	864	11.136	912	12.088	958	13.042
240	8.00	94	733	8.267	794	9.206	850	10.150	904	11.096	955	12.045	1002	12.998
300	10.00	96.4	749	8.251	812	9.188	869	10.131	925	11.075	976	12.024	1025	12.975
360	12.00	97.6	757	8.243	821	9.179	879	10.121	935	11.065	987	12.013	1036	12.964
540	18.00	98.8	766	8.234	830	9.170	888	10.112	945	11.055	997	12.003	1047	12.953
720	24.00	100	774	8.226	839	9.161	898	10.102	955	11.045	1008	11.992	1058	12.942

Note: (\*) Does not include settlements due to secondary consolidation

**Summary of Surcharge Requirements**

Initial Fill Height (m)	9	10	11	12	13	14
Final Height above O.G.S. (m)						
8.226	> 24.00	> 6.00	> 4.00	> 3.00	> 3.00	> 2.00
9.161		> 24.00	> 6.00	> 4.00	> 4.00	> 3.00
10.102			> 24.00	> 6.00	> 4.00	> 4.00
11.045				> 24.00	> 8.00	> 4.00
11.992					> 24.00	> 8.00
12.942						> 24.00

**HIGHWAY 11 - TROUT CREEK BY-PASS**  
**NORTH INTERCHANGE - SURCHARGE ANALYSIS**  
**Highway 11 North of Bridge - CPTUN5**  
**Most Likely Pre-Consolidation Pressures - 60 m wide embankment top**

Initial Fill Height (m)	2 m	3 m	4 m	5 m	5 m	5 m
Immediate Settlement (mm)	19 mm	29 mm	38 mm	48 mm	48 mm	48 mm
Primary Consol. Settl. (mm)	39 mm	52 mm	63 mm	72 mm	72 mm	72 mm
Total Settlement (mm) (*)	58 mm	81 mm	101 mm	120 mm	120 mm	120 mm
Final Height above O.G.S. (m)	1.942 m	2.919 m	3.899 m	4.88 m	4.88 m	4.88 m

Time		% Consolidation	Settlement	Height Above	Settlement	Height Above	Settlement	Height Above	Settlement	Height Above	Settlement	Height Above	Settlement	Height Above
(days)	(months)	U%	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)
15	0.50	28.9	30	1.970	44	2.956	56	3.944	69	4.931	69	4.931	69	4.931
30	1.00	41	35	1.965	50	2.950	64	3.936	78	4.922	78	4.922	78	4.922
60	2.00	60.2	42	1.958	60	2.940	76	3.924	91	4.909	91	4.909	91	4.909
90	3.00	69.9	46	1.954	65	2.935	82	3.918	98	4.902	98	4.902	98	4.902
120	4.00	79.5	50	1.950	70	2.930	88	3.912	105	4.895	105	4.895	105	4.895
180	6.00	89.2	54	1.946	75	2.925	94	3.906	112	4.888	112	4.888	112	4.888
240	8.00	94	56	1.944	78	2.922	97	3.903	116	4.884	116	4.884	116	4.884
300	10.00	96.4	57	1.943	79	2.921	99	3.901	117	4.883	117	4.883	117	4.883
360	12.00	97.6	57	1.943	80	2.920	99	3.901	118	4.882	118	4.882	118	4.882
540	18.00	98.8	58	1.942	80	2.920	100	3.900	119	4.881	119	4.881	119	4.881
720	24.00	100	58	1.942	81	2.919	101	3.899	120	4.880	120	4.880	120	4.880

Note: (\*) Does not include settlements due to secondary consolidation

**Summary of Surcharge Requirements**

Initial Fill Height (m)	2	3	4	5	5	5
Final Height above O.G.S. (m)						
1.942	> 24.00	> 1.00	> 0.50	#N/A	#N/A	#N/A
2.919		> 24.00	> 2.00	> 1.00	> 1.00	> 1.00
3.899			> 24.00	> 3.00	> 3.00	> 3.00
4.88				> 24.00	> 24.00	> 24.00
4.88					> 24.00	> 24.00
4.88						> 24.00

**HIGHWAY 11 - TROUT CREEK BY-PASS**  
**NORTH INTERCHANGE - SURCHARGE ANALYSIS**  
**Highway 11 North of Bridge - CPTUN5**  
**Reduced Pre-Consolidation Pressures - 60 m wide embankment top**

Initial Fill Height (m)	2 m	3 m	4 m	5 m	5 m	5 m
Immediate Settlement (mm)	19 mm	29 mm	38 mm	48 mm	48 mm	48 mm
Primary Consol. Settl. (mm)	103 mm	213 mm	322 mm	416 mm	416 mm	416 mm
Total Settlement (mm) (*)	122 mm	242 mm	360 mm	464 mm	464 mm	464 mm
Final Height above O.G.S. (m)	1.878 m	2.758 m	3.64 m	4.536 m	4.536 m	4.536 m

Time (days)	Time (months)	% Consolidation U%	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)
15	0.50	28.9	49	1.951	91	2.909	131	3.869	168	4.832	168	4.832	168	4.832
30	1.00	41	61	1.939	116	2.884	170	3.830	219	4.781	219	4.781	219	4.781
60	2.00	60.2	81	1.919	157	2.843	232	3.768	298	4.702	298	4.702	298	4.702
90	3.00	69.9	91	1.909	178	2.822	263	3.737	339	4.661	339	4.661	339	4.661
120	4.00	79.5	101	1.899	198	2.802	294	3.706	379	4.621	379	4.621	379	4.621
180	6.00	89.2	111	1.889	219	2.781	325	3.675	419	4.581	419	4.581	419	4.581
240	8.00	94	116	1.884	229	2.771	341	3.659	439	4.561	439	4.561	439	4.561
300	10.00	96.4	118	1.882	234	2.766	348	3.652	449	4.551	449	4.551	449	4.551
360	12.00	97.6	120	1.880	237	2.763	352	3.648	454	4.546	454	4.546	454	4.546
540	18.00	98.8	121	1.879	239	2.761	356	3.644	459	4.541	459	4.541	459	4.541
720	24.00	100	122	1.878	242	2.758	360	3.640	464	4.536	464	4.536	464	4.536

Note: (\*) Does not include settlements due to secondary consolidation

**Summary of Surcharge Requirements**

Initial Fill Height (m)	2	3	4	5	5	5
Final Height above O.G.S. (m)						
1.878	> 24.00	> 1.00	#N/A	#N/A	#N/A	#N/A
2.758		> 24.00	> 2.00	> 1.00	> 1.00	> 1.00
3.64			> 24.00	> 3.00	> 3.00	> 3.00
4.536				> 24.00	> 24.00	> 24.00
4.536					> 24.00	> 24.00
4.536						> 24.00

**HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - SURCHARGE ANALYSIS  
East Approach - Sta.10+120 to 10+240 - CPTUN1  
Most Likely Pre-Consolidation Pressures - 40 m wide embankment top**

Initial Fill Height (m)	6 m	7 m	8 m	9 m	10 m	11 m
Immediate Settlement (mm)	77 mm	90 mm	104 mm	117 mm	130 mm	143 mm
Primary Consol. Settl. (mm)	169 mm	232 mm	290 mm	342 mm	390 mm	435 mm
Total Settlement (mm) (*)	246 mm	322 mm	394 mm	459 mm	520 mm	578 mm
Final Height above O.G.S. (m)	5.754 m	6.678 m	7.606 m	8.541 m	9.48 m	10.422 m

Time		% Consolidation	Settlement	Height Above	Settlement	Height Above	Settlement	Height Above	Settlement	Height Above	Settlement	Height Above	Settlement	Height Above
(days)	(months)	U%	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)
15	0.50	34.4	135	5.885	170	6.830	204	7.796	235	8.765	264	9.736	293	10.707
30	1.00	53.1	167	5.833	213	6.787	258	7.742	299	8.701	337	9.663	374	10.628
60	2.00	68.8	193	5.807	250	6.750	304	7.696	352	8.648	398	9.602	442	10.558
90	3.00	84.4	220	5.780	286	6.714	349	7.651	406	8.594	459	9.541	510	10.490
120	4.00	90.6	230	5.770	300	6.700	367	7.633	427	8.573	483	9.517	537	10.463
180	6.00	96.9	241	5.759	315	6.685	385	7.615	448	8.552	508	9.492	565	10.435
240	8.00	98.1	243	5.757	318	6.682	388	7.612	453	8.547	513	9.487	570	10.430
300	10.00	99.1	244	5.756	320	6.680	391	7.609	456	8.544	516	9.484	574	10.426
360	12.00	99.7	245	5.755	321	6.679	393	7.607	458	8.542	519	9.481	577	10.423
540	18.00	100	246	5.754	322	6.678	394	7.606	459	8.541	520	9.480	578	10.422
720	24.00	100	246	5.754	322	6.678	394	7.606	459	8.541	520	9.480	578	10.422

Note: (\*) Does not include settlements due to secondary consolidation

**Summary of Surcharge Requirements**

Initial Fill Height (m)	6	7	8	9	10	11
Final Height above O.G.S. (m)						
5.754	> 24.00	> 1.00	> 0.50	> 0.50	#N/A	#N/A
6.678		> 24.00	> 2.00	> 1.00	> 0.50	> 0.50
7.606			> 24.00	> 2.00	> 1.00	> 1.00
8.541				> 24.00	> 2.00	> 2.00
9.48					> 24.00	> 3.00
10.422						> 24.00

**HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - SURCHARGE ANALYSIS  
East Approach - Sta.10+120 to 10+240 - CPTUN1  
Reduced Pre-Consolidation Pressures - 40 m wide embankment top**

Initial Fill Height (m)	6 m	7 m	8 m	9 m	10 m	11 m
Immediate Settlement (mm)	77 mm	90 mm	104 mm	117 mm	130 mm	143 mm
Primary Consol. Settl.(mm)	560 mm	624 mm	681 mm	734 mm	781 mm	826 mm
Total Settlement (mm) (*)	637 mm	714 mm	785 mm	851 mm	911 mm	969 mm
Final Height above O.G.S. (m)	5.363 m	6.286 m	7.215 m	8.149 m	9.089 m	10.031 m

Time (days)	Time (months)	% Consolidation U%	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)
15	0.50	34.4	270	5.730	305	6.695	338	7.662	369	8.631	399	9.601	427	10.573
30	1.00	53.1	374	5.626	421	6.579	466	7.534	507	8.493	545	9.455	582	10.418
60	2.00	68.8	462	5.538	519	6.481	573	7.427	622	8.378	667	9.333	711	10.289
90	3.00	84.4	550	5.450	617	6.383	679	7.321	736	8.264	789	9.211	840	10.160
120	4.00	90.6	584	5.416	655	6.345	721	7.279	782	8.218	838	9.162	891	10.109
180	6.00	96.9	620	5.380	695	6.305	764	7.236	828	8.172	887	9.113	943	10.057
240	8.00	98.1	626	5.374	702	6.298	772	7.228	837	8.163	896	9.104	953	10.047
300	10.00	99.1	632	5.368	708	6.292	779	7.221	844	8.156	904	9.096	962	10.038
360	12.00	99.7	635	5.365	712	6.288	783	7.217	849	8.151	909	9.091	967	10.033
540	18.00	100	637	5.363	714	6.286	785	7.215	851	8.149	911	9.089	969	10.031
720	24.00	100	637	5.363	714	6.286	785	7.215	851	8.149	911	9.089	969	10.031

Note: (\*) Does not include settlements due to secondary consolidation

**Summary of Surcharge Requirements**

Initial Fill Height (m)	6	7	8	9	10	11
Final Height above O.G.S. (m)						
5.363	> 24.00	> 3.00	> 2.00	> 2.00	> 1.00	> 1.00
6.286		> 24.00	> 3.00	> 2.00	> 2.00	> 2.00
7.215			> 24.00	> 4.00	> 2.00	> 2.00
8.149				> 24.00	> 4.00	> 3.00
9.089					> 24.00	> 4.00
10.031						> 24.00

**HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - SURCHARGE ANALYSIS  
East Approach - Sta. >10+240 - CPTUN4  
Most Likely Pre-Consolidation Pressures - 30 m wide embankment top**

Initial Fill Height (m)	3 m	4 m	5 m	6 m	7 m	8 m
Immediate Settlement (mm)	39 mm	53 mm	66 mm	80 mm	93 mm	107 mm
Primary Consol. Settl. (mm)	87 mm	179 mm	270 mm	354 mm	430 mm	501 mm
Total Settlement (mm) (*)	126 mm	232 mm	336 mm	434 mm	523 mm	608 mm
Final Height above O.G.S. (m)	2.874 m	3.768 m	4.664 m	5.566 m	6.477 m	7.392 m

Time		% Consolidation	Settlement	Height Above	Settlement	Height Above	Settlement	Height Above	Settlement	Height Above	Settlement	Height Above	Settlement	Height Above
(days)	(months)	U%	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)
15	0.50	21.3	58	2.942	91	3.909	124	4.876	155	5.845	185	6.815	214	7.786
30	1.00	31.9	67	2.933	110	3.890	152	4.848	193	5.807	230	6.770	267	7.733
60	2.00	44.7	78	2.922	133	3.867	187	4.813	238	5.762	285	6.715	331	7.669
90	3.00	55.3	87	2.913	152	3.848	215	4.785	276	5.724	331	6.669	384	7.616
120	4.00	63.8	95	2.905	167	3.833	238	4.762	306	5.694	367	6.633	427	7.573
180	6.00	74.5	104	2.896	186	3.814	267	4.733	344	5.656	413	6.587	480	7.520
240	8.00	83	111	2.889	202	3.798	290	4.710	374	5.626	450	6.550	523	7.477
300	10.00	89.4	117	2.883	213	3.787	307	4.693	396	5.604	477	6.523	555	7.445
360	12.00	91.5	119	2.881	217	3.783	313	4.687	404	5.596	486	6.514	565	7.435
540	18.00	97.9	124	2.876	228	3.772	330	4.670	427	5.573	514	6.486	597	7.403
720	24.00	98.7	125	2.875	230	3.770	332	4.668	429	5.571	517	6.483	601	7.399

Note: (\*) Does not include settlements due to secondary consolidation

**Summary of Surcharge Requirements**

Initial Fill Height (m)	3	4	5	6	7	8
Final Height above O.G.S. (m)						
2.874	> maximum time above	> 1.00	> 0.50	#N/A	#N/A	#N/A
3.768		> maximum time above	> 3.00	> 1.00	> 1.00	> 0.50
4.664			> maximum time above	> 4.00	> 3.00	> 2.00
5.566				> maximum time above	> 6.00	> 4.00
6.477					> maximum time above	> 8.00
7.392						> maximum time above

**HIGHWAY 11 - TROUT CREEK BY-PASS**  
**NORTH INTERCHANGE - SURCHARGE ANALYSIS**  
**East Approach - Sta. >10+240 - CPTUN4**  
**Reduced Pre-Consolidation Pressures - 30 m wide embankment top**

Initial Fill Height (m)	3 m	4 m	5 m	6 m	7 m	8 m
Immediate Settlement (mm)	39 mm	53 mm	66 mm	80 mm	93 mm	107 mm
Primary Consol. Settl. (mm)	382 mm	482 mm	573 mm	656 mm	733 mm	804 mm
Total Settlement (mm) (*)	421 mm	535 mm	639 mm	736 mm	826 mm	911 mm
Final Height above O.G.S. (m)	2.579 m	3.465 m	4.361 m	5.264 m	6.174 m	7.089 m

Time (days)	Time (months)	% Consolidation U <sub>x</sub>	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)
15	0.50	21.3	120	2.880	156	3.844	188	4.812	220	5.780	249	6.751	278	7.722
30	1.00	31.9	161	2.839	207	3.793	249	4.751	289	5.711	327	6.673	363	7.637
60	2.00	44.7	210	2.790	268	3.732	322	4.678	373	5.627	421	6.579	466	7.534
90	3.00	55.3	250	2.750	320	3.680	383	4.617	443	5.557	498	6.502	552	7.448
120	4.00	63.8	283	2.717	361	3.639	432	4.568	499	5.501	561	6.439	620	7.380
180	6.00	74.5	324	2.676	412	3.588	493	4.507	569	5.431	639	6.361	706	7.294
240	8.00	83	356	2.644	453	3.547	542	4.458	624	5.376	701	6.299	774	7.226
300	10.00	89.4	381	2.619	484	3.516	578	4.422	666	5.334	748	6.252	826	7.174
360	12.00	91.5	389	2.611	494	3.506	590	4.410	680	5.320	764	6.236	843	7.157
540	18.00	97.9	413	2.587	525	3.475	627	4.373	722	5.278	811	6.189	894	7.106
720	24.00	98.7	416	2.584	529	3.471	632	4.368	727	5.273	816	6.184	901	7.099

Note: (\*) Does not include settlements due to secondary consolidation

**Summary of Surcharge Requirements**

Initial Fill Height (m)	3	4	5	6	7	8
Final Height above O.G.S. (m)						
2.579	> maximum time above	> 6.00	> 3.00	> 2.00	> 2.00	> 1.00
3.465		> maximum time above	> 6.00	> 4.00	> 3.00	> 2.00
4.361			> maximum time above	> 8.00	> 4.00	> 4.00
5.264				> maximum time above	> 8.00	> 6.00
6.174					> maximum time above	> 10.00
7.089						> maximum time above



**HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - SURCHARGE ANALYSIS  
EW-N & SW Ramps - Sta.13+160 to 13+270 - CPTUN2  
Most Likely Pre-Consolidation Pressures - 30 m wide embankment top**

Initial Fill Height (m)	5 m	6 m	7 m	8 m	9 m	10 m
Immediate Settlement (mm)	61 mm	74 mm	87 mm	99 mm	112 mm	125 mm
Primary Consol. Settl. (mm)	107 mm	177 mm	250 mm	318 mm	381 mm	439 mm
Total Settlement (mm) (*)	168 mm	251 mm	337 mm	417 mm	493 mm	564 mm
Final Height above O.G.S. (m)	4.832 m	5.749 m	6.663 m	7.583 m	8.507 m	9.436 m

Time		% Consolidation	Settlement	Height Above	Settlement	Height Above	Settlement	Height Above	Settlement	Height Above	Settlement	Height Above	Settlement	Height Above
(days)	(months)	U%	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)
15	0.50	21	83	4.917	111	5.889	140	6.861	166	7.834	192	8.808	217	9.783
30	1.00	31.6	95	4.905	130	5.870	166	6.834	199	7.801	232	8.768	264	9.736
60	2.00	50	115	4.886	163	5.838	212	6.788	258	7.742	303	8.698	345	9.656
90	3.00	65.8	131	4.869	190	5.810	252	6.749	308	7.692	363	8.637	414	9.586
120	4.00	76.3	143	4.857	209	5.791	278	6.722	342	7.658	403	8.597	460	9.540
180	6.00	86.8	154	4.846	228	5.772	304	6.696	375	7.625	443	8.557	506	9.494
240	8.00	92.1	160	4.840	237	5.763	317	6.683	392	7.608	463	8.537	529	9.471
300	10.00	94.7	162	4.838	242	5.758	324	6.676	400	7.600	473	8.527	541	9.459
360	12.00	96.1	164	4.836	244	5.756	327	6.673	405	7.595	478	8.522	547	9.453
540	18.00	97.4	165	4.835	246	5.754	331	6.670	409	7.591	483	8.517	553	9.447
720	24.00	98.4	166	4.834	248	5.752	333	6.667	412	7.588	487	8.513	557	9.443

Note: (\*) Does not include settlements due to secondary consolidation

**Summary of Surcharge Requirements**

Initial Fill Height (m)	5	6	7	8	9	10
Final Height above O.G.S. (m)						
4.832	> maximum time above	> 2.00	> 1.00	> 0.50	#N/A	#N/A
5.749		> maximum time above	> 2.00	> 1.00	> 1.00	> 0.50
6.663			> maximum time above	> 3.00	> 2.00	> 1.00
7.583				> maximum time above	> 4.00	> 3.00
8.507					> maximum time above	> 4.00
9.436						> maximum time above

**HIGHWAY 11 - TROUT CREEK BY-PASS**  
**NORTH INTERCHANGE - SURCHARGE ANALYSIS**  
**EW-N & SW Ramps - Sta.13+160 to 13+270 - CPTUN2**  
**Reduced Pre-Consolidation Pressures - 30 m wide embankment top**

Initial Fill Height (m)	5 m	6 m	7 m	8 m	9 m	10 m
Immediate Settlement (mm)	61 mm	74 mm	87 mm	99 mm	112 mm	125 mm
Primary Consol. Settl. (mm)	561 mm	644 mm	719 mm	788 mm	851 mm	909 mm
Total Settlement (mm) (*)	622 mm	718 mm	806 mm	887 mm	963 mm	1034 mm
Final Height above O.G.S. (m)	4.378 m	5.282 m	6.194 m	7.113 m	8.037 m	8.966 m

Time (days)	Time (months)	% Consolidation U%	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)
15	0.50	31.8	239	4.761	279	5.721	316	6.684	350	7.650	383	8.617	414	9.586
30	1.00	44.7	312	4.688	362	5.638	408	6.592	451	7.549	492	8.508	531	9.469
60	2.00	61.2	404	4.596	468	5.532	527	6.473	581	7.419	633	8.367	681	9.319
90	3.00	71.8	464	4.536	536	5.464	603	6.397	665	7.335	723	8.277	778	9.222
120	4.00	78.8	503	4.497	581	5.419	654	6.346	720	7.280	783	8.217	841	9.159
180	6.00	87	549	4.451	634	5.366	713	6.287	785	7.215	852	8.148	916	9.084
240	8.00	91.8	576	4.424	665	5.335	747	6.253	822	7.178	893	8.107	959	9.041
300	10.00	94.1	589	4.411	680	5.320	764	6.236	841	7.159	913	8.087	980	9.020
360	12.00	95.3	596	4.404	688	5.312	772	6.228	850	7.150	923	8.077	991	9.009
540	18.00	97.1	606	4.394	699	5.301	785	6.215	864	7.136	938	8.062	1008	8.992
720	24.00	97.7	609	4.391	703	5.297	789	6.211	869	7.131	943	8.057	1013	8.987

Note: (\*) Does not include settlements due to secondary consolidation

**Summary of Surcharge Requirements**

Initial Fill Height (m)	5	6	7	8	9	10
Final Height above O.G.S. (m)						
4.378	> maximum time above	> 4.00	> 3.00	> 2.00	> 1.00	> 1.00
5.282		> maximum time above	> 6.00	> 3.00	> 2.00	> 2.00
6.194			> maximum time above	> 6.00	> 4.00	> 3.00
7.113				> maximum time above	> 6.00	> 4.00
8.037					> maximum time above	> 8.00
8.966						> maximum time above

**HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - SURCHARGE ANALYSIS  
EW-N & SW Ramps - Sta.13+270 to 13+350 - CPTUN3  
Most Likely Pre-Consolidation Pressures - 30 m wide embankment top**

Initial Fill Height (m)	3 m	4 m	5 m	6 m	7 m	7 m
Immediate Settlement (mm)	31 mm	42 mm	52 mm	63 mm	74 mm	74 mm
Primary Consol. Settl (mm)	46 mm	70 mm	124 mm	177 mm	228 mm	228 mm
Total Settlement (mm) (*)	77 mm	112 mm	176 mm	240 mm	302 mm	302 mm
Final Height above O.G.S. (m)	2.923 m	3.888 m	4.824 m	5.76 m	6.698 m	6.698 m

Time (days)	Time (months)	% Consolidation U%	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)
15	0.50	18.8	40	2.960	55	3.945	75	4.925	96	5.904	117	6.883	117	6.883
30	1.00	25	43	2.958	60	3.941	83	4.917	107	5.893	131	6.869	131	6.869
60	2.00	37.5	48	2.952	68	3.932	99	4.902	129	5.871	160	6.841	160	6.841
90	3.00	56.3	57	2.943	81	3.919	122	4.878	163	5.837	202	6.798	202	6.798
120	4.00	75	66	2.935	95	3.906	145	4.855	196	5.804	245	6.755	245	6.755
180	6.00	87.5	71	2.929	103	3.897	161	4.840	218	5.782	274	6.727	274	6.727
240	8.00	90.6	73	2.927	105	3.895	164	4.836	223	5.777	281	6.719	281	6.719
300	10.00	93.8	74	2.926	108	3.892	168	4.832	229	5.771	288	6.712	288	6.712
360	12.00	96.9	76	2.924	110	3.890	172	4.828	235	5.765	295	6.705	295	6.705
540	18.00	98.8	76	2.924	111	3.889	175	4.825	238	5.762	299	6.701	299	6.701
720	24.00	100	77	2.923	112	3.888	176	4.824	240	5.760	302	6.698	302	6.698

Note: (\*) Does not include settlements due to secondary consolidation

**Summary of Surcharge Requirements**

Initial Fill Height (m)	3	4	5	6	7	7
Final Height above O.G.S. (m)						
2.923	> 24.00	> 2.00	> 0.50	#N/A	#N/A	#N/A
3.888		> 24.00	> 2.00	> 1.00	#N/A	#N/A
4.824			> 24.00	> 3.00	> 2.00	> 2.00
5.76				> 24.00	> 3.00	> 3.00
6.698					> 24.00	> 24.00
6.698						> 24.00

**HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - SURCHARGE ANALYSIS  
EW-N & SW Ramps - Sta.13+270 to 13+350 - CPTUN3  
Reduced Pre-Consolidation Pressures - 30 m wide embankment top**

Initial Fill Height (m)	3 m	4 m	5 m	6 m	7 m	7 m
Immediate Settlement (mm)	31 mm	42 mm	52 mm	63 mm	74 mm	74 mm
Primary Consol. Settl. (mm)	463 mm	568 mm	659 mm	740 mm	813 mm	813 mm
Total Settlement (mm) (*)	494 mm	610 mm	711 mm	803 mm	887 mm	887 mm
Final Height above O.G.S. (m)	2.506 m	3.39 m	4.289 m	5.197 m	6.113 m	6.113 m

Time (days)	Time (months)	% Consolidation U%	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)
15	0.50	35.8	197	2.803	245	3.755	288	4.712	328	5.672	365	6.635	365	6.635
30	1.00	49.3	259	2.741	322	3.678	377	4.623	428	5.572	475	6.525	475	6.525
60	2.00	64.2	328	2.672	407	3.593	475	4.525	538	5.462	596	6.404	596	6.404
90	3.00	73.1	369	2.631	457	3.543	534	4.466	604	5.396	668	6.332	668	6.332
120	4.00	79.1	397	2.603	491	3.509	573	4.427	648	5.352	717	6.283	717	6.283
180	6.00	86.6	432	2.568	534	3.466	623	4.377	704	5.296	778	6.222	778	6.222
240	8.00	91	452	2.548	559	3.441	652	4.348	736	5.264	814	6.186	814	6.186
300	10.00	93.3	463	2.537	572	3.428	667	4.333	753	5.247	833	6.167	833	6.167
360	12.00	94	466	2.534	576	3.424	671	4.329	759	5.241	838	6.162	838	6.162
540	18.00	94.8	470	2.530	580	3.420	677	4.323	765	5.235	845	6.155	845	6.155
720	24.00	95.5	473	2.527	584	3.416	681	4.319	770	5.230	850	6.150	850	6.150

Note: (\*) Does not include settlements due to secondary consolidation

**Summary of Surcharge Requirements**

Initial Fill Height (m)	3	4	5	6	7	7
Final Height above O.G.S. (m)						
2.506	> maximum time above	> 4.00	> 2.00	> 1.00	> 1.00	> 1.00
3.39		> maximum time above	> 4.00	> 3.00	> 2.00	> 2.00
4.289			> maximum time above	> 6.00	> 3.00	> 3.00
5.197				> maximum time above	> 6.00	> 6.00
6.113					> maximum time above	> maximum time above
6.113						> maximum time above

**HIGHWAY 11 - TROUT CREEK BY-PASS  
NORTH INTERCHANGE - SURCHARGE ANALYSIS  
EW-N Ramp Sta. 13+350 to 13+458 & Hwy 1 South of Bridge - BH11BP  
Most Likely Pre-Consolidation Pressures - 60 m wide embankment top**

Initial Fill Height (m)	2 m	3 m	4 m	5 m	6 m	6 m
Immediate Settlement (mm)	19 mm	29 mm	38 mm	48 mm	58 mm	58 mm
Primary Consol. Settl (mm)	40 mm	54 mm	82 mm	139 mm	197 mm	197 mm
Total Settlement (mm) (*)	59 mm	83 mm	120 mm	187 mm	255 mm	255 mm
Final Height above O.G.S. (m)	1.941 m	2.917 m	3.88 m	4.813 m	5.745 m	5.745 m

Time (days)	Time (months)	% Consolidation U%	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)	Settlement (mm)	Height Above O.G.S. (m)
15	0.50	22.2	28	1.972	41	2.959	56	3.944	79	4.921	102	5.898	102	5.898
30	1.00	33.3	32	1.968	47	2.953	65	3.935	94	4.906	124	5.876	124	5.876
60	2.00	44.4	37	1.963	53	2.947	74	3.926	110	4.890	145	5.855	145	5.855
90	3.00	55.6	41	1.959	59	2.941	84	3.916	125	4.875	168	5.832	168	5.832
120	4.00	61.1	43	1.957	62	2.938	88	3.912	133	4.867	178	5.822	178	5.822
180	6.00	66.7	46	1.954	65	2.935	93	3.907	141	4.859	189	5.811	189	5.811
240	8.00	77.8	50	1.950	71	2.929	102	3.898	156	4.844	211	5.789	211	5.789
300	10.00	83.3	52	1.948	74	2.926	106	3.894	164	4.836	222	5.778	222	5.778
360	12.00	88.9	55	1.945	77	2.923	111	3.889	172	4.828	233	5.767	233	5.767
540	18.00	94.4	57	1.943	80	2.920	115	3.885	179	4.821	244	5.756	244	5.756
720	24.00	100	59	1.941	83	2.917	120	3.880	187	4.813	255	5.745	255	5.745

Note: (\*) Does not include settlements due to secondary consolidation

**Summary of Surcharge Requirements**

Initial Fill Height (m)	2	3	4	5	6	6
Final Height above O.G.S. (m)						
1.941	> 24.00	> 2.00	> 0.50	#N/A	#N/A	#N/A
2.917		> 24.00	> 2.00	> 0.50	#N/A	#N/A
3.88			> 24.00	> 2.00	> 0.50	> 0.50
4.813				> 24.00	> 4.00	> 4.00
5.745					> 24.00	> 24.00
5.745						> 24.00

**HIGHWAY 11 - TROUT CREEK BY-PASS**  
**NORTH INTERCHANGE - SURCHARGE ANALYSIS**  
**EW-N Ramp Sta. 13+350 to 13+458 & Hwy 1 South of Bridge - BH11BP**  
**Reduced Pre-Consolidation Pressures - 60 m wide embankment top**

Initial Fill Height (m)	2 m	3 m	4 m	5 m	6 m	6 m
Immediate Settlement (mm)	19 mm	29 mm	38 mm	48 mm	58 mm	58 mm
Primary Consol. Settl.(mm)	399 mm	544 mm	667 mm	774 mm	869 mm	869 mm
Total Settlement (mm) (*)	418 mm	573 mm	705 mm	822 mm	927 mm	927 mm
Final Height above O.G.S. (m)	1.582 m	2.427 m	3.295 m	4.178 m	5.073 m	5.073 m

Time		% Consolidation	Settlement	Height Above	Settlement	Height Above	Settlement	Height Above	Settlement	Height Above	Settlement	Height Above	Settlement	Height Above
(days)	(months)	U%	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)	(mm)	O.G.S. (m)
15	0.50	22.2	108	1.892	150	2.850	186	3.814	220	4.780	251	5.749	251	5.749
30	1.00	33.3	152	1.848	210	2.790	260	3.740	306	4.694	347	5.653	347	5.653
60	2.00	44.4	196	1.804	271	2.729	334	3.666	392	4.608	444	5.556	444	5.556
90	3.00	55.6	241	1.759	331	2.669	409	3.591	478	4.522	541	5.459	541	5.459
120	4.00	61.1	263	1.737	361	2.639	446	3.554	521	4.479	589	5.411	589	5.411
180	6.00	66.7	285	1.715	392	2.608	483	3.517	564	4.436	638	5.362	638	5.362
240	8.00	77.8	329	1.671	452	2.548	557	3.443	650	4.350	734	5.266	734	5.266
300	10.00	83.3	351	1.649	482	2.518	594	3.406	693	4.307	782	5.218	782	5.218
360	12.00	88.9	374	1.626	513	2.487	631	3.369	736	4.264	831	5.169	831	5.169
540	18.00	94.4	396	1.604	543	2.457	668	3.332	779	4.221	878	5.122	878	5.122
720	24.00	100	418	1.582	573	2.427	705	3.295	822	4.178	927	5.073	927	5.073

Note: (\*) Does not include settlements due to secondary consolidation

**Summary of Surcharge Requirements**

Initial Fill Height (m)	2	3	4	5	6	6
Final Height above O.G.S. (m)						
1.582	> 24.00	> 6.00	> 3.00	> 2.00	> 1.00	> 1.00
2.427		> 24.00	> 8.00	> 6.00	> 3.00	> 3.00
3.295			> 24.00	> 10.00	> 6.00	> 6.00
4.178				> 24.00	> 10.00	> 10.00
5.073					> 24.00	> 24.00
5.073						> 24.00

**HIGHWAY 11 - TROUT CREEK BY PASS - NORTH INTERCHANGE  
WICK DRAIN DESIGN ASSUMPTIONS**

Site Location	Test Hole	Ch (m <sup>2</sup> /y)	Cv (m <sup>2</sup> /y)	Embankment Load (kPa)	Wick Drain Drainage Length (m)	Disturbance Ratios		Discharge Capacity qw (m <sup>3</sup> /s)
						Diameter Ratio (s)	Permeability Ratio (kc/kc')	
McCarthy St - Sta. 10+014 to 10+120	CPTUN5	142	28	240	5	3	3	1.00E-05
McCarthy St (Sta. 10+120 to 10+240); W-N Ramp (13+091 to 13+160); S-W Ramp(13+630 to 13+540)	CPTUN1	168	34	210	10	3	3	1.00E-05
East Approach - Sta.10+240 to 10+310	CPTUN4	100	20	150	17	3	3	1.00E-05
EW-N Ramp 13+160 to 13+270; S-W Ramp (Sta.13+540 to 13+450)	CPTUN2	95	19	180	12	3	3	1.00E-05

Site Location	Test Hole	Target Percentage Consol. and Time					
		Schedule 1		Schedule 2		Schedule 3	
		After Stage 1	After Stage 2			After Stage 1	After Stage 2
McCarthy St - Sta. 10+014 to 10+120	CPTUN5	75% in 1 month	100% in 12 months	No wicks required		75% before Stage 2	100% in 6 months
McCarthy St (Sta. 10+120 to 10+240); W-N Ramp (13+091 to 13+160); S-W Ramp(13+630 to 13+540)	CPTUN1	75% in 1 month	100% in 12 months	No wicks required		75% before Stage 2	100% in 6 months
East Approach - Sta.10+240 to 10+310	CPTUN4	No wicks required		No wicks required		100% in 6 months	
EW-N Ramp 13+160 to 13+270; S-W Ramp (Sta.13+540 to 13+450)	CPTUN2	No wicks required		No wicks required		100% in 6 months	

**Schedule 1:** 2 months: Site preparation (includes installation of wick drains)  
3 months: Embankment Construction  
12 months: Waiting period for stabilization of settlements

**Schedule 2:** Embankment construction and stabilization of settlements in 12 months

**Schedule 3:** Embankment construction and stabilization of settlements in 6 months

TABLE B18

**NEW HANSBO METHOD (combined with Lambe & Whitman's book) recommendations**  
**"Consolidation of Clay by Band-Shaped Prefabricated Drains"**  
**Ground Engineering, Vol.12 No.5, 1979**  
**Formulation according to Equation 1 - Including well resistance and smearing**

Job Number: 19-1104-4  
 Title: Highway 11 - Trout Creek By-Pass  
 Case: North Interchange  
 Sub-case: Test Hole CPTUN5- East Abutment - Sta.10+040 to 10+120 - Most Likely Pc  
 Construction Schedule 1 - Target: 75% in one month and 100% in one year  
 Berm Width = 9 m

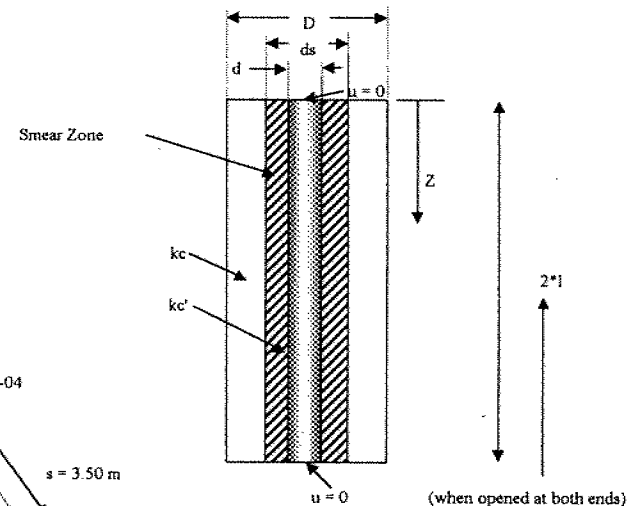
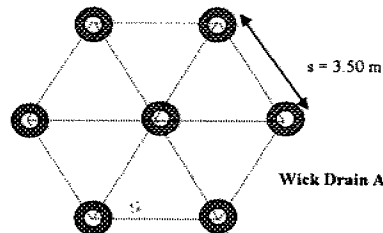
**INPUT PARAMETERS**

D	3.67	m	diameter of dewatered soil cylinder (Triangular Spacing equal to, $s =$	3.50	m)
d	0.065	m	equivalent diameter of band-shaped drain: $2(b+t)/\pi$ ; $n =$	56.5	
$C_R$	4.50E-06	$m^2/s$	consider reducing $c_h$ to account for smear; $Ch/C_v$ is often 2 to 5		
$C_v$	9.01E-07	$m^2/s$	determined by the oedometer test		
$\lambda$	9.01E-07	$m^2/s$	$=k_w/(r_w \cdot m_v)$ ; or $\lambda = C_v$ obtained from the oedometer test (Hansbo 1979)		
$d_s$	0.20	m	diameter of the smear zone (typically equal to 1.5 to 3 times d); $s=ds/d =$	3	
$k_c$	1.00E-09	m/s	undisturbed soil permeability		
$k'_c$	3.33E-10	m/s	soil permeability within the smear zone; $k_c/k'_c =$	3.00	
$q_w$	1.00E-05	$m^3/s$	drain discharge capacity; $k_c/q_w =$	1.00E-04	; well resistance cannot be ignored if $k_c/q_w > 3.33E-04$
l	5.00	m	length of the drain when open at one end only		
			half length of the drain when open at both ends		

Layer  
 Surcharge (kPa)  
 Drainage Path (m)  
 Settlement due to Primary Consolidation  
 n  
 $\alpha$

ML-CL  
 240.00 kPa  
 4.50 m  
 431 mm  
 56 (D/d; should always be >12)  
 0.3759384 ((D/d); regression from Figure 3 of the paper)

U<sub>v</sub> target: 41 %  
 TargetTime (days): 30 days  
 Time for Drainage Path: 30 days



Time Increment for table below = 0.17 month  
 Resultant Maximum Time = 10.17 months

% Consolidation	Time required (months)	
	U <sub>v</sub> and U <sub>h</sub>	U <sub>h</sub> only
50	0.50	0.67
75	0.83	1.17
90	1.50	1.83
98	2.50	3.17

TABLE B19



**NEW HANSBO METHOD ACCORDING TO ROBERTSON & CAMPANELLA 1988**  
(combined with Lambe & Whitman's book recommendations)

Hansbo 1979, "Consolidation of Clay by band-shaped prefabricated drains"  
Ground Engineering, Vol.12 No.5, 1979  
Formulation according to Equation 2 - No well resistance

Robertson and Campanella, 1988, "Prediction of wick drain performance using piezometer cone data"  
Canadian Geotechnical Journal, 25, 56-61 (1988)

Job Number: 19-1104-4  
Title: Highway 11 - Trout Creek By-Pass  
Case: North Interchange  
Sub-case: Test Hole CPTUN5- East Abutment - Sta.10+040 to 10+120 - Most Likely Pc  
Construction Schedule 1 - Target: 75% in one month and 100% in one year  
Berm Width = 9 m  
D 3.67 m diameter of dewatered soil cylinder (Triangular Spacing)  $s = 3.50$  m  
d 0.065 m equivalent diameter of band-shaped drain:  $2(b+t)/\pi$   
 $C_H$  4.50E-06 m<sup>2</sup>/s consider reducing  $C_H$  to account for smear;  $C_H/C_v$  is often 2 to 5  
 $C_v$  9.01E-07 m<sup>2</sup>/s determined by the oedometer test  
 $\lambda$  4.50E-07 m<sup>2</sup>/s  $=k_h/(\gamma_w \cdot m_v)$ ; for Piezocone  $\gamma = 0.1 \cdot C_H$  (Robertson & Campanella, 1988)  
Layer ML-CL  
Surcharge (kPa) 240.00 kPa  
Drainage Path (m) 4.50 m  
Settlement due to Primary Consolidation 431 mm  
 $n$  56 ( $D/d$ ; should always be  $>12$ )  
 $\alpha$  0.3759384  $f(D/d)$ ; regression from Figure 3 of the paper)

Time Increment for table below = 0.17 month  
Resultant Maximum Time = 10.17 months



% Consolidation	Time required (months)	
	Uv and Uh	Uh only
50	0.50	0.83
75	1.00	1.67
90	2.00	3.67
98	4.17	10.17

**NEW HANSBO METHOD (combined with Lambe & Whitman's book) recommendations**  
**"Consolidation of Clay by Band-Shaped Prefabricated Drains"**  
**Ground Engineering, Vol.12 No.5, 1979**  
**Formulation according to Equation 1 - Including well resistance and smearing**

Job Number: 19-1104-4  
 Title: Highway 11 - Trout Creek By-Pass  
 Case: North Interchange  
 Sub-case: Test Hole CPTUN5- East Abutment - Sta.10+040 to 10+120 - Most Likely Pc  
 Construction Schedule 3: Complete construction in 6 months (75% in 1 month between Stages 1 and 2 and 100% in 2.5 months after Stage 2)  
 (allowing 6 weeks for construction of Stage 1 and 1 week for construction of Stage 2)  
 9 m wide berm

**INPUT PARAMETERS**

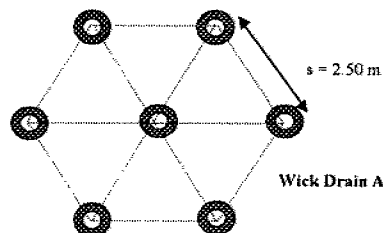
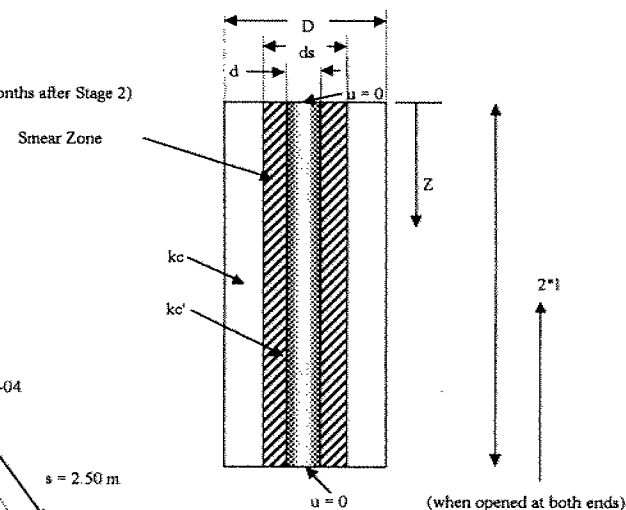
D	2.625	m	diameter of dewatered soil cylinder (Triangular Spacing equal to, $s =$	2.50	m)
d	0.065	m	equivalent diameter of band-shaped drain: $2(b+t)/\pi$ ; $n =$	40.4	
$C_H$	4.50E-06	$m^2/s$	consider reducing $c_u$ to account for smear, $C_H/C_v$ is often 2 to 5		
$C_v$	9.01E-07	$m^2/s$	determined by the oedometer test		
$\lambda$	9.01E-07	$m^2/s$	$=k_w/(\gamma_w \cdot m_v)$ , or $\lambda = C_v$ obtained from the oedometer test (Hansbo 1979)		
$d_s$	0.20	m	diameter of the smear zone (typically equal to 1.5 to 3 times d); $s = d_s/d =$	3	
$k_c$	1.00E-09	m/s	undisturbed soil permeability		
$k'_c$	3.33E-10	m/s	soil permeability within the smear zone, $k_c/k'_c =$	3.00	
$q_w$	1.00E-05	$m^3/s$	drain discharge capacity; $k_c/q_w =$ 1.00E-04 ;well resistance cannot be ignored if $k_c/q_w > 3.33E-04$		
l	5.00	m	length of the drain when open at one end only half length of the drain when open at both ends		

Layer  
 Surcharge (kPa)  
 Drainage Path (m)  
 Settlement due to Primary Consolidation  
 $n$   
 $\alpha$

ML-CL	240.00	kPa	U <sub>v</sub> target:	41 %
	4.50	m	TargetTime (days):	30 days
	431	mm	Time for Drainage Path:	30 days
	40	(D/d; should always be >12)		
	0.3532756	f(D/d); regression from Figure 3 of the paper)		

Time Increment for table below =  
 Resultant Maximum Time =

0.17 month  
 10.17 months



% Consolidation	Time required (months)	
	U <sub>v</sub> and U <sub>h</sub>	U <sub>h</sub> only
50	0.33	0.33
75	0.50	0.67
90	0.83	1.00
98	1.33	1.50

TABLE B21

**NEW HANSBO METHOD ACCORDING TO ROBERTSON & CAMPANELLA 1988**  
(combined with Lambe & Whitman's book recommendations)

Hansbo 1979, "Consolidation of Clay by band-shaped prefabricated drains"  
Ground Engineering, Vol.12 No.5, 1979  
Formulation according to Equation 2 - No well resistance

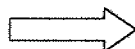
Robertson and Campanella, 1988, "Prediction of wick drain performance using piezometer cone data"  
Canadian Geotechnical Journal, 25, 56-61 (1988)

Job Number: 19-1104-4  
Title: Highway 11 - Trout Creek By-Pass  
Case: North Interchange  
Sub-case: Test Hole CPTUN5- East Abutment - Sta.10+040 to 10+120 - Most Likely Pc  
Construction Schedule 3: Complete construction in 6 months (75% in 1 month between Stages 1 and 2 and 100% in 2.5 months after Stage 2)  
(allowing 6 weeks for construction of Stage 1 and 1 week for construction of Stage 2)  
9 m wide berm

**INPUT PARAMETERS**

D	2.625	m	diameter of dewatered soil cylinder (Triangular Spacing)	$s = 2.50$ m
d	0.065	m	equivalent diameter of band-shaped drain: $2(b+t)/\pi$	
$C_H$	4.50E-06	m <sup>2</sup> /s	consider reducing $C_H$ to account for smear; $C_H/C_v$ is often 2 to 5	
$C_v$	9.01E-07	m <sup>2</sup> /s	determined by the oedometer test	
$\lambda$	4.50E-07	m <sup>2</sup> /s	$=k_w/(\gamma_w \cdot m_v)$ , for Piezocone $\gamma = 0.1 \cdot C_h$ (Robertson & Campanella, 1988)	
Layer	ML-CL			
Surcharge (kPa)	240.00	kPa		
Drainage Path (m)	4.50	m		
Settlement due to Primary Consolidation	431	mm		
n	40	(D/d; should always be >12)		
$\alpha$	0.3532756	$f(D/d)$ ; regression from Figure 3 of the paper)		

Time Increment for table below = 0.17 month  
Resultant Maximum Time = 10.17 months



% Consolidation	Time required (months)	
	Uv and Uh	Uh only
50	0.33	0.33
75	0.50	0.83
90	1.00	1.50
98	2.50	4.17

**NEW HANSBO METHOD (combined with Lambe & Whitman's book) recommendations**  
**"Consolidation of Clay by Band-Shaped Prefabricated Drains"**  
**Ground Engineering, Vol.12 No.5, 1979**  
**Formulation according to Equation 1 - Including well resistance and smearing**

Job Number: 19-1104-4  
 Title: Highway 11 - Trout Creek By-Pass  
 Case: North Interchange  
 Sub-case: Test Hole CPTUN1- East Approach - Sta.10+120 to 10+240 - Most Likely Pc  
 Construction Schedule 1: Target: 75% in one month and 100% in one year  
 Berm Width: 9m

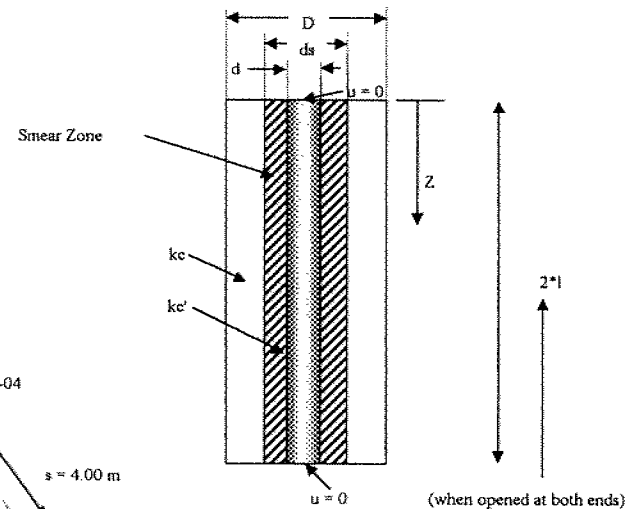
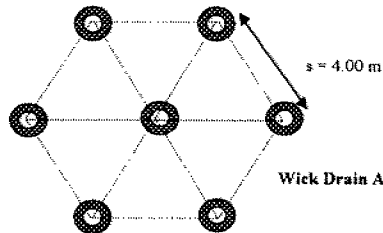
**INPUT PARAMETERS**

D	4.2	m	diameter of dewatered soil cylinder (Triangular Spacing equal to, $s =$	4.00	m)
d	0.065	m	equivalent diameter of band-shaped drain: $2(b+t)/\pi$ ; $n =$	64.6	
$C_H$	5.33E-06	$m^2/s$	consider reducing $c_h$ to account for smear; $C_H/C_v$ is often 2 to 5		
$C_v$	1.07E-06	$m^2/s$	determined by the oedometer test		
$\lambda$	1.07E-06	$m^2/s$	$=k_v/(\gamma_w \cdot m_v)$ , or $\lambda = C_v$ obtained from the oedometer test (Hansbo 1979)		
$d_s$	0.20	m	diameter of the smear zone (typically equal to 1.5 to 3 times d); $s=ds/d =$	3	
$k_c$	5.00E-08	m/s	undisturbed soil permeability		
$k'_c$	1.67E-08	m/s	soil permeability within the smear zone; $k_c/k'_c =$	3.00	
$q_w$	1.00E-05	$m^3/s$	drain discharge capacity; $k_c/q_w =$	5.00E-03	; well resistance cannot be ignored if $k_c/q_w > 3.33E-04$
$l$	10.00	m	length of the drain when open at one end only		
			half length of the drain when open at both ends		

Layer  
 Surcharge (kPa)  
 Drainage Path (m)  
 Settlement due to Primary Consolidation  
 $n$   
 $\alpha$

ML-CL  
 210.00 kPa  
 3.60 m  
 435 mm  
 65 (D/d; should always be >12)  
 0.3814703 (D/d); regression from Figure 3 of the paper

Uv target: 53 %  
 TargetTime (days): 30 days  
 Time for Drainage Path: 30 days



Time Increment for table below =  
 Resultant Maximum Time =

0.17 month  
 10.17 months

% Consolidation	Time required (months)	
	Uv and Uh	Uh only
50	0.50	0.83
75	0.83	1.50
90	1.50	2.50
98	2.50	4.17

TABLE B23

**NEW HANSBO METHOD ACCORDING TO ROBERTSON & CAMPANELLA 1988**  
(combined with Lambe & Whitman's book recommendations)

Hansbo 1979, "Consolidation of Clay by band-shaped prefabricated drains"

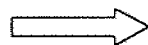
Ground Engineering, Vol.12 No.5, 1979

Formulation according to Equation 2 - No well resistance

Robertson and Campanella, 1988, "Prediction of wick drain performance using piezometer cone data"  
Canadian Geotechnical Journal, 25, 56-61 (1988)

Job Number: 19-1104-4  
Title: Highway 11 - Trout Creek By-Pass  
Case: North Interchange  
Sub-case: Test Hole CPTUN1- East Approach - Sta.10+120 to 10+240 - Most Likely Pc  
Construction Schedule 1: Target: 75% in one month and 100% in one year  
Berm Width: 9m  
D 4.2 m diameter of dewatered soil cylinder (Triangular Spacing)  $s = 4.00$  m  
d 0.065 m equivalent diameter of band-shaped drain:  $2(b+t)/\pi$   
 $C_H$  5.33E-06 m<sup>2</sup>/s consider reducing  $C_h$  to account for smear;  $C_h/C_v$  is often 2 to 5  
 $C_v$  1.07E-06 m<sup>2</sup>/s determined by the oedometer test  
 $\lambda$  5.33E-07 m<sup>2</sup>/s  $=k_h/(\gamma_w \cdot m_v)$ ; for Piezocone  $\gamma = 0.1 \cdot C_h$  (Robertson & Campanella, 1988)  
Layer ML-CL  
Surcharge (kPa) 210.00 kPa  
Drainage Path (m) 3.60 m  
Settlement due to Primary Consolidation 435 mm  
n 65 (D/d; should always be >12)  
 $\alpha$  0.3814703 f(D/d); regression from Figure 3 of the paper)

Time Increment for table below = 0.17 month  
Resultant Maximum Time = 10.17 months



% Consolidation	Time required (months)	
	U <sub>v</sub> and U <sub>h</sub>	U <sub>h</sub> only
50	0.50	1.00
75	1.00	2.17
90	1.83	4.67
98	3.50	more than maximum time entered

**NEW HANSBO METHOD (combined with Lambe & Whitman's book) recommendations**  
**"Consolidation of Clay by Band-Shaped Prefabricated Drains"**  
**Ground Engineering, Vol.12 No.5, 1979**  
**Formulation according to Equation 1 - Including well resistance and smearing**

Job Number: 19-1104-4  
 Title: Highway 11 - Trout Creek By-Pass  
 Case: North Interchange  
 Sub-case: Test Hole CPTUN1- East Approach - Sta.10+120 to 10+240 - Most Likely Pc  
 Construction Schedule 3: Complete construction in 6 months (75% in 1 month between Stages 1 and 2 and 100% in 2.5 months after Stage 2)  
 (allowing 5 weeks for construction of Stage 1 and 1 week for construction of Stage 2)  
 9 m wide berm

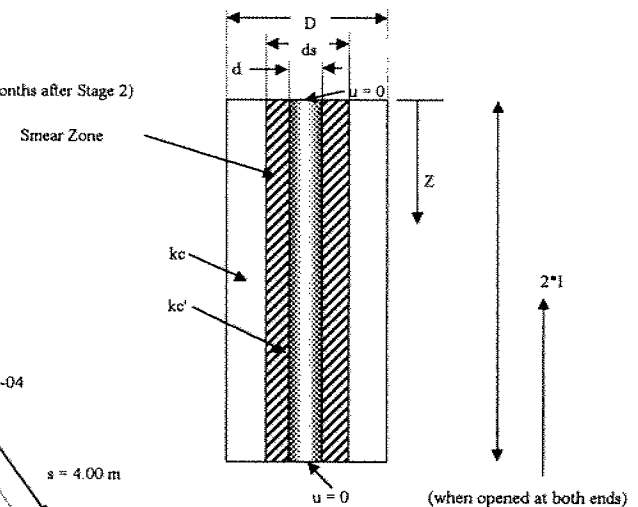
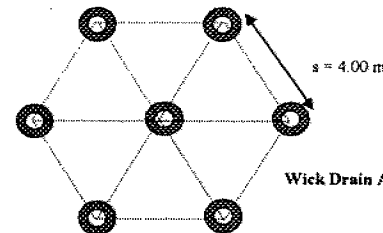
**INPUT PARAMETERS**

D	4.2	m	diameter of dewatered soil cylinder (Triangular Spacing equal to, $s =$	4.00	m)
d	0.065	m	equivalent diameter of band-shaped drain: $2(b+t)/\pi$ ; $n =$	64.6	
$C_H$	5.33E-06	$m^2/s$	consider reducing $c_h$ to account for smear; $C_H/C_v$ is often 2 to 5		
$C_v$	1.07E-06	$m^2/s$	determined by the oedometer test		
$\lambda$	1.07E-06	$m^2/s$	$=k_s/(\gamma_w \cdot m_v)$ ; or $\lambda = C_v$ obtained from the oedometer test (Hansbo 1979)		
$d_s$	0.20	m	diameter of the smear zone (typically equal to 1.5 to 3 times d); $s=ds/d =$	3	
$k_c$	5.00E-08	m/s	undisturbed soil permeability		
$k'_c$	1.67E-08	m/s	soil permeability within the smear zone; $k_c/k'_c =$	3.00	
$q_w$	1.00E-05	$m^3/s$	drain discharge capacity; $k_c/q_w =$	5.00E-03	well resistance cannot be ignored if $k_c/q_w > 3.33E-04$
l	10.00	m	length of the drain when open at one end only		
			half length of the drain when open at both ends		

Layer  
 Surcharge (kPa)  
 Drainage Path (m)  
 Settlement due to Primary Consolidation  
 n  
 $\alpha$

ML-CL  
 210.00 kPa  
 3.60 m  
 435 mm  
 65 (D/d; should always be >12)  
 0.3814703 R(D/d); regression from Figure 3 of the paper

Uv target: 53 %  
 TargetTime (days): 30 days  
 Time for Drainage Path: 30 days



Time Increment for table below =  
 Resultant Maximum Time =

0.17 month  
 10.17 months

% Consolidation	Time required (months)	
	Uv and Uh	Uh only
50	0.50	0.83
75	0.83	1.50
90	1.50	2.50
98	2.50	4.17

TABLE B25

**NEW HANSBO METHOD ACCORDING TO ROBERTSON & CAMPANELLA 1988**  
(combined with Lambe & Whitman's book recommendations)

Hansbo 1979, "Consolidation of Clay by band-shaped prefabricated drains"  
Ground Engineering, Vol.12 No.5, 1979  
Formulation according to Equation 2 - No well resistance

Robertson and Campanella, 1988, "Prediction of wick drain performance using piezometer cone data  
Canadian Geotechnical Journal, 25, 56-61 (1988)

Job Number: 19-1104-4  
Title: Highway 11 - Trout Creek By-Pass  
Case: North Interchange  
Sub-case: Test Hole CPTUN1- East Approach - Sta.10+120 to 10+240 - Most Likely Pc  
Construction Schedule 3: Complete construction in 6 months (75% in 1 month between Stages 1 and 2 and 100% in 2.5 months after Stage 2)  
(allowing 5 weeks for construction of Stage 1 and 1 week for construction of Stage 2)  
9 m wide berm  
D 4.2 m diameter of dewatered soil cylinder (Triangular Spacing)  $s = 4.00$  m  
d 0.065 m equivalent diameter of band-shaped drain:  $2(b+t)/\pi$   
 $C_R$  5.33E-06 m<sup>2</sup>/s consider reducing  $C_h$  to account for smear;  $C_h/C_v$  is often 2 to 5  
 $C_v$  1.07E-06 m<sup>2</sup>/s determined by the oedometer test  
 $\lambda$  5.33E-07 m<sup>2</sup>/s  $=k_w/(\gamma_w \cdot m_v)$ ; for Piezocone  $\gamma = 0.1 \cdot C_h$  (Robertson & Campanella, 1988)  
Layer ML-CL  
Surcharge (kPa) 210.00 kPa  
Drainage Path (m) 3.60 m  
Settlement due to Primary Consolidation 435 mm  
 $n$  65 ( $D/d$ ; should always be  $>12$ )  
 $\alpha$  0.3814703  $f(D/d)$ ; regression from Figure 3 of the paper)

Time Increment for table below = 0.17 month  
Resultant Maximum Time = 10.17 months

% Consolidation	Time required (months)	
	U <sub>v</sub> and U <sub>h</sub>	U <sub>h</sub> only
50	0.50	1.00
75	1.00	2.17
90	1.83	4.67
98	3.50	more than maximum time entered

**NEW HANSBO METHOD (combined with Lambe & Whitman's book) recommendations**  
**"Consolidation of Clay by Band-Shaped Prefabricated Drains"**  
**Ground Engineering, Vol.12 No.5, 1979**  
**Formulation according to Equation 1 - Including well resistance and smearing**

Job Number: 19-1104-4  
 Title: Highway 11 - Trout Creek By-Pass  
 Case: North Interchange  
 Sub-case: Test Hole CPTUN4- East Approach - Sta.10+240 to Sta. 10+310 - Most Likely Pc  
 Construction Schedule 3: Target: 100% in 6 months (3 weeks for construction and 21 weeks for 100% consolidation)

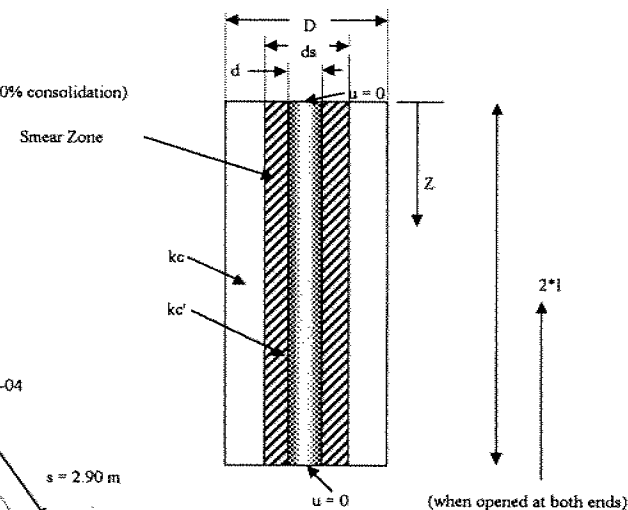
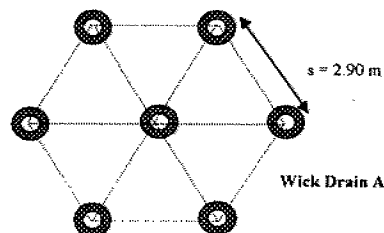
**INPUT PARAMETERS**

D	3.045	m	diameter of dewatered soil cylinder (Triangular Spacing equal to, $s =$	2.90	m)
d	0.065	m	equivalent diameter of band-shaped drain: $2(b+t)/\pi$ ; $n =$	46.8	
$C_H$	3.17E-06	$m^2/s$	consider reducing $c_h$ to account for smear; $C_H/C_v$ is often 2 to 5		
$C_v$	6.34E-07	$m^2/s$	determined by the oedometer test		
$\lambda$	6.34E-07	$m^2/s$	$=k_w/(\gamma_w \cdot m_v)$ ; or $\lambda = C_v$ obtained from the oedometer test (Hansbo 1979)		
$d_s$	0.20	m	diameter of the smear zone (typically equal to 1.5 to 3 times d); $s=ds/d =$	3	
$k_c$	1.00E-09	m/s	undisturbed soil permeability		
$k'_c$	3.33E-10	m/s	soil permeability within the smear zone, $k_c/k'_c =$	3.00	
$q_w$	1.00E-05	$m^3/s$	drain discharge capacity; $k_c/q_w =$	1.00E-04	well resistance cannot be ignored if $k_c/q_w > 3.33E-04$
l	17.00	m	length of the drain when open at one end only		
			half length of the drain when open at both ends		

Layer  
 Surcharge (kPa)  
 Drainage Path (m)  
 Settlement due to Primary Consolidation  
 n  
 $\alpha$

ML-CL  
 150.00 kPa  
 4.60 m  
 465 mm  
 47 (D/d; should always be >12)  
 0.3649627 (D/d); regression from Figure 3 of the paper

Uv target: 45 %  
 Target Time (days): 60 days  
 Time for Drainage Path: 60 days



Time Increment for table below =  
 Resultant Maximum Time =

0.25 month  
 15.25 months

% Consolidation	Time required (months)	
	Uv and Uh	Uh only
50	0.50	0.75
75	1.00	1.25
90	1.50	1.75
98	2.50	3.00

TABLE B27



NEW HANSBO METHOD ACCORDING TO ROBERTSON & CAMPANELLA 1988  
(combined with Lambe & Whitman's book recommendations)

Hansbo 1979, "Consolidation of Clay by band-shaped prefabricated drains"

Ground Engineering, Vol. 12 No.5, 1979

Formulation according to Equation 2 - No well resistance

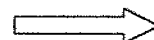
Robertson and Campanella, 1988, "Prediction of wick drain performance using piezometer cone data  
Canadian Geotechnical Journal, 25, 56-61 (1988)

Job Number: 19-1104-4  
Title: Highway 11 - Trout Creek By-Pass  
Case: North Interchange  
Sub-case: Test Hole CPTUN4- East Approach - Sta. 10+240 to Sta. 10+310 - Most Likely Pc  
Construction Schedule 3: Target: 100% in 6 months (3 weeks for construction and 21 weeks for 100% consolidation)

INPUT PARAMETERS

D	3.045	m	diameter of dewatered soil cylinder (Triangular Spacing)	$s = 2.90$ m
d	0.065	m	equivalent diameter of band-shaped drain: $2(b+t)/\pi$	
$C_H$	3.17E-06	m <sup>2</sup> /s	consider reducing $C_h$ to account for smear; $C_h/C_v$ is often 2 to 5	
$C_v$	6.34E-07	m <sup>2</sup> /s	determined by the oedometer test	
$\lambda$	3.17E-07	m <sup>2</sup> /s	$=k_y/(\gamma_w \cdot m_v)$ ; for Piezocone $\gamma = 0.1 \cdot C_h$ (Robertson & Campanella, 1988)	
Layer	ML-CL			
Surcharge (kPa)	150.00	kPa		
Drainage Path (m)	4.60	m		
Settlement due to Primary Consolidation	465	mm		
n	47	(D/d; should always be >12)		
$\alpha$	0.3649627	f(D/d); regression from Figure 3 of the paper		

Time Increment for table below = 0.25 month  
Resultant Maximum Time = 15.25 months



% Consolidation	Time required (months)	
	U <sub>v</sub> and U <sub>h</sub>	U <sub>h</sub> only
50	0.50	1.00
75	1.25	2.00
90	2.50	4.00
98	5.25	11.25

**NEW HANSBO METHOD (combined with Lambe & Whitman's book) recommendations**  
**"Consolidation of Clay by Band-Shaped Prefabricated Drains"**  
**Ground Engineering, Vol.12 No.5, 1979**  
**Formulation according to Equation 1 - Including well resistance and smearing**

Job Number: 19-1104-4  
 Title: Highway 11 - Trout Creek By-Pass  
 Case: North Interchange  
 Sub-case: Test Hole CPTUN2- EW-N Ramps 13+160 to 13+270 - Most Likely Pc  
 Construction Schedule 3: Target: 100% in 6 months (3 weeks for construction and 21 weeks for 100% consolidation)

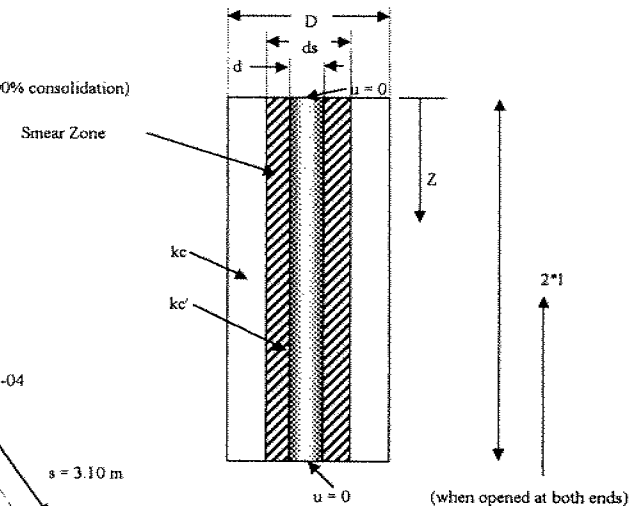
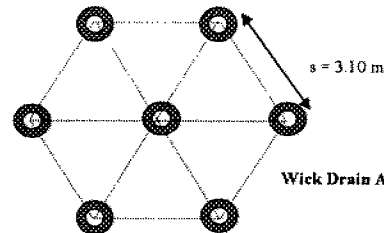
**INPUT PARAMETERS**

D	3.255	m	diameter of dewatered soil cylinder (Triangular Spacing equal to, $s =$	3.10	m)
d	0.065	m	equivalent diameter of band-shaped drain: $2(b+t)/\pi$ ; $n =$	50	1
$C_H$	$3.01E-06$	$m^2/s$	consider reducing $c_h$ to account for smear; $C_H/C_v$ is often 2 to 5		
$C_v$	$6.02E-07$	$m^2/s$	determined by the oedometer test		
$\lambda$	$6.02E-07$	$m^2/s$	$=k_w/(\gamma_w \cdot m_v)$ ; or $\lambda = C_v$ obtained from the oedometer test (Hansbo 1979)		
$d_s$	0.20	m	diameter of the smear zone (typically equal to 1.5 to 3 times d); $s=ds/d =$	3	
$k_c$	$5.00E-08$	m/s	undisturbed soil permeability		
$k'_c$	$1.67E-08$	m/s	soil permeability within the smear zone; $k_c/k'_c =$	3.00	
$q_w$	$1.00E-05$	$m^3/s$	drain discharge capacity; $k_c/q_w =$	$5.00E-03$	; well resistance cannot be ignored if $k_c/q_w > 3.33E-04$
l	12.00	m	length of the drain when open at one end only		
			half length of the drain when open at both ends		

Layer  
 Surcharge (kPa)  
 Drainage Path (m)  
 Settlement due to Primary Consolidation  
 n  
 $\alpha$

ML-CL  
 180.00 kPa  
 4.10 m  
 284 mm  
 50 (D/d; should always be  $>12$ )  
 0.3693519  $f(D/d)$ ; regression from Figure 3 of the paper

Uv target: 50 %  
 TargetTime (days): 60 days  
 Time for Drainage Path: 60 days



Time Increment for table below = 0.25 month  
 Resultant Maximum Time = 15.25 months

% Consolidation	Time required (months)	
	Uv and Uh	Uh only
50	0.50	1.00
75	1.25	1.75
90	2.00	2.75
98	3.50	4.75

TABLE B29

**NEW HANSBO METHOD ACCORDING TO ROBERTSON & CAMPANELLA 1988**  
(combined with Lambe & Whitman's book recommendations)

Hansbo 1979, "Consolidation of Clay by band-shaped prefabricated drains"

Ground Engineering, Vol.12 No.5, 1979

Formulation according to Equation 2 - No well resistance

Robertson and Campanella, 1988, "Prediction of wick drain performance using piezometer cone data"  
Canadian Geotechnical Journal, 25, 56-61 (1988)

Job Number: 19-1104-4  
Title: Highway 11 - Trout Creek By-Pass  
Case: North Interchange  
Sub-case: Test Hole CPTUN2- EW-N Ramps 13+160 to 13+270 - Most Likely Pc  
Construction Schedule 3: Target: 100% in 6 months (3 weeks for construction and 21 weeks for 100% consolidation)

**INPUT PARAMETERS**

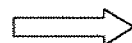
<b>D</b>	3.255	<b>m</b>	diameter of dewatered soil cylinder (Triangular Spacing)	$s = 3.10 \text{ m}$
<b>d</b>	0.065	<b>m</b>	equivalent diameter of band-shaped drain: $2(b+t)/\pi$	
<b>C<sub>H</sub></b>	3.01E-06	<b>m<sup>2</sup>/s</b>	consider reducing C <sub>h</sub> to account for smear; C <sub>h</sub> /C <sub>v</sub> is often 2 to 5	
<b>C<sub>v</sub></b>	6.02E-07	<b>m<sup>2</sup>/s</b>	determined by the oedometer test	
<b>λ</b>	3.01E-07	<b>m<sup>2</sup>/s</b>	$=k_h/(\gamma_w \cdot m_v)$ ; for Piezocone $\gamma = 0.1 \cdot C_h$ (Robertson & Campanella, 1988)	
<b>Layer</b>	ML-CL			
<b>Surcharge (kPa)</b>	180.00	<b>kPa</b>		
<b>Drainage Path (m)</b>	4.10	<b>m</b>		
<b>Settlement due to Primary Consolidation</b>	284	<b>mm</b>		
<b>n</b>	50		(D/d; should always be >12)	
<b>α</b>	0.3693519		f(D/d); regression from Figure 3 of the paper)	

Time Increment for table below =

0.25 month

Resultant Maximum Time =

15.25 months



% Consolidation	Time required (months)	
	U <sub>v</sub> and U <sub>h</sub>	U <sub>h</sub> only
50	0.50	1.00
75	1.25	2.25
90	2.50	4.75
98	5.25	12.75

**HIGHWAY 11 - TROUT CREEK BY-PASS - NORTH INTERCHANGE**  
**DESIGN RECOMMENDATIONS FOR DIFFERENT CONSTRUCTION SCHEDULES**  
**MAXIMUM BERM WIDTH = 9 m**

**Schedule 1:** 2 months: Site preparation (includes installation of wick drains)  
 3 months: Embankment Construction  
 12 months: Waiting period for stabilization of settlements

**Schedule 2:** Embankment construction and stabilization of settlements in 12 months

**Schedule 3:** Embankment construction and stabilization of settlements in 6 months

**Location:** East Abutment - McCarthy St. (St. 10+040 to 10+120) - Height = 9 to 10.5 m plus surcharge  
**Surcharge:** 1.5 m  
**Berm Height:** 6 m below the pavement final design elevation  
**Berm Width:** 9 m

Construction Sequence	Description	Elapsed Time from Beginning of Construction			Monitoring Requirements: Maximum EPP before this stage
		Schedule 1	Schedule 2	Schedule 3	
		Wick Spacing = 3.5 m	Wick Spacing = N/A	Wick Spacing = 2.5 m	
Stage 1	H=0 to 11.5 m	0 to 6 weeks	0 to 6 weeks	0 to 6 weeks	No EPP requirement
Stage 2	Wait - No construction	6 to 10 weeks	6 to 19 weeks	6 to 8 weeks	-
Stage 3	Complete Embankment to top of surcharge	10 to 11 weeks	19 to 20 weeks	8 to 9 weeks	57 kPa
Stage 4	Wait - No construction	11 to 28 weeks	20 to 52 weeks	9 to 19 weeks	-
Stage 5	Trim to Final Elevation	Start after 28 weeks	Start after 52 weeks	Start after 19 weeks	0 kPa

**Location:** McCarthy St. (St. 10+120 - 10+240); W-N Ramp (St. 13+091- 13+160); S-EW Ramp (13+630 - 13+540) -Height=6 m to 9 m plus surcharge  
**Surcharge:** 1.5 m  
**Berm Width:** 9 m  
**Berm Height:** 6 m below the pavement final design elevation

Construction Sequence	Description	Elapsed Time from Beginning of Construction			Monitoring Requirements: Maximum EPP before this stage
		Schedule 1	Schedule 2	Schedule 3	
		Wick Spacing = 4.0 m	Wick Spacing = N/A	Wick Spacing = 4.0 m	
Stage 1	H=0 to 8.8 m	0 to 5 weeks	0 to 5 weeks	0 to 5 weeks	No EPP requirement
Stage 2	Wait - No construction	5 to 9 weeks	5 to 20 weeks	5 to 9 weeks	-
Stage 3	Complete Embankment to top of surcharge	9 to 10 weeks	20 to 21 weeks	9 to 10 weeks	44 kPa
Stage 4	Wait - No construction	10 to 24 weeks	21 to 52 weeks	10 to 24 weeks	-
Stage 5	Trim to Final Elevation	Start after 24 weeks	Start after 52 weeks	Start after 24 weeks	0 kPa (*)

**Location:** McCarthy St.(St. 10+240 - 10+310); W-N Ramp (Sta. 13+160 - 13+270); S-EW Ramp (St. 13+540 - 13+450) -Height=4.5 m to 6 m plus surcharge  
**Surcharge:** 1.5 m  
**Berm Height:** None

Construction Sequence	Description	Elapsed Time from Beginning of Construction			Monitoring Requirements: Maximum EPP before this stage
		Schedule 1	Schedule 2	Schedule 3	
		Wick Spacing = N/A	Wick Spacing = N/A	Wick Spacing = 3.0 m	
Stage 1	Complete Embankment to top of surcharge	0 to 3 weeks	0 to 3 weeks	0 to 3 weeks	No EPP requirement
Stage 2	Wait - No construction	3 to 52 weeks	3 to 52 weeks	3 to 23 weeks	-
Stage 3	Trim to Final Elevation	Start after 52 weeks	Start after 52 weeks	Start after 23 weeks	0 kPa (*)

**Location:** McCarthy St. -Sta.> 10+310; W-N Ramp (Sta. > 13+270); S-EW Ramp (St.>13+450); Hwy 11 - Height<4.5 m plus surcharge  
**Surcharge:** See table below  
**Berm Height:** None

Construction Sequence	Description	Elapsed Time from Beginning of Construction			Monitoring Requirements: Maximum EPP before this stage
		Schedule 1	Schedule 2	Schedule 3	
		Wick Spacing = N/A	Wick Spacing = N/A	Wick Spacing = N/A	
		Surcharge = 1.5 m	Surcharge = 1.5 m	Surcharge = 2.5 m	
Stage 1	Complete Embankment to top of surcharge	0 to 2 weeks	0 to 2 weeks	0 to 2 weeks	No EPP requirement
Stage 2	Wait - No construction	2 to 52 weeks	2 to 52 weeks	3 to 24 weeks	-
Stage 3	Trim to Final Elevation	Start after 52 weeks	Start after 52 weeks	Start after 24 weeks	0 kPa (*)

Note: (\*) Trimming to final elevation can only be carried out after both EPP and settlements due to primary consolidation have stabilized within 2% of the value assessed according to the hyperbolic



**APPENDIX C**

**ConeTec Report**

**(North Interchange Test Holes Only)**

**PRESENTATION OF CONE PENETRATION TEST DATA,  
Trout Creek Interchanges**

**Trout Creek, Ontario**

**Prepared for:**

**Thurber Engineering Ltd.  
Etobicoke, Ontario**

**Prepared by:**

**CONETEC INVESTIGATIONS LTD.**

**March 31, 1999**

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- 1.0 INTRODUCTION
- 2.0 FIELD EQUIPMENT AND PROCEDURES
- 3.0 CONE PENETRATION TEST DATA
  - 3.1 CPT Data
  - 3.2 Pore Pressure Dissipation Data

## APPENDICES

- Appendix A CPT Plots
- Appendix B CPT Interpretations
- Appendix C Summary of Dissipations and Pore Pressure Plots



## 1.0 INTRODUCTION

This report presents the results of a cone penetration testing (CPT) program carried out at the location of the South and North Trout Creek Interchanges, near Trout Creek, Ontario. A total of 10 CPT's with pore pressure dissipation tests were performed for this investigation, with 5 CPTs at each of the south and north interchange sites between the period of March 25<sup>th</sup> and March 26<sup>th</sup>, 1999.

## 2.0 FIELD EQUIPMENT AND PROCEDURES

### 2.1 CPT Procedures

The cone penetration tests (CPT's) were carried out by **ConeTec Investigations Ltd.** of Vancouver, B.C. using an integrated electronic cone system. A 20 ton compression type cone was used for all of the soundings. The 20 ton cone has a tip area of 15 sq cm and friction sleeve area of 225 sq cm. A piezometer element 6 mm thick is located immediately behind the cone tip. The compression cones are designed with an equal end area friction sleeve and a tip end area ratio of 0.85. The cone system used during the program recorded the following parameters at 2.5 cm depth increments:

- Tip Resistance (Qc) in bars
- Sleeve Friction (Fs) in bars
- Dynamic Pore Pressure (Ut) in metres of water

The above parameters were printed simultaneously on a printer and stored on digital media for future analysis and reference.

The porous plastic pore pressure element was located directly behind the cone tip. Each of the elements were saturated in glycerin under vacuum pressure prior to penetration. Pore pressure dissipations were recorded at 5 second intervals during all pauses in the penetration.

A complete set of baseline readings were taken prior to and after each sounding to determine if any zero load offsets had occurred due a temperature change of the probe. Establishing the presence of temperature shifts and load offsets enables the operator to make corrections to the cone data if necessary. These corrections can be important, especially where the load conditions are relatively low, and generally are the single largest source of error with respect to the accuracy of cone data. Since the probes are temperature compensated, load shifts due to changes in probe

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temperature are only a problem when there are extreme temperature changes from before the test is started and while the probe is in situ. For the testing done on this project keeping the cone within an operating temperature range that did not produce load offsets was not a problem. The cone was pushed using track mounted CME 75 provided by All Terrain Drilling. All CPTs where pushed to refusal.

The following is a list of the CPT names, test depths and water tables. The bracketed values in the water table column are from dissipation tests at refusal.

CPT File	CPT Test Name	Depth (m)	Water Table (m)
141cps1	CPT-S1	13.05	0.0 (-0.3)
141cps2	CPT-S2	9.75	0.0 (-0.05)
141cps3	CPT-S3	16.925	0.0 (-0.2)
141cps4	CPT-S4	15.275	0.0
141cps5	CPT-S5	22.075	0.0 (-0.3)
141cpn1	CPT-N1	12.10	0.0 (-0.3)
141cpn2	CPT-N2	17.325	0.0 (0.1)
141cpn3	CPT-N3	15.125	0.0 (0.0)
141cpn4	CPT-N4	19.925 (20.6) *	0.0 (-0.4)
141cpn5	CPT-N5	11.85 (12.55) *	0.0 (-0.45)

\* Pore pressure data at depths below recorded CPT Data (CPT data not recorded)

### 3.0 CONE PENETRATION TEST DATA

#### 3.1 CPT Data

The cone penetration test data is presented in graphical form in Appendix A following the text of this report. For each test there are two sets of plots. The first plot consists of Tip Resistance (Qt) in bars, Sleeve Friction (Fs) in bars, Pore Pressure (U) in metres of water, and Friction Ratio (Rf) plotted versus depth. The second plot consists of Qt, SPT N60, SPT (N1)60, and Undrained Strength (Su) in kPa. The CPT data is also stored as ASCII text on the accompanying data disk. Penetration data is referenced to existing ground. Stratigraphic interpretations appears on the right side of both plot

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sets. The stratigraphic interpretation is based on a chart relating cone bearing  $Q_c$ , and sleeve friction  $F_s$  developed by Robertson et al, 1986 as shown in Figure 1. Detailed interpretations of the CPT data are included in Appendix B. A description of the interpretation methods is included at the end of Appendix B.

### 3.2 Pore Pressure Dissipation Test Results

Pore pressure dissipations were recorded during selected pauses in penetration for all CPTs tests. The pore pressure data was recorded at 5 second intervals. The pore pressure dissipation data for each CPT is included on the data disk. Pore pressure dissipation data in fine grained soils provides a good indication of the consolidation characteristics. Data from pore pressure dissipation tests in tabular format is presented in Appendix C. The coefficient of consolidation in the horizontal direction,  $c_h$ , was calculated using the equation following equation.

$$c_h = \frac{T^* r^2 \sqrt{I_r}}{t}$$

where:

$T^*$	-	time constant = 0.245 for 50% dissipation
$r$	-	radius of the cone
$I_r$	-	Rigidity Index = $G/S_u$
$t$	-	time for dissipation

For all the dissipations the time for 50 percent dissipation was used to calculate  $c_h$ . A value of 200 for the rigidity index was used in all calculations. The resulting values of  $c_h$  ranged from 1.8 cm<sup>2</sup>/min to 110 cm<sup>2</sup>/min, with most values falling between 2 cm<sup>2</sup>/min to 9 cm<sup>2</sup>/min. Pore pressure dissipation tests in the highly permeable sand layer below the clayey silt reached equilibrium almost instantaneously. The equilibrium values of pore pressure indicate the water table was at the surface to about 0.3 above the surface.

**Thurber Engineering**

We trust that the information presented in this report is sufficient for your purposes. If you have any questions regarding the contents of this report, please do not hesitate to contact our office.

Yours truly,

**ConeTec Investigations Ltd.**

Per: 

**Ilmar Weemees, P.Eng.**

ref: 99-141.wpd

**ConeTec Investigations Ltd.**

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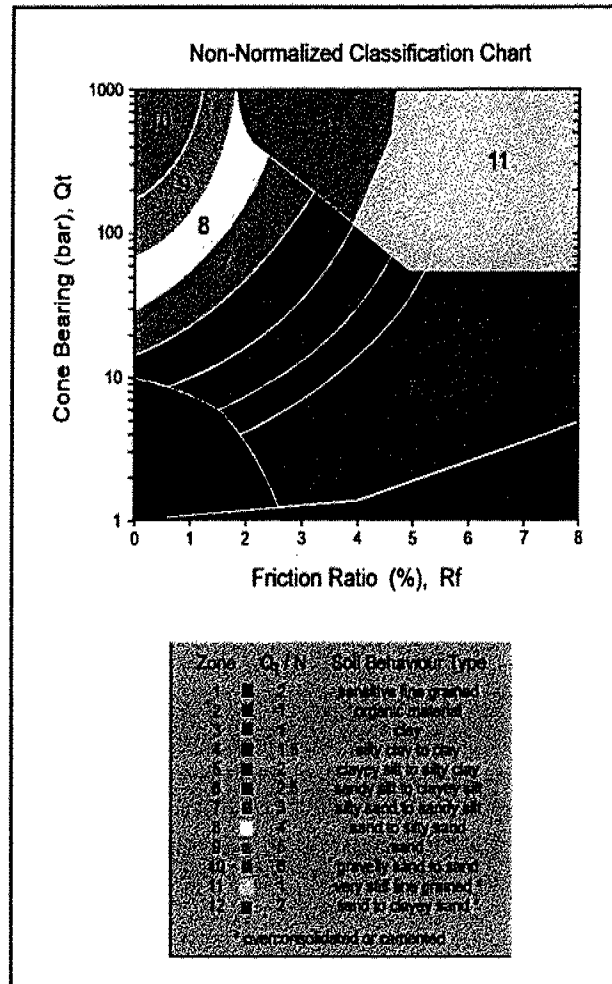


Figure 1. Soil Behaviour Type Classification Chart

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## **APPENDIX A**

### **CPT Plots**

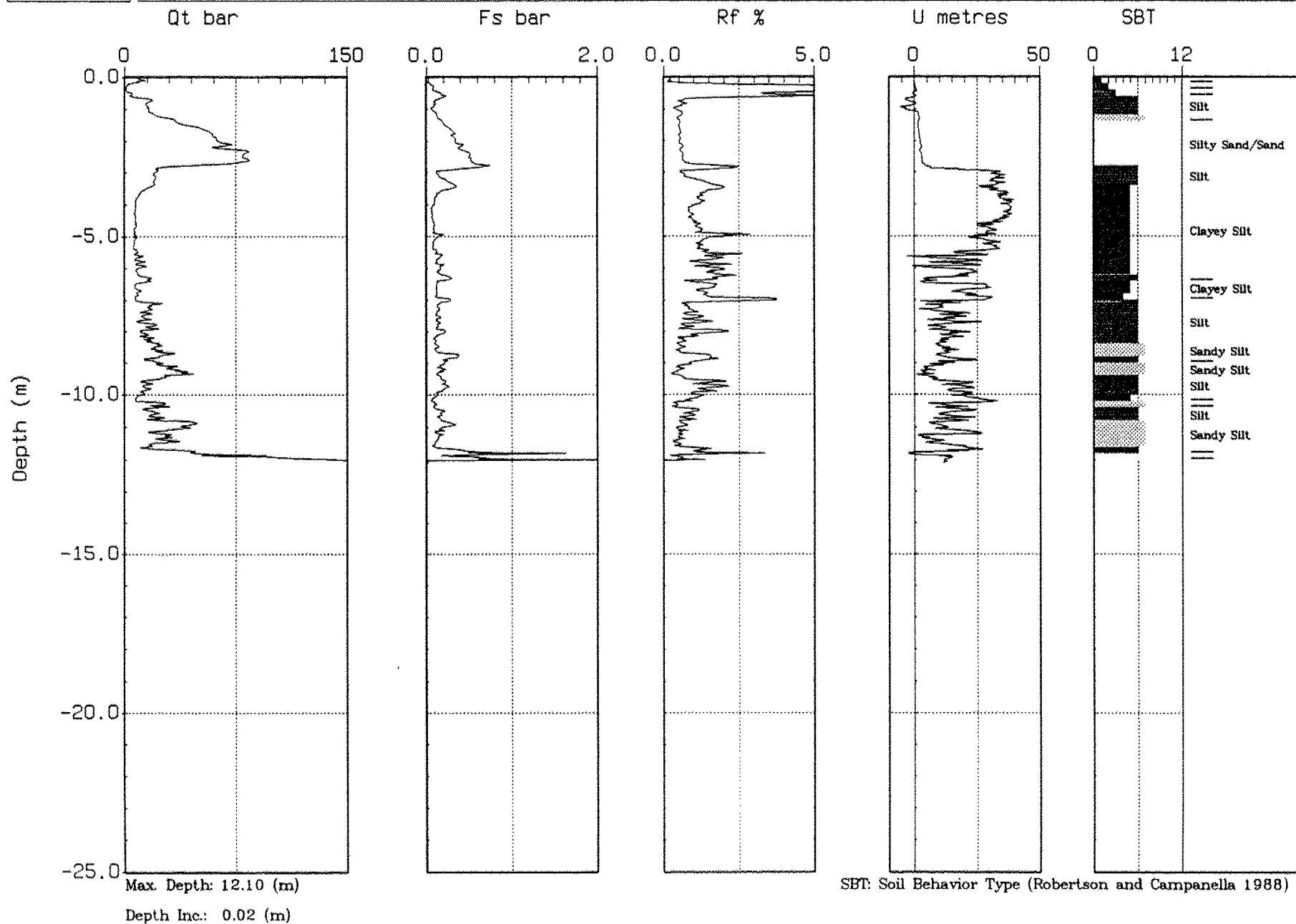
**ConeTec Investigations Ltd.**



Thurber Engineering

Site: 99-141 CPT-N1  
Location: NINTERCHANGE

Cone: 20 TON A 058  
Date: 03/26/99 08:19

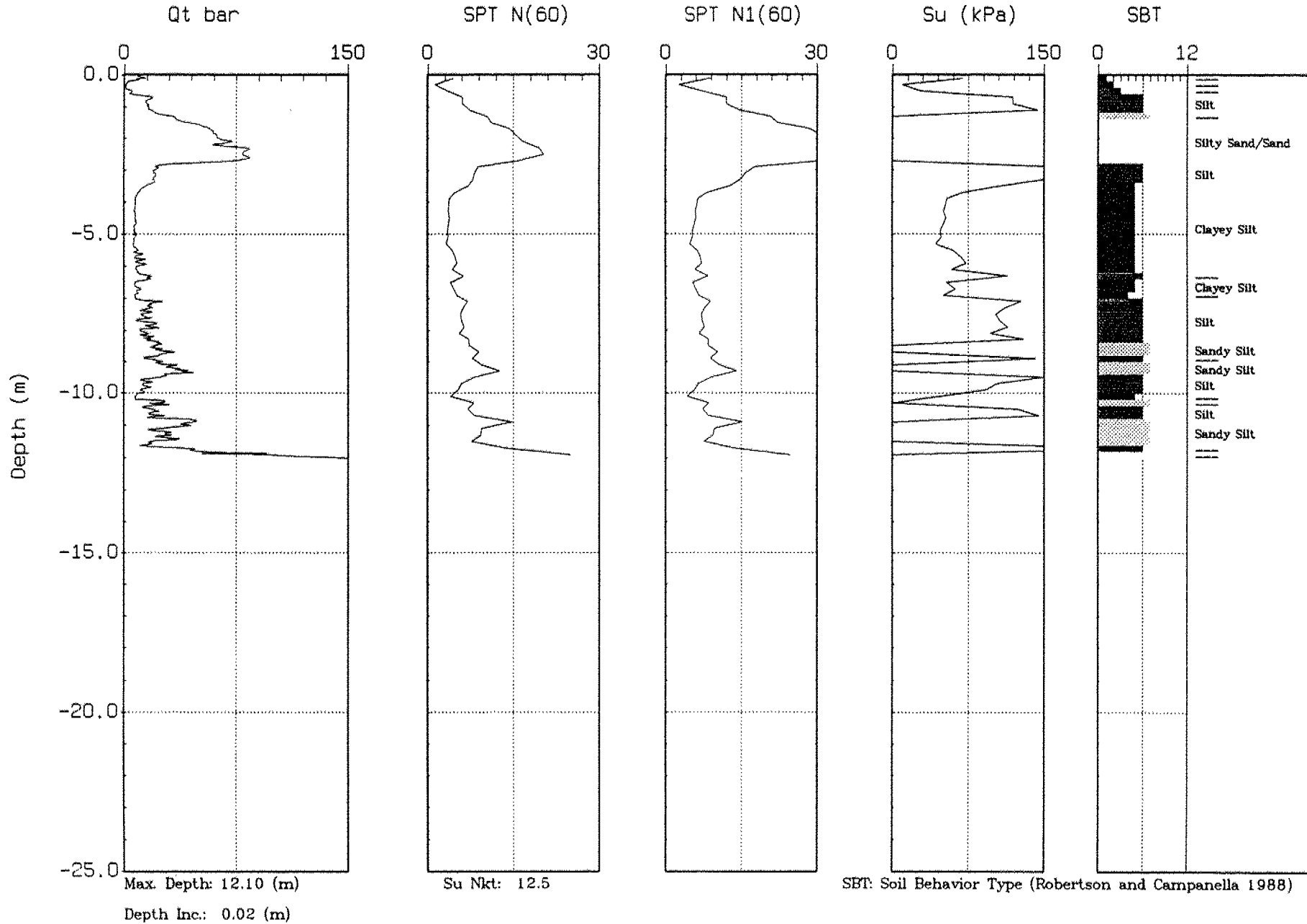




Thurber Engineering

Site: 99-141 CPT-N1  
Location: NINTERCHANGE

Cone: 20 TON A 058  
Date: 03/26/99 08:19



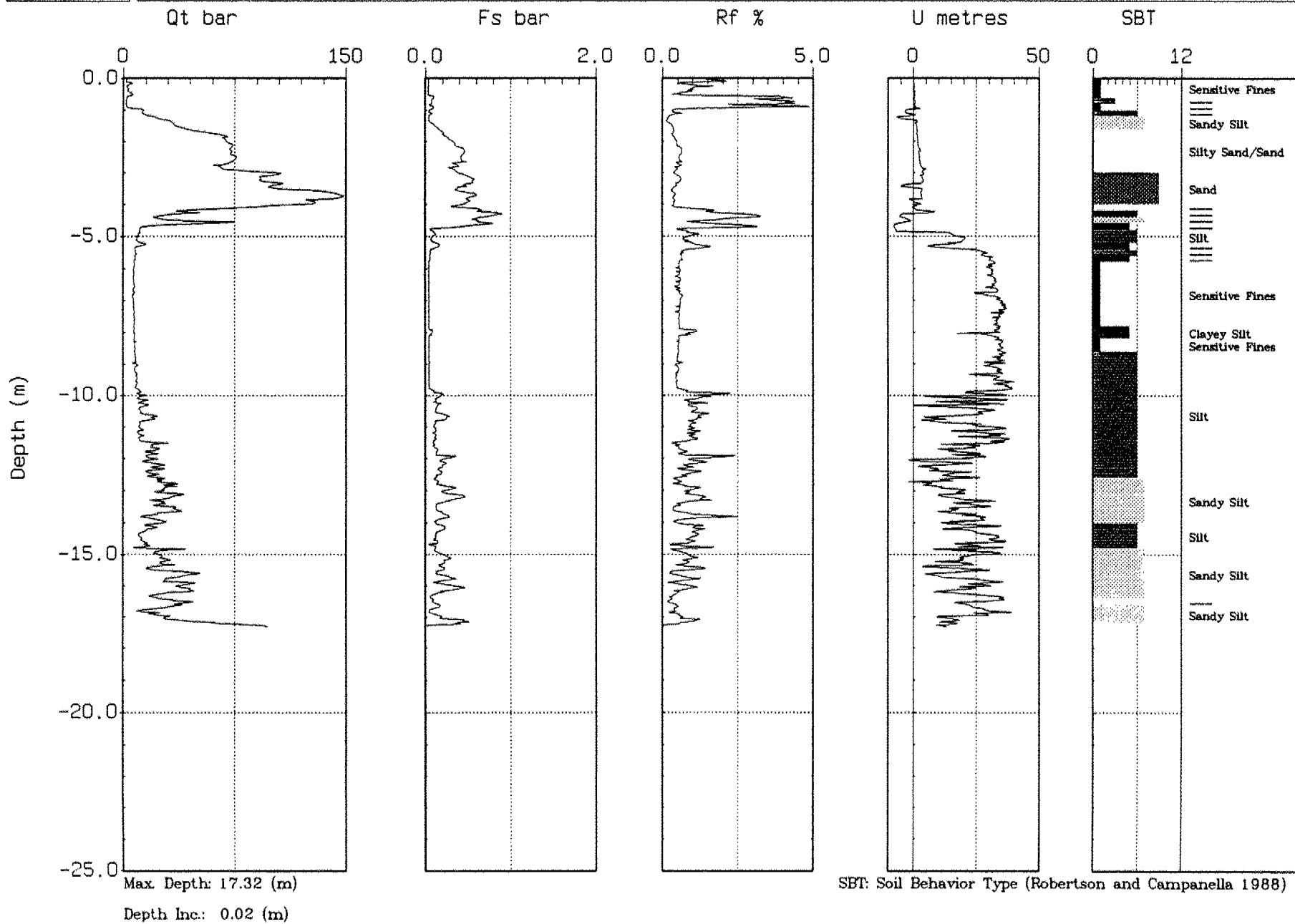




Thurber Engineering

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Location: INTERCHANGE

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Date: 03/26/99 09:54

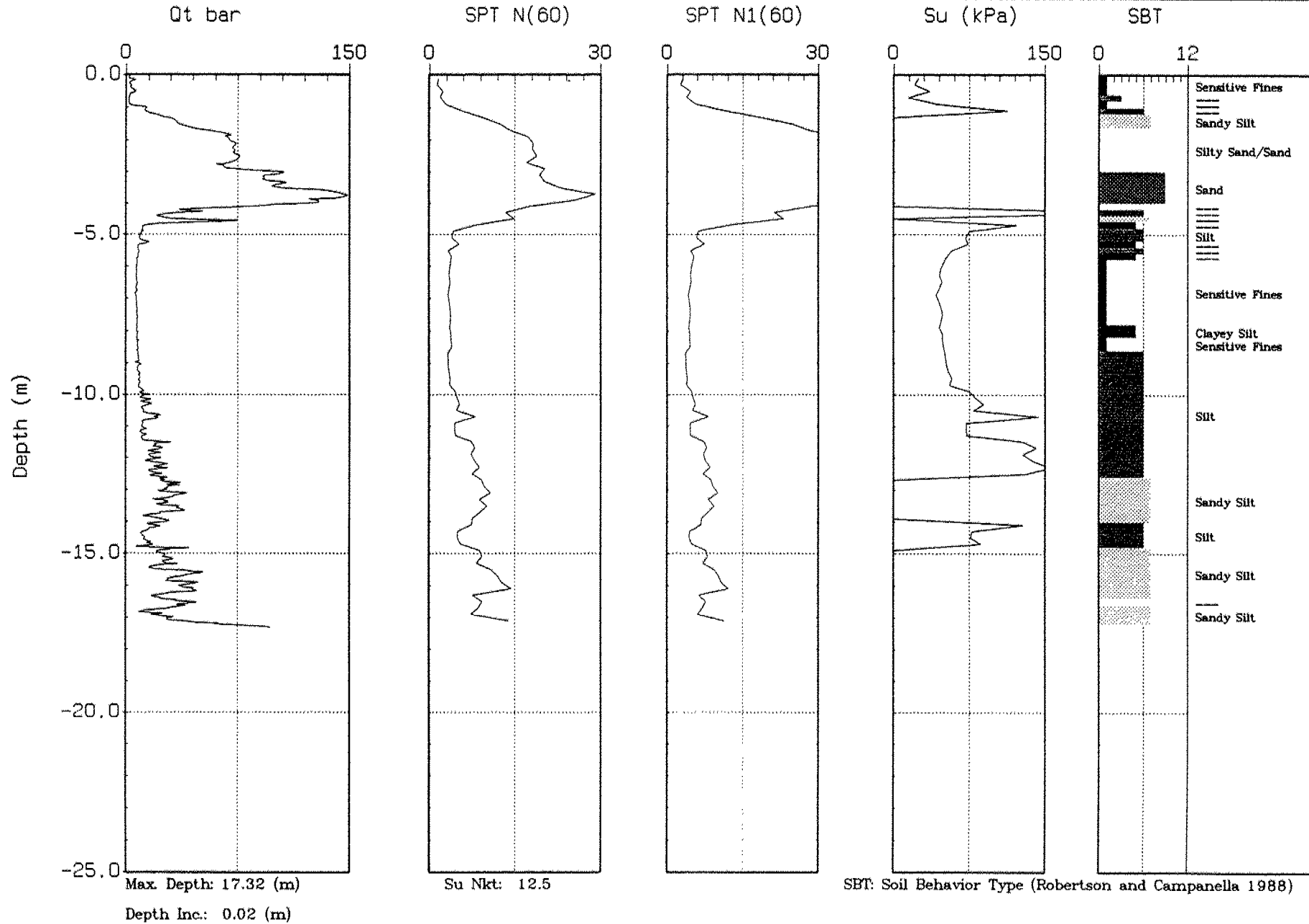




Thurber Engineering

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Location: N/INTERCHANGE

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Date: 03/26/99 09:54

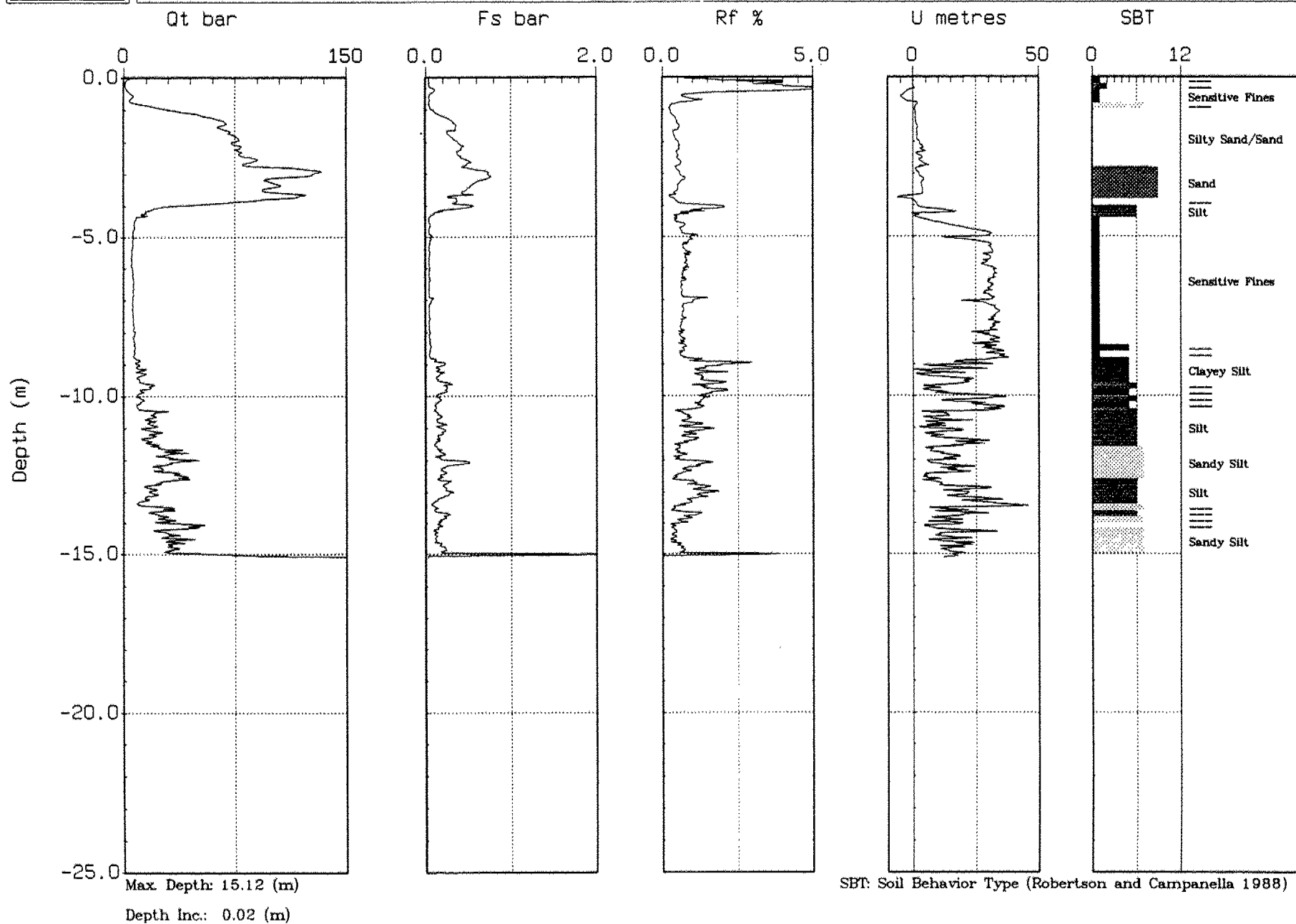




Thurber Engineering

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Location: NINTERCHANGE

Cone: 20 TON A 058  
Date: 03/26/99 11:48

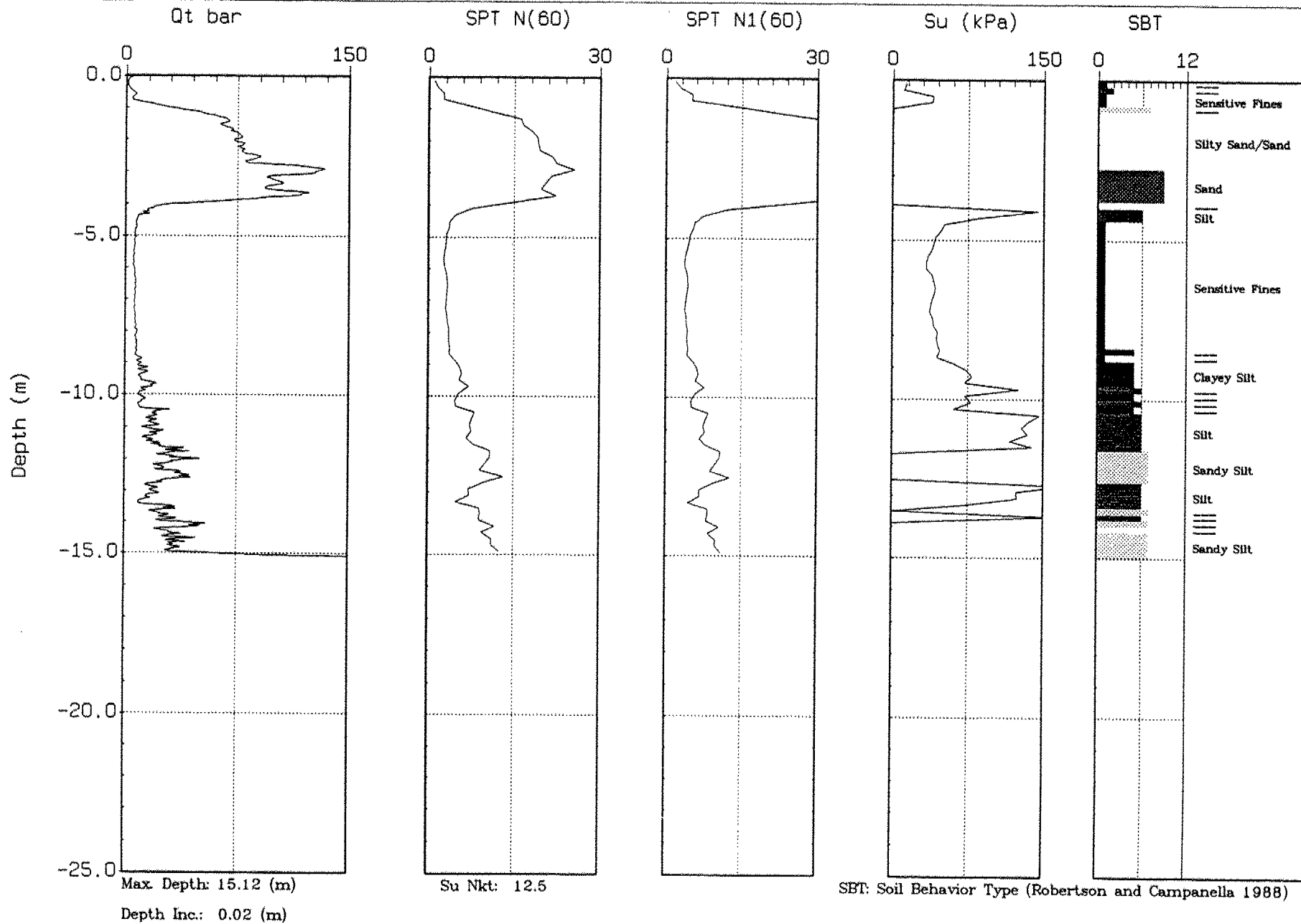




Thurber Engineering

Site: 99-141 CPT-N3  
Location: N.INTERCHANGE

Cone: 20 TON A 058  
Date: 03/26/99 11:48

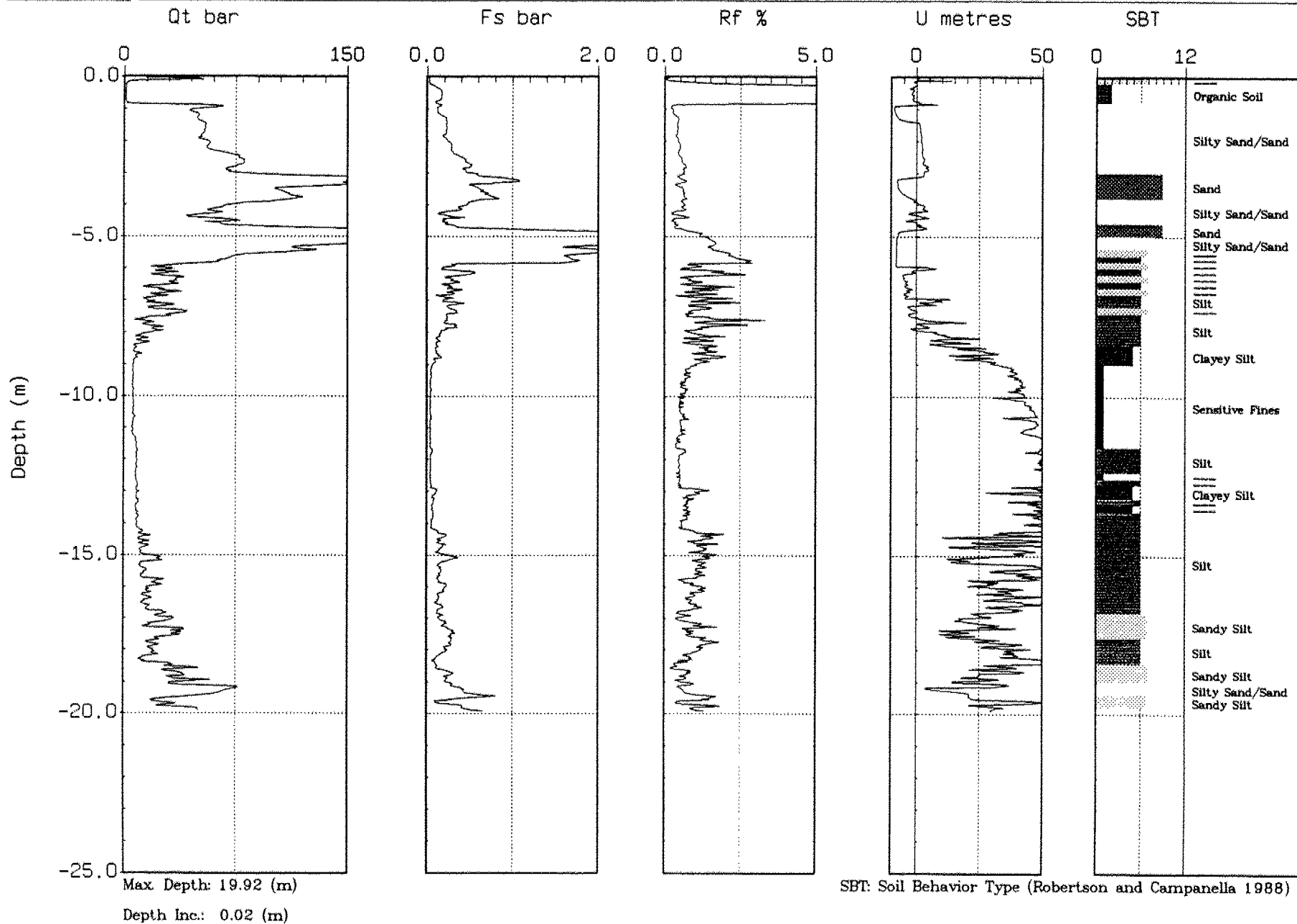




Thurber Engineering

Site: 99-141 CPT-N4  
Location: N.INTERCHANGE

Cone: 20 TON A 058  
Date: 03/26/99 14:44

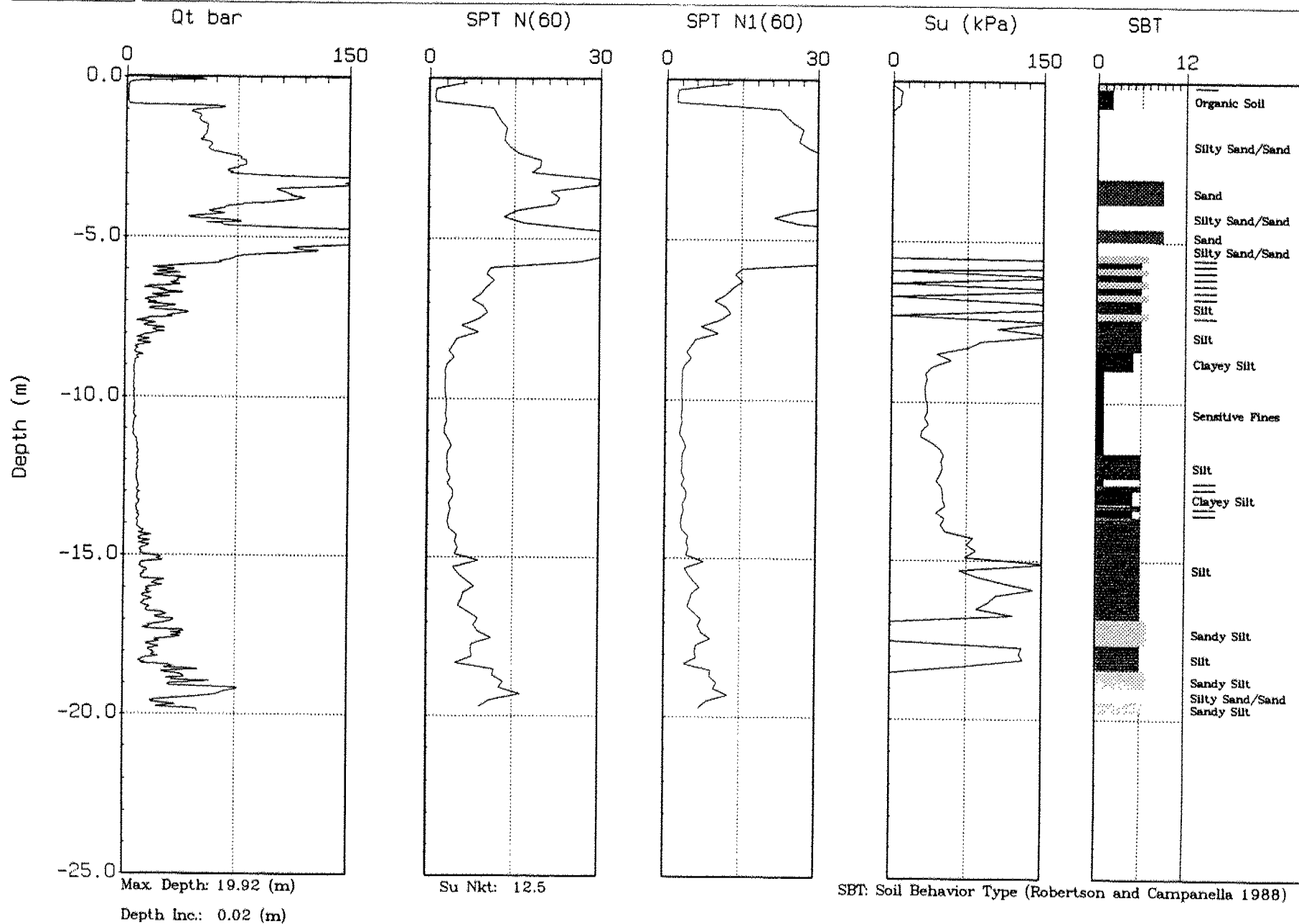




Thurber Engineering

Site: 99-141 DPT-N4  
Location: INTERCHANGE

Cone: 20 TON A 058  
Date: 03/26/99 14:44

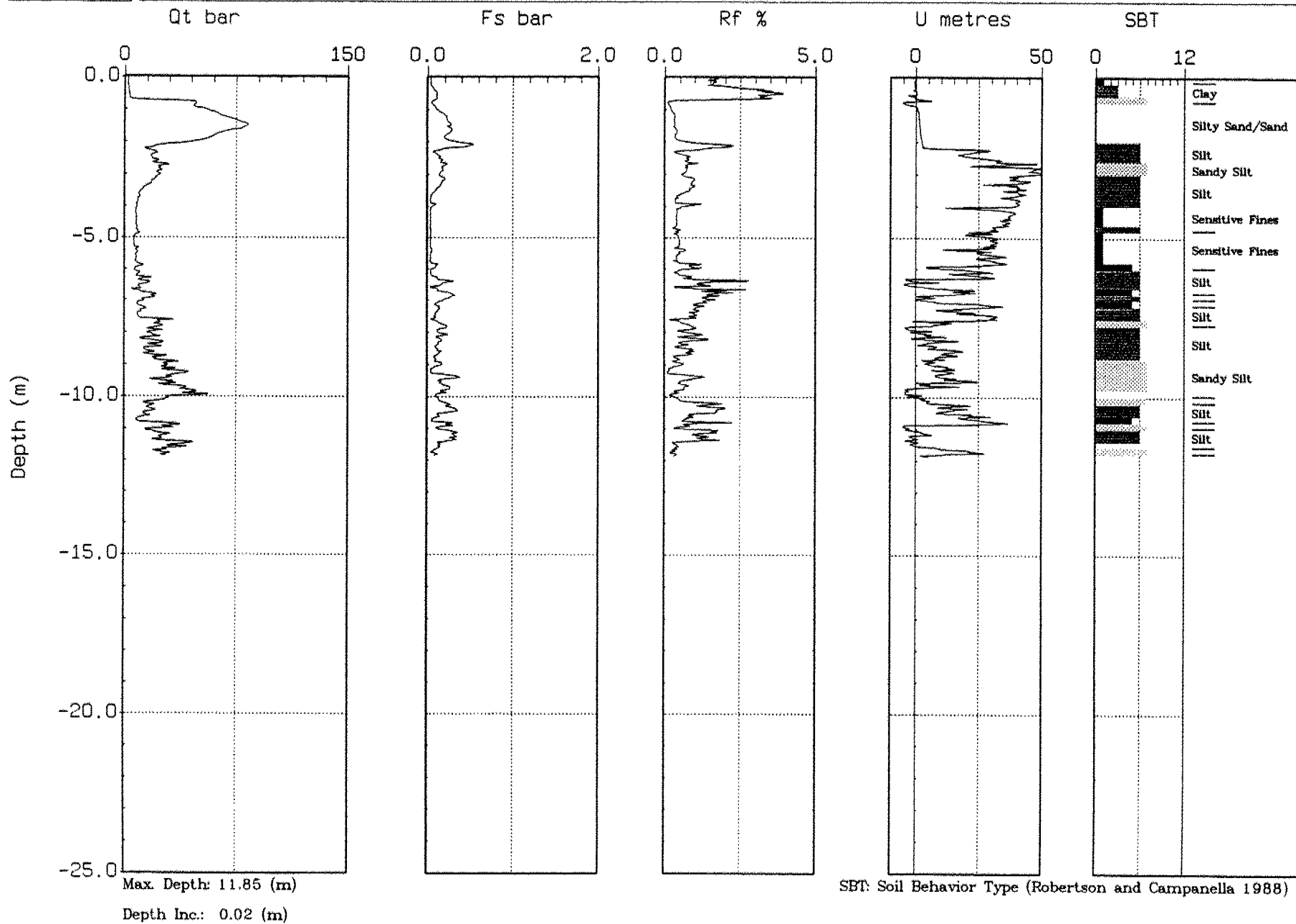




Thurber Engineering

Site: 99-141 CPT-N5  
Location: N.INTERCHANGE

Cone: 20 TON A 058  
Date: 03/26/99 13:39

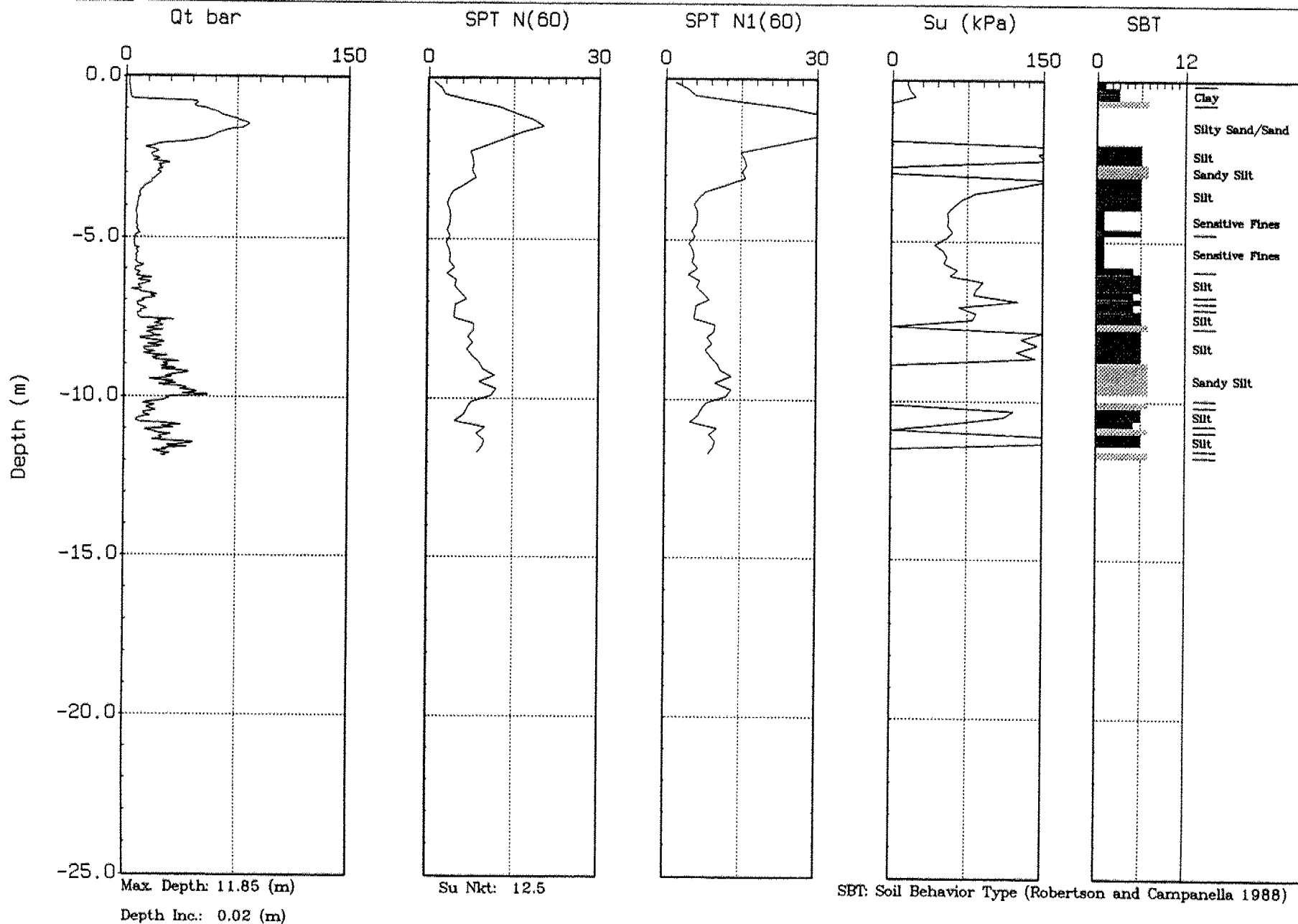




Thurber Engineering

Site: 99-141 CPT-N5  
Location: NINTERCHANGE

Cone: 20 TON A 058  
Date: 03/26/99 13:39





**Thurber Engineering**

## **APPENDIX B**

### **CPT Interpretations**

**ConeTec Investigations Ltd.**



### ConeTec CPT Interpretations as of January 7, 1999 (Release 1.00.19)

ConeTec's interpretation routine should be considered a calculator of current published CPT correlations and is subject to change to reflect the current state of practice. The interpreted values are not considered valid for all soil types. The interpretations are presented only as a guide for geotechnical use and should be carefully scrutinized for consideration in any geotechnical design. Reference to current literature is strongly recommended.

The CPT interpretations are based on values of tip, sleeve friction and pore pressure averaged over a user specified interval (typically 0.25m). Note that  $Q_t$  is the recorded tip value,  $Q_c$ , corrected for pore pressure effects. Since all ConeTec cones have equal end area friction sleeves, pore pressure corrections to sleeve friction,  $F_s$ , are not required.

The tip correction is:  $Q_t = Q_c + (1-a) \cdot U_d$

where:  $Q_t$  is the corrected tip load

$Q_c$  is the recorded tip load

$U_d$  is the recorded dynamic pore pressure

$a$  is the Net Area Ratio for the cone (typically 0.85 for ConeTec cones)

Effective vertical overburden stresses are calculated based on a hydrostatic distribution of equilibrium pore pressures below the water table or from a user defined equilibrium pore pressure profile (this can be obtained from CPT dissipation tests). The stress calculations use unit weights assigned to the Soil Behaviour Type zones or from a user defined unit weight profile.

Details regarding the interpretation methods for all of the interpreted parameters is given in table 1. The appropriate references referred to in table 1 are listed in table 2.

The estimated Soil Behaviour Type is based on the charts developed by Robertson and Campanella shown in figure 1.

**Table 1 CPT Interpretation Methods**

Interpreted Parameter	Description	Equation	Ref
Depth	mid layer depth		
Avg $Q_t$	Averaged corrected tip ( $Q_t$ )	$AvgQ_t = \frac{1}{n} \sum_{i=1}^n Q_{t_i}$	
Avg $F_s$	Averaged sleeve friction ( $F_s$ )	$AvgF_s = \frac{1}{n} \sum_{i=1}^n F_{s_i}$	
Avg $R_f$	Averaged friction ratio ( $R_f$ )	$AvgR_f = 100\% \cdot \frac{AvgF_s}{AvgQ_t}$	
Avg $U_d$	Averaged dynamic pore pressure ( $U_d$ )	$AvgU_d = \frac{1}{n} \sum_{i=1}^n U_{d_i}$	
SBT	Soil Behavior Type as defined by Robertson and Campanella		1

# CPT Interpretations

U.Wt.	Unit Weight of soil determined from: 1) uniform value or 2) value assigned to each SBT zone 3) user supplied unit weight profile		
TStress	Total vertical overburden stress at mid layer depth	$TStress = \sum_{i=1}^n \gamma_i h_i$ where $\gamma_i$ is layer unit weight $h_i$ is layer thickness	
EStress	Effective vertical overburden stress at mid layer depth	$EStress = TStress - Ueq$	
Ueq	Equilibrium pore pressure determined from: 1) hydrostatic from water table depth 2) user supplied profile		
Cn	SPT $N_{60}$ overburden correction factor	$Cn = (\sigma_v')^{0.5}$ where $\sigma_v'$ is in tsf $0.5 < Cn < 2.0$	
$N_{60}$	SPT N value at 60% energy calculated from Qt/N ratios assigned to each SBT zone		3
$(N1)_{60}$	SPT $N_{60}$ value corrected for overburden pressure	$N1_{60} = Cn \cdot N_{60}$	3
$\Delta(N1)_{60}$	Equivalent Clean Sand Correction to $(N1)_{60}$	$\Delta(N1)_{60} = \frac{K_{SPT}}{1 - K_{SPT}} \cdot (N1)_{60}$ Where: $K_{SPT}$ is defined as: 0.0 for FC < 5% 0.0167 • (FC - 5) for 5% < FC < 35% 0.5 for FC > 35% FC - Fines Content in %	7
$(N1)_{60cs}$	Equivalent Clean Sand $(N1)_{60}$	$(N1)_{60cs} = (N1)_{60} + \Delta(N1)_{60}$	7
Su	Undrained shear strength - Nkt is use selectable	$Su = \frac{Qt - \sigma_v}{N_{kt}}$	2
k	Coefficient of permeability (assigned to each SBT zone)		6
Bq	Pore pressure parameter	$Bq = \frac{\Delta u}{Qt - \sigma_v}$	2
Qtn	Normalized Qt for Soil Behavior Type classification as defined by Robertson, 1990	$Qtn = \frac{Qt - \sigma_v}{\sigma_v}$	4
Rfn	Normalized Rf for Soil Behavior Type classification as defined by Robertson, 1990	$Rfn = 100\% \cdot \frac{f_s}{Qt - \sigma_v}$	4
SBTn	Normalized Soil Behavior Type (slightly modified from that published by Robertson, 1990. This version includes all the soil zones of the original non-normalized SBT chart - see figure 1)		4
Qc1	Normalized Qt for seismic analysis	$qc1 = qc \cdot (Pa/\sigma_v')^{0.5}$ where: Pa = atm. pressure	5
Qc1N	Dimensionless Normalized Qt1	$qc1N = qc1 / Pa$ where: Pa = atm. pressure	

# CPT Interpretations

$\Delta Q_{c1N1}$	Equivalent clean sand correction	$\Delta q_{c1N} = \frac{K_{crr}}{1 - K_{crr}} \cdot q_{c1N}$ <p>Where: <math>K_{CPT}</math> is defined as:</p> <p>0.0 for <math>FC &lt; 5\%</math>  <math>0.0267 \cdot (FC - 5)</math> for <math>5\% &lt; FC &lt; 35\%</math>  0.5 for <math>FC &gt; 35\%</math></p> <p>FC - Fines Content in %</p>	5
$Q_{c1Ncs}$	Clean Sand equivalent $Q_{c1N}$	$q_{c1Ncs} = q_{c1N} + \Delta q_{c1N}$	5
$I_c$	Soil index for estimating grain characteristics	$I_c = [(3.47 - \log Q)^2 + (\log F + 1.22)^2]^{0.5}$	5
FC	Fines content (%)	$FC = 1.75(I_c^{0.25}) - 3.7$ $FC = 100$ for $I_c > 3.5$ $FC = 0$ for $I_c < 1.26$ $FC = 5\%$ if $1.64 < I_c < 2.6$ AND $R_{fm} < 0.5$	8
PHI	Friction Angle	Campanella and Robertson Durunoglu and Mitchel	1
Dr	Relative Density	Janbu Ticino Sand Hokksund Sand Schmertmann 1976 Jamolkowski - All Sands	1
OCR	Over Consolidation Ratio		1
State Parameter			9
CRR	Cyclic Resistance Ratio		7

# CPT Interpretations

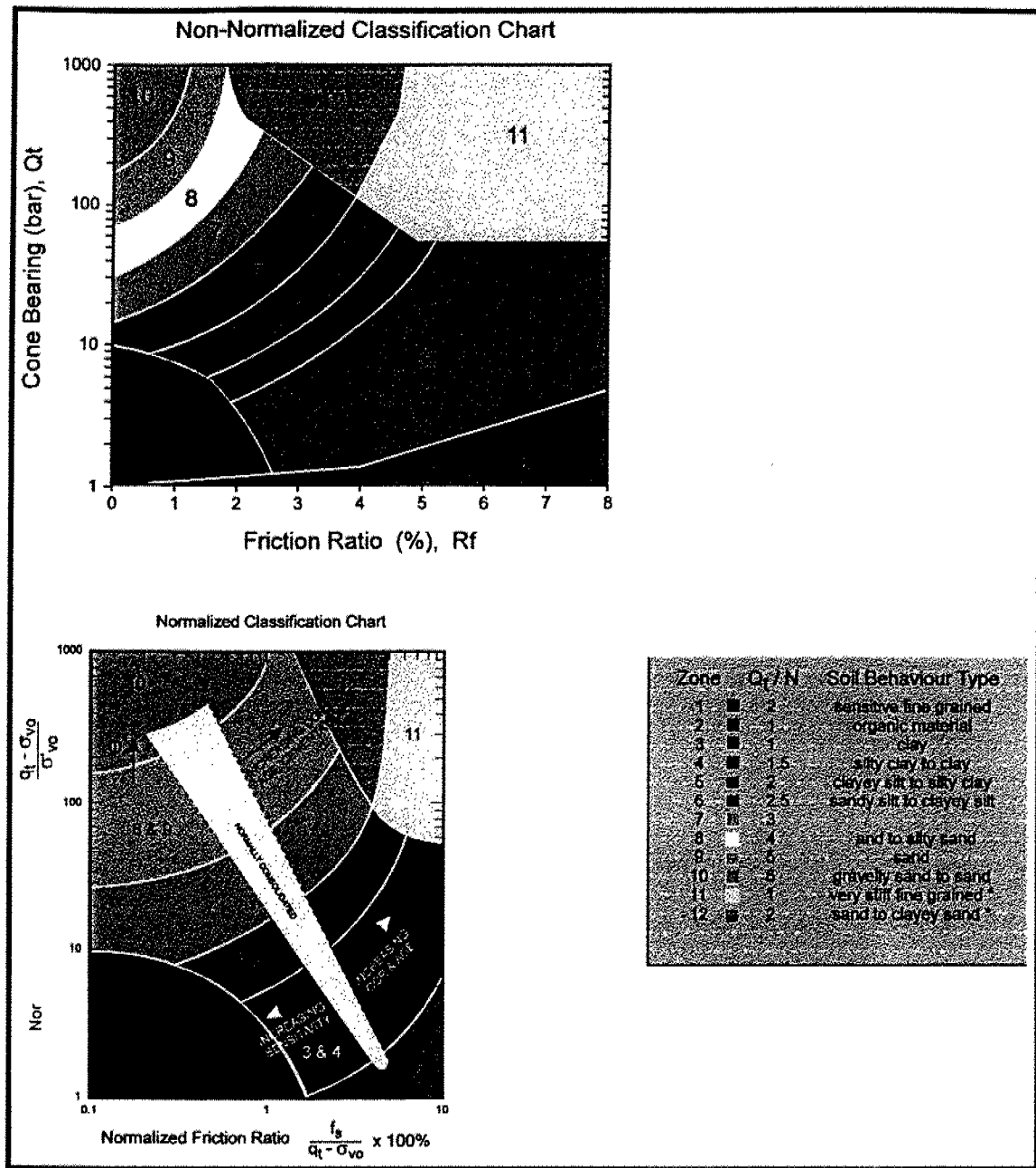


Figure 1 Non-Normalized and Normalized Soil Behaviour Type Classification Charts

## CPT Interpretations

**Table 2    References**

No.	Reference
1	Robertson, P.K. and Campanella, R.G., 1986, "Guidelines for Use, Interpretation and Application of the CPT and CPTU", UBC, Soil Mechanics Series No. 105, Civil Eng. Dept., Vancouver, B.C., Canada
2	Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, "Use of Piezometer Cone Data", Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.
3	Robertson, P.K. and Campanella, R.G., 1989, "Guidelines for Geotechnical Design Using CPT and CPTU", UBC, Soil Mechanics Series No. 120, Civil Eng. Dept., Vancouver, B.C., Canada
4	Robertson, P.K., 1990, "Soil Classification Using the Cone Penetration Test", Canadian Geotechnical Journal, Volume 27.
5	Robertson, P.K. and Fear, C.E., 1995, "Liquefaction of Sands and its Evaluation", Keynote Lecture, First International Conference on Earthquake Geotechnical Engineering, Tokyo, Japan.
6	ConeTec Internal Report
7	Robertson, P.K. and Wride, C.E., 1997, "Cyclic Liquefaction and its Evaluation Based on SPT and CPT", NCEER Workshop Paper, January 22, 1997
8	Wride, C.E. and Robertson, P.K., 1997, "Phase II Data Review Report (Massey and Kidd Sites, Fraser River Delta)", Volume 1 - Data Report (June 1997), University of Alberta.
9	Plewes, H.D., Davies, M.P. and Jefferies, M.G., 1992, "CPT Based Screening Procedure for Evaluating Liquefaction Susceptibility", 45th Canadian Geotechnical Conference, Toronto, Ontario, October 1992.

Run No: 99-0331-0824-2828

Job No: 99-141

Client: Thurber Engineering

Project: Trout Lake By-Pass

Site: 99-141 CPT-N1

Location: N.INTERCHANGE

Cone: 20 TON A 058

CPT Date: 99/26/03

CPT Time: 08:19

CPT File: 141CPN1.COR

Northing (m): 0.000

Easting (m): 0.000

Elevation (m): 0.000

Water Table (m): 0.00 (ft): 0.0

Su Nkt used: 12.50

Averaging Increment (m): 0.25

Phi Method: Robertson and Campanella, 1983

Dr Method: Jamiolkowski - All Sands

State Parameter M: 1.20

Used Unit Weights Assigned to Soil Zones

Values of 1.0E9 or UnDef are printed for parameters that are not valid for the material type (SBT)

Depth (m)	AvgQt (bar)	AvgFs (bar)	AvgRf (%)	AvgUd (m)	SBT	U.Wt. (kN/m <sup>3</sup> )	TStress (kPa)	ESTress (kPa)	Ueq (kPa)	Cn	N60 (blows/ft)	(N1)60 (blows/ft)	Su (kPa)	CRR
0.12	7.4	0.03	0.42	0.01	1	17.5	2.2	1.0	1.23	2.00	3.7	7.4	58.7	0.00
0.38	1.7	0.10	5.74	0.31	2	12.5	5.9	2.3	3.68	2.00	1.7	3.5	13.5	0.00
0.62	10.6	0.16	1.50	-1.38	5	18.0	9.8	3.6	6.13	2.00	5.3	10.6	84.4	0.00
0.88	15.0	0.08	0.51	-2.11	6	18.0	14.2	5.7	8.58	2.00	6.0	12.0	118.9	0.08
1.12	19.3	0.11	0.55	0.77	6	18.0	18.8	7.7	11.04	2.00	7.7	15.5	153.3	0.09
1.38	35.6	0.18	0.50	1.69	7	18.5	23.3	9.8	13.49	2.00	11.9	23.7	UnDef	0.11
1.62	53.3	0.28	0.53	1.65	8	19.0	28.0	12.1	15.94	2.00	13.3	26.7	UnDef	0.19
1.88	61.0	0.32	0.53	2.08	8	19.0	32.8	14.4	18.39	2.00	15.3	30.5	UnDef	0.25
2.12	66.0	0.38	0.58	2.57	8	19.0	37.5	16.7	20.85	2.00	16.5	33.0	UnDef	0.29
2.38	80.8	0.48	0.59	2.57	8	19.0	42.2	19.0	23.30	2.00	20.2	40.4	UnDef	0.00
2.62	75.8	0.54	0.71	3.20	8	19.0	47.0	21.2	25.75	2.00	19.0	37.9	UnDef	0.40
2.88	25.3	0.40	1.58	15.90	6	18.0	51.6	23.4	28.20	2.00	10.1	20.3	198.4	0.11
3.12	20.2	0.17	0.85	32.82	6	18.0	56.1	25.5	30.66	1.94	8.1	15.7	157.5	0.09
3.38	18.2	0.31	1.68	31.61	6	18.0	60.6	27.5	33.11	1.87	7.3	13.6	140.8	0.10
3.62	10.5	0.14	1.37	34.13	5	18.0	65.1	29.6	35.56	1.80	5.3	9.5	79.1	0.09
3.88	7.6	0.09	1.21	36.62	5	18.0	69.6	31.6	38.01	1.74	3.8	6.6	55.1	0.09
4.12	7.3	0.06	0.85	37.73	5	18.0	74.1	33.7	40.47	1.69	3.6	6.1	52.1	0.09
4.38	7.2	0.07	0.93	35.79	5	18.0	78.6	35.7	42.92	1.64	3.6	5.9	51.5	0.09
4.62	7.4	0.08	1.12	29.52	5	18.0	83.1	37.8	45.37	1.59	3.7	5.9	52.8	0.10
4.88	6.7	0.11	1.70	29.03	5	18.0	87.6	39.8	47.82	1.55	3.4	5.2	46.8	0.09
5.12	6.8	0.09	1.30	28.26	5	18.0	92.1	41.8	50.28	1.51	3.4	5.2	47.3	0.09
5.38	7.2	0.09	1.30	28.50	5	18.0	96.6	43.9	52.73	1.48	3.6	5.3	49.7	0.09
5.62	8.5	0.14	1.69	16.67	5	18.0	101.1	45.9	55.18	1.44	4.2	6.1	59.6	0.10
5.88	10.6	0.15	1.38	12.94	5	18.0	105.6	48.0	57.63	1.41	5.3	7.5	76.4	0.12
6.12	9.0	0.17	1.85	21.72	5	18.0	110.1	50.0	60.09	1.38	4.5	6.3	63.5	0.10
6.38	13.5	0.18	1.37	9.34	6	18.0	114.6	52.1	62.54	1.36	5.4	7.3	98.7	0.10
6.62	8.3	0.12	1.40	23.94	5	18.0	119.1	54.1	64.99	1.33	4.2	5.5	56.9	0.10
6.88	7.7	0.18	2.28	25.95	4	18.0	123.6	56.2	67.44	1.31	5.1	6.7	51.8	0.09
7.12	17.1	0.15	0.88	9.65	6	18.0	128.1	58.2	69.90	1.28	6.8	8.8	126.3	0.09
7.38	14.5	0.13	0.88	12.13	6	18.0	132.6	60.3	72.35	1.26	5.8	7.3	105.0	0.09
7.62	13.2	0.14	1.04	15.96	6	18.0	137.1	62.3	74.80	1.24	5.3	6.5	94.7	0.10
7.88	16.8	0.15	0.89	10.25	6	18.0	141.6	64.4	77.25	1.22	6.7	8.2	123.1	0.09
8.12	13.7	0.13	0.97	15.07	6	18.0	146.1	66.4	79.71	1.20	5.5	6.6	98.2	0.10
8.38	20.0	0.11	0.57	10.48	7	18.5	150.7	68.5	82.16	1.18	6.7	7.9	UnDef	0.09
8.62	25.0	0.18	0.72	12.69	7	18.5	155.3	70.7	84.61	1.16	8.3	9.7	UnDef	0.09
8.88	20.2	0.27	1.33	15.30	6	18.0	159.9	72.8	87.06	1.15	8.1	9.2	148.5	0.11
9.12	29.6	0.17	0.58	6.11	7	18.5	164.4	74.9	89.52	1.13	9.9	11.2	UnDef	0.09
9.38	33.6	0.15	0.44	5.09	7	18.5	169.1	77.1	91.97	1.11	11.2	12.5	UnDef	0.09
9.62	14.6	0.21	1.45	15.63	6	18.0	173.6	79.2	94.42	1.10	5.9	6.4	103.3	0.13

Run No: 99-0331-0824-2828

CPT File: 141CPN1.COR

Depth (m)	AvgQt (bar)	AvgFs (bar)	AvgRf (%)	AvgUd (m)	SBT	U.Wt. (kN/m <sup>3</sup> )	TStress (kPa)	ESTress (kPa)	Ueq (kPa)	Cn	N60 (blows/ft)	(N1)60 (blows/ft)	Su (kPa)	CRR
9.88	13.5	0.17	1.26	19.24	6	18.0	178.1	81.3	96.87	1.09	5.4	5.9	93.8	0.12
10.12	10.9	0.08	0.74	24.84	6	18.0	182.6	83.3	99.33	1.07	4.4	4.7	72.5	0.10
10.38	20.0	0.11	0.57	15.60	7	18.5	187.2	85.4	101.78	1.06	6.7	7.1	UnDef	0.09
10.62	19.8	0.16	0.80	13.32	6	18.0	191.8	87.5	104.23	1.05	7.9	8.3	143.1	0.10
10.88	39.9	0.25	0.62	12.07	7	18.5	196.3	89.6	106.68	1.03	13.3	13.7	UnDef	0.10
11.12	28.4	0.15	0.52	15.64	7	18.5	200.9	91.8	109.14	1.02	9.5	9.7	UnDef	0.09
11.38	27.6	0.12	0.43	6.20	7	18.5	205.6	94.0	111.59	1.01	9.2	9.3	UnDef	0.00
11.62	24.8	0.25	1.01	17.77	6	18.0	210.1	96.1	114.04	1.00	9.9	9.9	181.9	0.11
11.88	89.3	0.76	0.85	7.74	8	19.0	214.8	98.3	116.49	0.99	22.3	22.0	UnDef	0.19



Interpretation Output - Release 1.00.17

Run No: 99-0331-0824-2828

Job No: 99-141

Client: Thurber Engineering

Project: Trout Lake By-Pass

Site: 99-141 CPT-N1

Location: N.INTERCHANGE

Cone: 20 TON A 058

CPT Date: 99/26/03

CPT Time: 08:19

CPT File: 141CPN1.COR

Northing (m): 0.000

Easting (m): 0.000

Elevation (m): 0.000

Water Table (m): 0.00 (ft): 0.0

Su Nkt used: 12.50

Averaging Increment (m): 0.25

Phi Method : Robertson and Campanella, 1983

Dr Method : Jamiolkowski - All Sands

State Parameter M: 1.20

Used Unit Weights Assigned to Soil Zones

Values of 1.0E9 or UnDef are printed for parameters that are not valid for the material type (SBT)

Depth (m)	k (cm/s)	Bq	Qtn	Rfn (%)	SBTn	Qc1N	DeltaQc1N	Qc1Ncs	Fc (%)	Phi (Deg)	Dr (%)	OCR	State Del(n1)60 (N1) Param
0.12	1.0E-07	0.00	762.8	0.42	10	14.7	0.0	14.7	0.0	UnDef	UnDef	10.0	UnDef 0.0
0.38	1.0E-15	0.00	74.6	5.94	11	3.5	UnDef	UnDef	0.0	UnDef	UnDef	10.0	UnDef UnDef
0.62	5.0E-06	-0.02	291.4	1.52	9	21.3	0.9	22.2	6.6	UnDef	UnDef	10.0	UnDef 0.3
0.88	5.0E-05	-0.02	262.3	0.52	9	30.0	0.0	30.0	1.6	46	54.0	10.0	-0.22 0.0
1.12	5.0E-05	0.00	248.4	0.56	9	38.7	0.0	38.7	2.1	46	56.9	10.0	-0.22 0.0
1.38	5.0E-04	0.00	360.0	0.51	10	71.2	0.0	71.2	0.4	48	70.9	1.0	-0.24 0.0
1.62	5.0E-03	0.00	439.7	0.53	10	106.6	0.0	106.6	0.0	48	79.6	1.0	-0.26 0.0
1.88	5.0E-03	0.00	422.9	0.53	10	122.1	0.0	122.1	0.1	48	80.9	1.0	-0.26 0.0
2.12	5.0E-03	0.00	394.3	0.58	10	132.1	0.0	132.1	0.6	48	81.1	1.0	-0.26 0.0
2.38	5.0E-03	0.00	424.0	0.60	10	161.6	0.0	161.6	0.4	48	85.0	1.0	-0.27 0.0
2.62	5.0E-03	0.00	354.6	0.72	9	151.6	0.0	151.6	1.7	48	81.5	1.0	-0.27 0.0
2.88	5.0E-05	0.05	105.9	1.62	7	50.6	15.6	66.3	13.8	42	48.7	10.0	-0.24 3.5
3.12	5.0E-05	0.15	77.3	0.87	9	40.1	9.1	49.3	12.0	42	41.1	10.0	-0.14 2.1
3.38	5.0E-05	0.16	64.0	1.74	7	34.7	21.8	56.5	19.5	40	36.9	10.0	-0.18 4.3
3.62	5.0E-06	0.30	33.5	1.46	7	19.4	24.4	43.8	25.9	UnDef	UnDef	6.0	UnDef 5.1
3.88	5.0E-06	0.47	21.8	1.34	7	13.5	33.2	46.7	31.6	UnDef	UnDef	6.0	UnDef 5.3
4.12	5.0E-06	0.51	19.3	0.95	7	12.5	25.9	38.4	30.3	UnDef	UnDef	6.0	UnDef 4.5
4.38	5.0E-06	0.48	18.0	1.04	7	12.1	32.6	44.7	32.3	UnDef	UnDef	6.0	UnDef 5.0
4.62	5.0E-06	0.37	17.5	1.26	6	12.1	47.4	59.5	34.8	UnDef	UnDef	6.0	UnDef 5.9
4.88	5.0E-06	0.41	14.7	1.95	6	10.7	42.6	53.3	43.4	UnDef	UnDef	6.0	UnDef 5.2
5.12	5.0E-06	0.38	14.1	1.51	6	10.6	42.2	52.8	40.9	UnDef	UnDef	6.0	UnDef 5.2
5.38	5.0E-06	0.37	14.1	1.50	6	10.8	43.3	54.2	40.8	UnDef	UnDef	6.0	UnDef 5.3
5.62	5.0E-06	0.15	16.2	1.92	6	12.5	49.9	62.4	41.2	UnDef	UnDef	6.0	UnDef 6.1
5.88	5.0E-06	0.07	19.9	1.53	6	15.3	58.2	73.5	34.7	UnDef	UnDef	6.0	UnDef 7.4
6.12	5.0E-06	0.19	15.9	2.10	6	12.8	51.1	63.9	42.9	UnDef	UnDef	6.0	UnDef 6.3
6.38	5.0E-05	0.02	23.7	1.49	7	18.7	44.8	63.5	31.4	34	30.0	6.0	-0.08 5.8
6.62	5.0E-06	0.24	13.1	1.63	6	11.3	45.2	56.4	43.3	UnDef	UnDef	6.0	UnDef 5.5
6.88	5.0E-07	0.29	11.5	2.72	4	10.3	41.2	51.5	53.3	UnDef	UnDef	3.0	UnDef 6.7
7.12	5.0E-05	0.02	27.1	0.95	7	22.4	25.4	47.8	24.9	36	30.0	6.0	-0.06 4.4
7.38	5.0E-05	0.04	21.8	0.97	7	18.6	31.2	49.9	28.5	34	30.0	6.0	-0.04 4.7
7.62	5.0E-05	0.07	19.0	1.16	7	16.7	46.2	62.9	32.5	34	30.0	6.0	-0.03 5.6
7.88	5.0E-05	0.02	23.9	0.97	7	20.9	29.8	50.8	27.0	34	30.0	6.0	-0.05 4.8
8.12	5.0E-05	0.06	18.5	1.08	7	16.8	45.4	62.2	32.3	32	30.0	6.0	-0.03 5.5
8.38	5.0E-04	0.01	27.0	0.62	7	24.2	19.3	43.4	21.6	36	30.0	1.0	-0.03 3.0
8.62	5.0E-04	0.02	33.2	0.77	7	29.7	20.8	50.6	20.4	36	32.5	1.0	-0.06 3.4
8.88	5.0E-05	0.03	25.5	1.44	7	23.6	46.9	70.5	29.9	36	30.0	6.0	-0.08 6.6
9.12	5.0E-04	-0.01	37.4	0.61	7	34.2	16.7	51.0	17.3	38	36.6	1.0	-0.06 2.9
9.38	5.0E-04	-0.01	41.4	0.47	9	38.2	0.0	38.2	5.0	38	39.7	1.0	-0.04 0.0
9.62	5.0E-05	0.05	16.3	1.65	6	16.5	65.8	82.3	39.2	32	30.0	6.0	-0.05 6.4

Run No: 99-0331-0824-2828

CPT File: 141CPN1.COR

Depth (m)	k (cm/s)	Bq	Qtn	Rfn (%)	SBTn	Qc1N	DeltaQc1N	Qc1Ncs	Fc (%)	Phi (Deg)	Dr (%)	OCR	State Del(n1) Param	60 (N)
9.88	5.0E-05	0.08	14.4	1.45	6	15.0	60.0	74.9	40.0	32	30.0	6.0	-0.02	5.9
10.12	5.0E-05	0.16	10.9	0.89	6	11.9	47.7	59.6	40.5	30	30.0	3.0	0.05	4.7
10.38	5.0E-04	0.03	21.3	0.63	7	21.7	25.6	47.3	25.3	34	30.0	1.0	0.00	3.6
10.62	5.0E-05	0.01	20.4	0.88	7	21.2	36.4	57.6	28.7	34	30.0	6.0	-0.03	5.4
10.88	5.0E-04	0.00	42.3	0.65	7	42.1	18.0	60.2	16.2	38	42.5	1.0	-0.07	3.2
11.12	5.0E-04	0.02	28.8	0.56	7	29.7	19.9	49.6	20.0	36	32.4	1.0	-0.02	3.2
11.38	5.0E-04	-0.02	27.2	0.47	7	28.5	0.0	28.5	5.0	36	31.3	1.0	-0.01	0.0
11.62	5.0E-05	0.03	23.7	1.10	7	25.3	42.0	67.4	28.4	34	30.0	6.0	-0.06	6.3
11.88	5.0E-03	0.00	88.7	0.87	9	90.1	16.4	106.4	10.8	42	64.3	1.0	-0.16	2.3

Interpretation Output - Release 1.00.17

Run No: 99-0331-0827-2069

Job No: 99-141

Client: Thurber Engineering

Project: Trout Lake By-Pass

Site: 99-141 CPT-N2

Location: N.INTERCHANGE

Cone: 20 TON A 058

CPT Date: 99/26/03

CPT Time: 09:54

CPT File: 141CPN2.COR

Northing (m): 0.000

Easting (m): 0.000

Elevation (m): 0.000

Water Table (m): 0.00 (ft): 0.0

Su Nkt used: 12.50

Averaging Increment (m): 0.25

Phi Method: Robertson and Campanella, 1983

Dr Method: Jamiolkowski - All Sands

State Parameter M: 1.20

Used Unit Weights Assigned to Soil Zones

Values of 1.0E9 or UnDef are printed for parameters that are not valid for the material type (SBT)

Depth (m)	AvgQt (bar)	AvgFs (bar)	AvgRf (%)	AvgUd (m)	SBT	U.Wt. (kN/m <sup>3</sup> )	TStress (kPa)	ESTress (kPa)	Ueq (kPa)	Cn	N60 (blows/ft)	(N1)60	Su (kPa)	CRR
0.12	2.9	0.03	1.05	0.13	1	17.5	2.2	1.0	1.23	2.00	1.4	2.9	22.7	0.00
0.38	3.8	0.03	0.72	0.28	1	17.5	6.6	2.9	3.68	2.00	1.9	3.8	29.7	0.00
0.62	2.7	0.08	2.87	0.34	3	17.5	10.9	4.8	6.13	2.00	2.7	5.4	20.6	0.00
0.88	4.8	0.07	1.43	0.24	1	17.5	15.3	6.7	8.58	2.00	2.4	4.8	36.8	0.00
1.12	15.8	0.05	0.30	-3.13	6	18.0	19.8	8.7	11.04	2.00	6.3	12.6	124.6	0.08
1.38	31.6	0.06	0.21	0.14	7	18.5	24.3	10.8	13.49	2.00	10.5	21.1	UnDef	0.10
1.62	47.5	0.17	0.35	1.29	8	19.0	29.0	13.1	15.94	2.00	11.9	23.8	UnDef	0.16
1.88	68.1	0.26	0.38	1.50	8	19.0	33.8	15.4	18.39	2.00	17.0	34.0	UnDef	0.31
2.12	72.3	0.38	0.52	1.83	8	19.0	38.5	17.7	20.85	2.00	18.1	36.2	UnDef	0.36
2.38	73.6	0.44	0.60	2.29	8	19.0	43.2	20.0	23.30	2.00	18.4	36.8	UnDef	0.38
2.62	71.4	0.39	0.55	2.52	8	19.0	48.0	22.2	25.75	2.00	17.8	35.7	UnDef	0.35
2.88	77.7	0.35	0.45	3.67	8	19.0	52.8	24.5	28.20	1.98	19.4	38.4	UnDef	0.43
3.12	95.7	0.51	0.53	3.53	8	19.0	57.5	26.8	30.66	1.89	23.9	45.2	UnDef	0.00
3.38	102.0	0.46	0.45	-0.08	9	19.5	62.3	29.2	33.11	1.81	20.4	37.0	UnDef	0.00
3.62	138.1	0.49	0.36	3.08	9	19.5	67.2	31.6	35.56	1.74	27.6	48.1	UnDef	0.00
3.88	130.9	0.48	0.37	1.36	9	19.5	72.1	34.0	38.01	1.68	26.2	43.9	UnDef	0.00
4.12	65.4	0.53	0.82	2.18	8	19.0	76.9	36.4	40.47	1.62	16.4	26.5	UnDef	0.20
4.38	28.4	0.71	2.50	-3.09	6	18.0	81.5	38.6	42.92	1.58	11.4	17.9	220.8	0.14
4.62	35.3	0.52	1.48	-5.94	7	18.5	86.1	40.7	45.37	1.53	11.8	18.1	UnDef	0.12
4.88	10.4	0.09	0.84	6.62	6	18.0	90.6	42.8	47.82	1.50	4.2	6.2	76.2	0.09
5.12	10.8	0.09	0.88	17.76	6	18.0	95.1	44.8	50.28	1.46	4.3	6.3	78.8	0.09
5.38	8.5	0.09	1.05	19.09	5	18.0	99.6	46.9	52.73	1.43	4.2	6.0	59.7	0.10
5.62	7.9	0.05	0.62	29.46	5	18.0	104.1	48.9	55.18	1.40	4.0	5.6	55.2	0.09
5.88	7.3	0.04	0.54	30.99	1	17.5	108.6	50.9	57.63	1.37	3.6	5.0	49.3	0.09
6.12	6.9	0.04	0.53	31.06	1	17.5	112.9	52.9	60.09	1.35	3.5	4.7	46.3	0.09
6.38	7.0	0.04	0.56	32.27	1	17.5	117.3	54.8	62.54	1.32	3.5	4.6	46.3	0.09
6.62	7.1	0.04	0.51	31.46	1	17.5	121.7	56.7	64.99	1.30	3.5	4.6	46.7	0.09
6.88	6.6	0.04	0.61	31.59	1	17.5	126.1	58.6	67.44	1.28	3.3	4.2	42.5	0.09
7.12	6.9	0.04	0.56	35.77	1	17.5	130.4	60.5	69.90	1.26	3.5	4.4	45.1	0.09
7.38	7.2	0.04	0.54	34.34	1	17.5	134.8	62.5	72.35	1.24	3.6	4.5	47.2	0.09
7.62	7.4	0.04	0.54	32.81	1	17.5	139.2	64.4	74.80	1.22	3.7	4.5	47.9	0.09
7.88	7.1	0.05	0.77	33.46	5	18.0	143.6	66.4	77.25	1.20	3.5	4.3	45.3	0.09
8.12	7.5	0.05	0.63	30.58	5	18.0	148.1	68.4	79.71	1.18	3.7	4.4	47.8	0.09
8.38	7.7	0.04	0.52	34.79	1	17.5	152.6	70.4	82.16	1.17	3.8	4.5	49.3	0.09
8.62	7.8	0.04	0.51	34.86	1	17.5	156.9	72.3	84.61	1.15	3.9	4.5	50.0	0.09
8.88	8.1	0.04	0.51	32.97	6	18.0	161.4	74.3	87.06	1.14	3.2	3.7	51.6	0.09
9.12	8.1	0.04	0.49	32.45	6	18.0	165.9	76.4	89.52	1.12	3.3	3.6	51.8	0.09
9.38	8.9	0.04	0.45	31.28	6	18.0	170.4	78.4	91.97	1.11	3.5	3.9	57.2	0.09
9.62	8.6	0.04	0.46	36.69	6	18.0	174.9	80.5	94.42	1.09	3.5	3.8	55.1	0.09

Depth (m)	AvgQt (bar)	AvgFs (bar)	AvgRf (%)	AvgUd (m)	SBT	U.Wt. (kN/m <sup>3</sup> )	TStress (kPa)	ESTress (kPa)	Ueq (kPa)	Cn	N60 (blows/ft)	(N1)60	Su (kPa)	CRR
9.88	10.7	0.11	1.06	31.03	6	18.0	179.4	82.5	96.87	1.08	4.3	4.6	71.6	0.10
10.12	11.8	0.13	1.12	21.73	6	18.0	183.9	84.5	99.33	1.06	4.7	5.0	79.3	0.10
10.38	12.6	0.13	1.00	21.16	6	18.0	188.4	86.6	101.78	1.05	5.1	5.3	86.0	0.11
10.62	17.7	0.22	1.23	16.88	6	18.0	192.9	88.6	104.23	1.04	7.1	7.3	125.9	0.16
10.88	12.0	0.12	0.98	18.30	6	18.0	197.4	90.7	106.68	1.03	4.8	4.9	80.1	0.10
11.12	10.9	0.11	0.99	29.29	6	18.0	201.9	92.7	109.14	1.02	4.3	4.4	70.7	0.10
11.38	12.2	0.11	0.86	32.36	6	18.0	206.4	94.8	111.59	1.01	4.9	4.9	80.8	0.10
11.62	21.8	0.11	0.50	19.89	7	18.5	210.9	96.9	114.04	0.99	7.3	7.2	UnDef	0.09
11.88	17.6	0.19	1.09	22.58	6	18.0	215.5	99.0	116.49	0.98	7.0	6.9	123.6	0.14
12.12	20.5	0.20	0.96	8.70	6	18.0	220.0	101.1	118.95	0.97	8.2	8.0	146.2	0.12
12.38	20.1	0.17	0.84	12.28	6	18.0	224.5	103.1	121.40	0.96	8.0	7.7	142.8	0.11
12.62	22.1	0.12	0.55	12.92	7	18.5	229.1	105.2	123.85	0.95	7.4	7.0	UnDef	0.09
12.88	30.0	0.24	0.79	12.05	7	18.5	233.7	107.4	126.30	0.94	10.0	9.4	UnDef	0.10
13.12	31.8	0.34	1.06	15.39	7	18.5	238.3	109.6	128.76	0.94	10.6	9.9	UnDef	0.12
13.38	24.8	0.19	0.75	25.33	7	18.5	242.9	111.7	131.21	0.93	8.3	7.7	UnDef	0.10
13.62	32.1	0.16	0.50	16.64	7	18.5	247.6	113.9	133.66	0.92	10.7	9.8	UnDef	0.09
13.88	21.0	0.18	0.87	22.96	6	18.0	252.1	116.0	136.11	0.91	8.4	7.6	147.9	0.14
14.12	17.9	0.19	1.06	22.06	6	18.0	256.6	118.1	138.57	0.90	7.2	6.5	122.7	0.13
14.38	11.5	0.12	1.03	29.87	6	18.0	261.1	120.1	141.02	0.89	4.6	4.1	71.0	0.09
14.62	14.4	0.10	0.73	26.36	6	18.0	265.6	122.2	143.47	0.89	5.7	5.1	93.6	0.11
14.88	22.3	0.13	0.60	26.64	7	18.5	270.2	124.3	145.92	0.88	7.4	6.5	UnDef	0.11
15.12	27.1	0.24	0.90	18.32	7	18.5	274.8	126.4	148.38	0.87	9.0	7.9	UnDef	0.12
15.38	22.4	0.19	0.83	18.81	6	18.0	279.4	128.5	150.83	0.86	9.0	7.7	156.9	0.15
15.62	41.5	0.20	0.48	12.29	7	18.5	283.9	130.7	153.28	0.86	13.8	11.8	UnDef	0.10
15.88	35.9	0.22	0.63	28.70	7	18.5	288.6	132.8	155.73	0.85	12.0	10.2	UnDef	0.10
16.12	39.9	0.29	0.72	17.22	7	18.5	293.2	135.0	158.19	0.84	13.3	11.2	UnDef	0.11
16.38	25.8	0.07	0.28	31.83	7	18.5	297.8	137.2	160.64	0.84	8.6	7.2	UnDef	0.00
16.62	33.8	0.12	0.37	22.58	7	18.5	302.4	139.3	163.09	0.83	11.3	9.3	UnDef	0.00
16.88	19.8	0.09	0.47	26.55	7	18.5	307.1	141.5	165.54	0.82	6.6	5.4	UnDef	0.13
17.12	49.5	0.32	0.65	14.83	8	19.0	311.8	143.8	168.00	0.82	12.4	10.1	UnDef	0.11

Interpretation Output - Release 1.00.17

Run No: 99-0331-0827-2069

Job No: 99-141

Client: Thurber Engineering

Project: Trout Lake By-Pass

Site: 99-141 CPT-N2

Location: N.INTERCHANGE

Cone: 20 TON A 058

CPT Date: 99/26/03

CPT Time: 09:54

CPT File: 141CPN2.COR

Northing (m): 0.000

Easting (m): 0.000

Elevation (m): 0.000

Water Table (m): 0.00 (ft): 0.0

Su Nkt used: 12.50

Averaging Increment (m): 0.25

Phi Method: Robertson and Campanella, 1983

Dr Method: Jamiolkowski - All Sands

State Parameter M: 1.20

Used Unit Weights Assigned to Soil Zones

Values of 1.0E9 or UnDef are printed for parameters that are not valid for the material type (SBT)

Depth (m)	k (cm/s)	Bq	Qtn	Rfn (%)	SBTn	Qc1N	DeltaQc1N	Qc1Ncs	Fc (%)	Phi (Deg)	Dr (%)	OCR	State Del(n1)60 (N1) Param	
0.12	1.0E-07	0.00	295.8	1.06	9	5.7	0.0	5.7	4.3	UnDef	UnDef	10.0	UnDef	0.0
0.38	1.0E-07	0.00	128.6	0.73	9	7.5	0.4	7.9	6.9	UnDef	UnDef	10.0	UnDef	0.1
0.62	5.0E-08	-0.01	53.6	2.99	7	5.4	8.2	13.6	27.7	UnDef	UnDef	10.0	UnDef	3.3
0.88	1.0E-07	-0.01	68.4	1.48	7	9.5	4.6	14.1	17.2	UnDef	UnDef	10.0	UnDef	1.2
1.12	5.0E-05	-0.03	178.8	0.30	9	31.5	0.0	31.5	1.6	44	49.3	10.0	-0.14	0.0
1.38	5.0E-04	0.00	290.1	0.21	10	63.3	0.0	63.3	0.0	46	66.2	1.0	-0.15	0.0
1.62	5.0E-03	0.00	361.9	0.36	10	95.1	0.0	95.1	0.0	48	75.1	1.0	-0.21	0.0
1.88	5.0E-03	0.00	441.3	0.38	10	136.2	0.0	136.2	0.0	48	83.1	1.0	-0.24	0.0
2.12	5.0E-03	0.00	407.6	0.52	10	144.7	0.0	144.7	0.1	48	82.8	1.0	-0.26	0.0
2.38	5.0E-03	0.00	366.7	0.60	10	147.2	0.0	147.2	0.9	48	81.6	1.0	-0.26	0.0
2.62	5.0E-03	0.00	318.5	0.56	9	142.7	0.0	142.7	1.1	46	79.1	1.0	-0.24	0.0
2.88	5.0E-03	0.00	314.3	0.45	10	155.3	0.0	155.3	0.5	46	80.2	1.0	-0.22	0.0
3.12	5.0E-03	0.00	354.2	0.53	10	184.6	0.0	184.6	0.6	48	84.9	1.0	-0.25	0.0
3.38	5.0E-02	0.00	347.3	0.46	10	188.8	0.0	188.8	0.2	48	85.5	1.0	-0.23	0.0
3.62	5.0E-02	0.00	434.4	0.36	10	245.5	0.0	245.5	0.0	48	93.0	1.0	-0.23	0.0
3.88	5.0E-02	0.00	382.3	0.37	10	224.3	0.0	224.3	0.0	48	90.4	1.0	-0.22	0.0
4.12	5.0E-03	0.00	177.6	0.83	9	108.5	1.6	110.0	5.5	44	69.6	1.0	-0.22	0.2
4.38	5.0E-05	-0.03	71.5	2.57	7	45.7	39.4	85.2	22.3	40	44.9	10.0	-0.27	7.3
4.62	5.0E-04	-0.03	84.6	1.52	7	55.3	21.0	76.4	15.3	42	50.3	1.0	-0.22	3.8
4.88	5.0E-05	0.02	22.2	0.92	7	15.9	24.6	40.5	27.7	34	30.0	6.0	-0.04	3.8
5.12	5.0E-05	0.13	22.0	0.96	7	16.1	26.5	42.7	28.3	34	30.0	6.0	-0.03	4.0
5.38	5.0E-06	0.18	15.9	1.19	6	12.4	49.4	61.8	36.0	UnDef	UnDef	6.0	UnDef	6.0
5.62	5.0E-06	0.34	14.1	0.71	7	11.4	34.9	46.2	33.3	UnDef	UnDef	6.0	UnDef	5.0
5.88	1.0E-07	0.40	12.1	0.63	7	10.2	40.6	50.8	35.2	UnDef	UnDef	3.0	UnDef	5.0
6.12	1.0E-07	0.42	11.0	0.64	6	9.5	38.1	47.6	37.2	UnDef	UnDef	3.0	UnDef	4.7
6.38	1.0E-07	0.44	10.6	0.67	6	9.4	37.6	47.0	38.4	UnDef	UnDef	3.0	UnDef	4.6
6.62	1.0E-07	0.42	10.3	0.62	6	9.4	37.5	46.9	38.2	UnDef	UnDef	3.0	UnDef	4.6
6.88	1.0E-07	0.46	9.1	0.75	6	8.6	34.4	42.9	42.7	UnDef	UnDef	3.0	UnDef	4.2
7.12	1.0E-07	0.50	9.3	0.69	6	8.9	35.7	44.6	41.4	UnDef	UnDef	3.0	UnDef	4.4
7.38	1.0E-07	0.45	9.4	0.66	6	9.2	36.7	45.8	40.6	UnDef	UnDef	3.0	UnDef	4.5
7.62	1.0E-07	0.41	9.3	0.67	6	9.2	36.8	46.0	41.0	UnDef	UnDef	3.0	UnDef	4.5
7.88	5.0E-06	0.44	8.5	0.97	6	8.7	34.9	43.6	46.7	UnDef	UnDef	3.0	UnDef	4.3
8.12	5.0E-06	0.37	8.7	0.79	6	9.0	36.0	45.1	44.0	UnDef	UnDef	3.0	UnDef	4.4
8.38	1.0E-07	0.42	8.8	0.65	6	9.2	36.7	45.8	42.1	UnDef	UnDef	3.0	UnDef	4.5
8.62	1.0E-07	0.41	8.6	0.64	6	9.2	36.8	46.0	42.3	UnDef	UnDef	3.0	UnDef	4.5
8.88	5.0E-05	0.37	8.7	0.64	6	9.4	37.4	46.8	42.1	30	30.0	3.0	0.12	3.7
9.12	5.0E-05	0.35	8.5	0.62	6	9.3	37.2	46.6	42.3	30	30.0	3.0	0.12	3.6
9.38	5.0E-05	0.30	9.1	0.56	6	10.0	40.0	50.0	39.9	30	30.0	3.0	0.11	3.9
9.62	5.0E-05	0.39	8.6	0.58	6	9.6	38.5	48.2	41.6	30	30.0	3.0	0.13	3.8

Depth (m)	k (cm/s)	Bq	Qtn	Rfn (%)	SBTn	Qc1N	DeltaQc1N	Qc1Ncs	Fc (%)	Phi (Deg)	Dr (%)	OCR	State Del(n1)60 (N1) Param	
9.88	5.0E-05	0.23	10.9	1.27	6	11.8	47.3	59.2	44.4	30	30.0	3.0	0.03	4.6
10.12	5.0E-05	0.11	11.7	1.33	6	12.8	51.1	63.9	43.3	30	30.0	3.0	0.01	5.0
10.38	5.0E-05	0.10	12.4	1.17	6	13.6	54.3	67.9	40.6	32	30.0	3.0	0.01	5.3
10.62	5.0E-05	0.04	17.8	1.38	6	18.8	75.1	93.9	35.5	32	30.0	6.0	-0.04	7.3
10.88	5.0E-05	0.07	11.0	1.17	6	12.6	50.4	63.0	43.0	30	30.0	3.0	0.02	4.9
11.12	5.0E-05	0.20	9.5	1.22	6	11.3	45.1	56.4	46.8	30	30.0	3.0	0.04	4.4
11.38	5.0E-05	0.20	10.7	1.04	6	12.5	50.0	62.5	42.5	30	30.0	3.0	0.04	4.9
11.62	5.0E-04	0.04	20.3	0.56	7	22.1	25.8	47.9	25.2	34	30.0	1.0	0.01	3.7
11.88	5.0E-05	0.07	15.6	1.24	6	17.7	70.8	88.5	36.8	32	30.0	6.0	-0.02	6.9
12.12	5.0E-05	-0.02	18.1	1.08	7	20.4	57.2	77.6	32.6	32	30.0	6.0	-0.03	6.8
12.38	5.0E-05	0.00	17.3	0.94	7	19.8	51.6	71.4	32.1	32	30.0	6.0	-0.02	6.4
12.62	5.0E-04	0.00	18.8	0.62	7	21.5	31.0	52.5	27.1	34	30.0	1.0	0.01	4.1
12.88	5.0E-04	0.00	25.7	0.85	7	28.9	32.4	61.3	24.8	36	31.7	1.0	-0.05	4.7
13.12	5.0E-04	0.01	26.8	1.15	7	30.3	42.4	72.7	26.8	36	33.1	1.0	-0.07	5.7
13.38	5.0E-04	0.05	20.1	0.83	7	23.5	39.3	62.8	28.4	34	30.0	1.0	-0.02	4.9
13.62	5.0E-04	0.01	26.0	0.54	7	30.1	23.1	53.2	21.3	36	32.8	1.0	-0.01	3.7
13.88	5.0E-05	0.05	15.9	0.98	7	19.5	66.8	86.3	34.0	32	30.0	6.0	-0.01	7.2
14.12	5.0E-05	0.05	13.0	1.24	6	16.5	65.9	82.4	40.3	32	30.0	6.0	0.00	6.5
14.38	5.0E-05	0.17	7.4	1.33	6	10.5	41.9	52.4	53.8	30	30.0	3.0	0.06	4.1
14.62	5.0E-05	0.10	9.6	0.90	6	13.0	51.9	64.9	43.2	30	30.0	3.0	0.05	5.1
14.88	5.0E-04	0.06	15.8	0.68	7	20.0	45.0	65.0	30.9	32	30.0	1.0	0.02	5.0
15.12	5.0E-04	0.01	19.2	1.00	7	24.1	53.4	77.5	30.8	34	30.0	1.0	-0.03	6.0
15.38	5.0E-05	0.02	15.3	0.95	6	19.8	73.0	92.8	34.5	32	30.0	6.0	0.00	7.5
15.62	5.0E-04	-0.01	29.6	0.51	7	36.3	22.0	58.2	19.1	36	38.2	1.0	-0.02	3.7
15.88	5.0E-04	0.04	24.9	0.68	7	31.2	30.4	61.6	23.5	34	33.9	1.0	-0.02	4.5
16.12	5.0E-04	0.00	27.4	0.78	7	34.4	32.1	66.5	23.1	36	36.6	1.0	-0.05	4.9
16.38	5.0E-04	0.07	16.6	0.31	7	22.0	0.0	22.0	5.0	32	30.0	1.0	0.07	0.0
16.62	5.0E-04	0.02	22.1	0.40	7	28.6	0.0	28.6	5.0	34	31.4	1.0	0.03	0.0
16.88	5.0E-04	0.06	11.8	0.55	7	16.6	62.1	78.8	34.5	30	30.0	1.0	0.06	5.3
17.12	5.0E-03	0.00	32.3	0.69	7	41.3	27.6	68.9	20.0	36	41.9	1.0	-0.05	3.4

Run No: 99-0331-0827-2553

Job No: 99-141

Client: Thurber Engineering

Project: Trout Lake By-Pass

Site: 99-141 CPT-N3

Location: N.INTERCHANGE

Cone: 20 TON A 058

CPT Date: 99/26/03

CPT Time: 11:48

CPT File: 141CPN3.COR

Northing (m): 0.000

Easting (m): 0.000

Elevation (m): 0.000

Water Table (m): 0.00 (ft): 0.0

Su Nkt used: 12.50

Averaging Increment (m): 0.25

Phi Method : Robertson and Campanella, 1983

Dr Method : Jamiolkowski - All Sands

State Parameter M: 1.20

Used Unit Weights Assigned to Soil Zones

Values of 1.0E9 or UnDef are printed for parameters that are not valid for the material type (SBT)

Depth (m)	AvgQt (bar)	AvgFs (bar)	AvgRf (%)	AvgUd (m)	SBT	U.Wt. (kN/m <sup>3</sup> )	TStress (kPa)	ESTress (kPa)	Ueq (kPa)	Cn	N60 (blows/ft)	(N1)60 (blows/ft)	Su (kPa)	CRR
0.12	1.5	0.03	2.33	-0.04	1	17.5	2.2	1.0	1.23	2.00	0.8	1.5	11.9	0.00
0.38	2.4	0.08	3.23	-1.49	3	17.5	6.6	2.9	3.68	2.00	2.4	4.9	19.1	0.00
0.62	5.0	0.04	0.81	-3.92	1	17.5	10.9	4.8	6.13	2.00	2.5	5.0	39.5	0.00
0.88	18.3	0.05	0.26	1.00	7	18.5	15.4	6.9	8.58	2.00	6.1	12.2	UnDef	0.08
1.12	48.1	0.15	0.32	0.82	8	19.0	20.1	9.1	11.04	2.00	12.0	24.1	UnDef	0.16
1.38	66.5	0.32	0.48	1.05	8	19.0	24.9	11.4	13.49	2.00	16.6	33.2	UnDef	0.30
1.62	68.9	0.30	0.44	1.54	8	19.0	29.6	13.7	15.94	2.00	17.2	34.4	UnDef	0.32
1.88	75.8	0.32	0.43	1.69	8	19.0	34.4	16.0	18.39	2.00	18.9	37.9	UnDef	0.40
2.12	76.5	0.41	0.54	3.19	8	19.0	39.1	18.3	20.85	2.00	19.1	38.3	UnDef	0.41
2.38	80.5	0.39	0.48	2.96	8	19.0	43.9	20.6	23.30	2.00	20.1	40.3	UnDef	0.00
2.62	85.7	0.48	0.56	3.66	8	19.0	48.6	22.9	25.75	2.00	21.4	42.8	UnDef	0.00
2.88	123.5	0.62	0.50	1.88	9	19.5	53.4	25.2	28.20	1.95	24.7	48.1	UnDef	0.00
3.12	105.8	0.69	0.65	3.44	9	19.5	58.3	27.7	30.66	1.86	21.2	39.4	UnDef	0.00
3.38	100.3	0.52	0.52	3.50	9	19.5	63.2	30.1	33.11	1.78	20.1	35.8	UnDef	0.00
3.62	111.1	0.39	0.35	0.71	9	19.5	68.1	32.5	35.56	1.72	22.2	38.2	UnDef	0.00
3.88	68.0	0.40	0.59	0.61	8	19.0	72.9	34.9	38.01	1.66	17.0	28.2	UnDef	0.22
4.12	17.7	0.26	1.45	8.62	6	18.0	77.5	37.0	40.47	1.61	7.1	11.4	135.6	0.09
4.38	9.7	0.04	0.44	3.20	6	18.0	82.0	39.1	42.92	1.57	3.9	6.1	71.1	0.00
4.62	7.0	0.04	0.64	15.96	1	17.5	86.4	41.1	45.37	1.53	3.5	5.4	49.2	0.09
4.88	6.3	0.05	0.80	28.89	1	17.5	90.8	43.0	47.82	1.49	3.1	4.7	43.0	0.09
5.12	6.1	0.05	0.84	23.79	1	17.5	95.2	44.9	50.28	1.46	3.0	4.4	40.9	0.09
5.38	5.6	0.04	0.75	31.13	1	17.5	99.6	46.8	52.73	1.43	2.8	4.0	36.8	0.09
5.62	5.3	0.04	0.77	29.90	1	17.5	103.9	48.8	55.18	1.40	2.7	3.7	34.4	0.09
5.88	5.3	0.04	0.78	29.45	1	17.5	108.3	50.7	57.63	1.37	2.6	3.6	33.6	0.08
6.12	6.0	0.04	0.68	32.42	1	17.5	112.7	52.6	60.09	1.35	3.0	4.1	39.1	0.09
6.38	6.4	0.04	0.67	30.95	1	17.5	117.1	54.5	62.54	1.33	3.2	4.3	42.1	0.09
6.62	6.4	0.04	0.63	30.31	1	17.5	121.4	56.4	64.99	1.30	3.2	4.2	41.3	0.09
6.88	6.2	0.05	0.87	30.46	1	17.5	125.8	58.4	67.44	1.28	3.1	4.0	39.6	0.09
7.12	6.0	0.05	0.79	28.64	1	17.5	130.2	60.3	69.90	1.26	3.0	3.8	37.4	0.09
7.38	6.0	0.04	0.66	33.29	1	17.5	134.6	62.2	72.35	1.24	3.0	3.7	37.4	0.09
7.62	6.4	0.04	0.65	32.22	1	17.5	138.9	64.1	74.80	1.22	3.2	3.9	40.3	0.09
7.88	6.9	0.04	0.61	30.72	1	17.5	143.3	66.1	77.25	1.20	3.5	4.2	43.7	0.09
8.12	6.9	0.04	0.62	30.86	1	17.5	147.7	68.0	79.71	1.19	3.5	4.1	43.4	0.09
8.38	7.4	0.05	0.68	29.85	5	18.0	152.1	70.0	82.16	1.17	3.7	4.3	46.6	0.09
8.62	7.1	0.04	0.62	33.56	1	17.5	156.6	72.0	84.61	1.15	3.5	4.1	44.2	0.09
8.88	8.8	0.14	1.53	27.45	5	18.0	161.0	73.9	87.06	1.14	4.4	5.0	57.9	0.09
9.12	10.5	0.16	1.48	11.48	5	18.0	165.5	76.0	89.52	1.12	5.3	5.9	70.9	0.10
9.38	11.3	0.14	1.25	12.94	6	18.0	170.0	78.0	91.97	1.11	4.5	5.0	76.6	0.10
9.62	15.8	0.23	1.48	14.10	6	18.0	174.5	80.1	94.42	1.09	6.3	6.9	112.7	0.14

Depth (m)	AvgQt (bar)	AvgFs (bar)	AvgRf (%)	AvgUd (m)	SBT	U.Wt. (kN/m <sup>3</sup> )	TStress (kPa)	EStress (kPa)	Ueq (kPa)	Cn	N60 (blows/ft)	(N1)60	Su (kPa)	CRR
9.88	11.3	0.19	1.69	12.92	5	18.0	179.0	82.1	96.87	1.08	5.7	6.1	76.3	0.10
10.12	11.4	0.15	1.27	24.62	6	18.0	183.5	84.2	99.33	1.07	4.6	4.9	76.6	0.10
10.38	13.7	0.11	0.81	28.96	6	18.0	188.0	86.2	101.78	1.05	5.5	5.8	94.9	0.12
10.62	18.7	0.15	0.80	10.73	6	18.0	192.5	88.3	104.23	1.04	7.5	7.8	134.0	0.10
10.88	18.6	0.17	0.90	9.12	6	18.0	197.0	90.3	106.68	1.03	7.4	7.7	132.9	0.11
11.12	19.3	0.19	0.97	11.68	6	18.0	201.5	92.4	109.14	1.02	7.7	7.9	138.5	0.11
11.38	15.7	0.13	0.84	19.07	6	18.0	206.0	94.4	111.59	1.01	6.3	6.3	109.3	0.13
11.62	27.4	0.14	0.50	10.41	7	18.5	210.6	96.5	114.04	1.00	9.1	9.1	UnDef	0.09
11.88	33.4	0.18	0.53	15.05	7	18.5	215.2	98.7	116.49	0.99	11.1	11.0	UnDef	0.09
12.12	28.9	0.35	1.21	12.20	7	18.5	219.8	100.9	118.95	0.97	9.6	9.4	UnDef	0.12
12.38	31.5	0.19	0.61	11.85	7	18.5	224.4	103.0	121.40	0.96	10.5	10.1	UnDef	0.10
12.62	32.5	0.20	0.62	7.58	7	18.5	229.1	105.2	123.85	0.95	10.8	10.3	UnDef	0.10
12.88	17.6	0.24	1.35	19.80	6	18.0	233.6	107.3	126.30	0.94	7.0	6.7	122.1	0.14
13.12	17.7	0.21	1.17	22.80	6	18.0	238.1	109.4	128.76	0.94	7.1	6.6	122.2	0.14
13.38	15.0	0.09	0.59	30.20	6	18.0	242.6	111.4	131.21	0.93	6.0	5.6	100.8	0.11
13.62	24.9	0.17	0.67	17.38	7	18.5	247.2	113.5	133.66	0.92	8.3	7.6	UnDef	0.10
13.88	27.1	0.19	0.69	13.64	7	18.5	251.8	115.7	136.11	0.91	9.0	8.2	UnDef	0.10
14.12	41.8	0.14	0.33	10.69	8	19.0	256.5	117.9	138.57	0.90	10.5	9.4	UnDef	0.09
14.38	30.4	0.11	0.37	17.06	7	18.5	261.2	120.2	141.02	0.89	10.1	9.1	UnDef	0.00
14.62	34.5	0.14	0.41	16.21	7	18.5	265.8	122.3	143.47	0.88	11.5	10.2	UnDef	0.08
14.88	36.5	0.46	1.27	15.49	7	18.5	270.4	124.5	145.92	0.88	12.2	10.7	UnDef	0.14



Interpretation Output - Release 1.00.17

Run No: 99-0331-0827-2553

Job No: 99-141

Client: Thurber Engineering

Project: Trout Lake By-Pass

Site: 99-141 CPT-N3

Location: N.INTERCHANGE

Cone: 20 TON A 058

CPT Date: 99/26/03

CPT Time: 11:48

CPT File: 141CPN3.COR

Northing (m): 0.000

Easting (m): 0.000

Elevation (m): 0.000

Water Table (m): 0.00 (ft): 0.0

Su Nkt used: 12.50

Averaging Increment (m): 0.25

Phi Method : Robertson and Campanella, 1983

Dr Method : Jamiolkowski - All Sands

State Parameter M: 1.20

Used Unit Weights Assigned to Soil Zones

Values of 1.0E9 or UnDef are printed for parameters that are not valid for the material type (SBT)

Depth (m)	k (cm/s)	Bq	Qtn	Rfn (%)	SBTn	Qc1N	DeltaQc1N	Qc1Ncs	Fc (%)	Phi (Deg)	Dr (%)	OCR	State Del(n1)60 (N1 Param
0.12	1.0E-07	-0.01	154.3	2.36	7	3.0	1.0	4.0	14.2	UnDef	UnDef	10.0	UnDef 0.3
0.38	5.0E-08	-0.08	82.6	3.32	7	4.9	4.9	9.8	23.8	UnDef	UnDef	10.0	UnDef 2.2
0.62	1.0E-07	-0.09	102.7	0.83	9	10.1	1.3	11.4	9.2	UnDef	UnDef	10.0	UnDef 0.4
0.88	5.0E-04	0.00	265.5	0.26	10	36.7	0.0	36.7	0.0	46	57.1	1.0	-0.16 0.0
1.12	5.0E-03	0.00	527.5	0.32	10	96.3	0.0	96.3	0.0	48	80.7	1.0	-0.24 0.0
1.38	5.0E-03	0.00	581.4	0.48	10	132.9	0.0	132.9	0.0	50	86.7	1.0	-0.28 0.0
1.62	5.0E-03	0.00	501.1	0.44	10	137.7	0.0	137.7	0.0	48	85.1	1.0	-0.26 0.0
1.88	5.0E-03	0.00	471.9	0.43	10	151.5	0.0	151.5	0.0	48	85.6	1.0	-0.25 0.0
2.12	5.0E-03	0.00	416.6	0.54	10	153.1	0.0	153.1	0.2	48	84.0	1.0	-0.26 0.0
2.38	5.0E-03	0.00	389.1	0.48	10	161.0	0.0	161.0	0.0	48	83.7	1.0	-0.25 0.0
2.62	5.0E-03	0.00	372.5	0.57	10	171.4	0.0	171.4	0.7	48	84.0	1.0	-0.26 0.0
2.88	5.0E-02	0.00	487.5	0.50	10	245.9	0.0	245.9	0.0	48	93.1	1.0	-0.27 0.0
3.12	5.0E-02	0.00	380.5	0.66	9	201.2	0.0	201.2	1.1	48	87.3	1.0	-0.27 0.0
3.38	5.0E-02	0.00	331.4	0.52	10	182.9	0.0	182.9	0.8	48	84.6	1.0	-0.24 0.0
3.62	5.0E-02	0.00	339.9	0.35	10	194.9	0.0	194.9	0.0	48	86.4	1.0	-0.21 0.0
3.88	5.0E-03	0.00	193.1	0.60	9	115.2	0.0	115.2	3.5	44	71.3	1.0	-0.20 0.0
4.12	5.0E-05	0.03	45.8	1.52	7	29.1	24.4	53.6	22.1	38	31.9	6.0	-0.15 4.6
4.38	5.0E-05	-0.01	22.8	0.48	7	15.5	0.0	15.5	5.0	34	30.0	6.0	0.01 0.0
4.62	1.0E-07	0.18	15.0	0.73	7	10.9	29.9	40.9	32.4	UnDef	UnDef	6.0	UnDef 4.5
4.88	1.0E-07	0.44	12.5	0.93	6	9.6	38.4	47.9	38.0	UnDef	UnDef	6.0	UnDef 4.7
5.12	1.0E-07	0.36	11.4	1.00	6	9.1	36.2	45.3	40.7	UnDef	UnDef	3.0	UnDef 4.4
5.38	1.0E-07	0.55	9.8	0.91	6	8.2	32.7	40.9	42.8	UnDef	UnDef	3.0	UnDef 4.0
5.62	1.0E-07	0.55	8.8	0.95	6	7.6	30.6	38.2	45.7	UnDef	UnDef	3.0	UnDef 3.7
5.88	1.0E-07	0.55	8.3	0.98	6	7.4	29.7	37.1	47.4	UnDef	UnDef	3.0	UnDef 3.6
6.12	1.0E-07	0.53	9.3	0.84	6	8.3	33.1	41.4	43.3	UnDef	UnDef	3.0	UnDef 4.1
6.38	1.0E-07	0.46	9.7	0.82	6	8.7	34.9	43.6	42.1	UnDef	UnDef	3.0	UnDef 4.3
6.62	1.0E-07	0.45	9.1	0.78	6	8.5	33.9	42.4	42.8	UnDef	UnDef	3.0	UnDef 4.2
6.88	1.0E-07	0.47	8.5	1.09	6	8.1	32.5	40.6	48.1	UnDef	UnDef	3.0	UnDef 4.0
7.12	1.0E-07	0.45	7.8	1.00	6	7.7	30.8	38.5	49.2	UnDef	UnDef	3.0	UnDef 3.8
7.38	1.0E-07	0.54	7.5	0.86	6	7.6	30.5	38.2	48.2	UnDef	UnDef	3.0	UnDef 3.7
7.62	1.0E-07	0.48	7.9	0.83	6	8.0	32.1	40.1	46.9	UnDef	UnDef	3.0	UnDef 3.9
7.88	1.0E-07	0.41	8.3	0.77	6	8.5	34.0	42.5	44.9	UnDef	UnDef	3.0	UnDef 4.2
8.12	1.0E-07	0.41	8.0	0.79	6	8.4	33.5	41.9	46.0	UnDef	UnDef	3.0	UnDef 4.1
8.38	5.0E-06	0.36	8.3	0.86	6	8.8	35.1	43.9	45.9	UnDef	UnDef	3.0	UnDef 4.3
8.62	1.0E-07	0.44	7.7	0.80	6	8.4	33.4	41.8	47.0	UnDef	UnDef	3.0	UnDef 4.1
8.88	5.0E-06	0.25	9.8	1.86	6	10.3	41.2	51.5	51.6	UnDef	UnDef	3.0	UnDef 5.0
9.12	5.0E-06	0.03	11.7	1.76	6	12.1	48.3	60.3	46.9	UnDef	UnDef	3.0	UnDef 5.9
9.38	5.0E-05	0.04	12.3	1.47	6	12.8	51.0	63.8	43.5	32	30.0	3.0	-0.01 5.0
9.62	5.0E-05	0.03	17.6	1.66	6	17.7	70.8	88.5	37.9	32	30.0	6.0	-0.06 6.9

Run No: 99-0331-0827-2553

CPT File: 141CPN3.COR

Depth (m)	k (cm/s)	Bq	Qtn	Rfn (%)	SBTn	Qc1N	DeltaQc1N	Qc1Ncs	Fc (%)	Phi (Deg)	Dr (%)	OCR	State Del(n1)60 (N1 Param	
9.88	5.0E-06	0.03	11.6	2.00	6	12.5	50.0	62.5	48.7	UnDef	UnDef	3.0	UnDef	6.1
10.12	5.0E-05	0.15	11.4	1.51	6	12.4	49.7	62.2	45.5	30	30.0	3.0	0.01	4.9
10.38	5.0E-05	0.15	13.8	0.94	6	14.8	59.2	74.0	36.2	32	30.0	6.0	0.02	5.8
10.62	5.0E-05	0.00	19.0	0.89	7	19.9	39.8	59.7	30.0	34	30.0	6.0	-0.02	5.6
10.88	5.0E-05	-0.01	18.4	1.01	7	19.6	48.2	67.8	31.6	32	30.0	6.0	-0.03	6.1
11.12	5.0E-05	0.00	18.7	1.08	7	20.1	52.1	72.2	32.0	34	30.0	6.0	-0.03	6.5
11.38	5.0E-05	0.06	14.5	0.97	6	16.2	64.7	80.9	35.6	32	30.0	6.0	0.00	6.3
11.62	5.0E-04	0.00	26.2	0.54	7	27.8	21.1	48.9	21.1	36	30.6	1.0	-0.01	3.4
11.88	5.0E-04	0.01	31.6	0.57	7	33.6	19.9	53.5	18.9	36	36.0	1.0	-0.03	3.3
12.12	5.0E-04	0.00	26.5	1.31	7	28.8	47.5	76.3	28.3	36	31.6	1.0	-0.08	6.0
12.38	5.0E-04	0.00	28.4	0.66	7	31.0	24.0	55.1	21.3	36	33.7	1.0	-0.04	3.8
12.62	5.0E-04	-0.02	28.7	0.67	7	31.7	24.5	56.2	21.3	36	34.3	1.0	-0.04	3.9
12.88	5.0E-05	0.04	14.2	1.56	6	17.0	68.0	85.0	41.2	32	30.0	6.0	-0.03	6.7
13.12	5.0E-05	0.06	14.0	1.35	6	16.9	67.5	84.4	39.9	32	30.0	6.0	-0.02	6.6
13.38	5.0E-05	0.13	11.3	0.71	6	14.2	56.9	71.2	37.5	30	30.0	3.0	0.06	5.6
13.62	5.0E-04	0.02	19.7	0.74	7	23.3	36.2	59.5	27.8	34	30.0	1.0	-0.01	4.7
13.88	5.0E-04	0.00	21.3	0.76	7	25.2	35.0	60.2	26.8	34	30.0	1.0	-0.02	4.7
14.12	5.0E-03	-0.01	33.3	0.35	7	38.5	0.0	38.5	5.0	36	39.9	1.0	0.00	0.0
14.38	5.0E-04	0.01	23.1	0.40	7	27.8	0.0	27.8	5.0	34	30.5	1.0	0.02	0.0
14.62	5.0E-04	0.00	26.0	0.44	7	31.2	0.0	31.2	5.0	36	33.9	1.0	0.00	0.0
14.88	5.0E-04	0.00	27.1	1.37	7	32.7	54.5	87.1	28.4	36	35.2	1.0	-0.09	6.8

Run No: 99-0331-0827-3041  
 Job No: 99-141  
 Client: Thurber Engineering  
 Project: Trout Lake By-Pass  
 Site: 99-141 CPT-N4  
 Location: N.INTERCHANGE  
 Cone: 20 TON A 058  
 CPT Date: 99/26/03  
 CPT Time: 14:44  
 CPT File: 141CPN4.COR  
 Northing (m): 0.000  
 Easting (m): 0.000  
 Elevation (m): 0.000

Water Table (m): 0.00 (ft): 0.0  
 Su Nkt used: 12.50  
 Averaging Increment (m): 0.25  
 Phi Method: Robertson and Campanella, 1983  
 Dr Method: Jamiolkowski - All Sands  
 State Parameter M: 1.20

Used Unit Weights Assigned to Soil Zones

Values of 1.0E9 or UnDef are printed for parameters that are not valid for the material type (SBT)

Depth (m)	AvgQt (bar)	AvgFs (bar)	AvgRf (%)	AvgUd (m)	SBT	U.Wt. (kN/m <sup>3</sup> )	TStress (kPa)	EStress (kPa)	Ueq (kPa)	Cn	N60 (blows/ft)	(N1)60 (blows/ft)	Su (kPa)	CRR
0.12	15.8	0.03	0.16	1.28	6	18.0	2.2	1.0	1.23	2.00	6.3	12.6	126.1	0.08
0.38	1.1	0.13	12.54	-1.51	2	12.5	6.1	2.4	3.68	2.00	1.1	2.2	8.1	0.00
0.62	1.0	0.16	15.93	-1.50	2	12.5	9.2	3.1	6.13	2.00	1.0	2.0	7.2	0.00
0.88	35.9	0.12	0.34	-3.65	7	18.5	13.1	4.5	8.58	2.00	12.0	24.0	UnDef	0.11
1.12	47.7	0.14	0.30	-8.46	8	19.0	17.8	6.7	11.04	2.00	11.9	23.9	UnDef	0.16
1.38	51.9	0.22	0.43	-2.65	8	19.0	22.5	9.0	13.49	2.00	13.0	26.0	UnDef	0.18
1.62	54.0	0.22	0.41	1.38	8	19.0	27.2	11.3	15.94	2.00	13.5	27.0	UnDef	0.20
1.88	52.5	0.21	0.40	1.75	8	19.0	32.0	13.6	18.39	2.00	13.1	26.3	UnDef	0.19
2.12	56.4	0.28	0.49	2.07	8	19.0	36.8	15.9	20.85	2.00	14.1	28.2	UnDef	0.21
2.38	69.9	0.33	0.47	2.28	8	19.0	41.5	18.2	23.30	2.00	17.5	35.0	UnDef	0.33
2.62	79.4	0.46	0.58	2.92	8	19.0	46.2	20.5	25.75	2.00	19.8	39.7	UnDef	0.45
2.88	72.4	0.47	0.64	3.85	8	19.0	51.0	22.8	28.20	2.00	18.1	36.2	UnDef	0.36
3.12	150.4	0.83	0.55	-2.42	9	19.5	55.8	25.2	30.66	1.95	30.1	58.7	UnDef	0.00
3.38	130.2	0.64	0.49	-7.50	9	19.5	60.7	27.6	33.11	1.86	26.0	48.5	UnDef	0.00
3.62	111.3	0.70	0.63	-5.21	9	19.5	65.6	30.0	35.56	1.79	22.3	39.8	UnDef	0.00
3.88	93.9	0.58	0.62	-0.22	8	19.0	70.4	32.4	38.01	1.72	23.5	40.4	UnDef	0.00
4.12	62.2	0.32	0.52	2.22	8	19.0	75.1	34.7	40.47	1.66	15.6	25.9	UnDef	0.19
4.38	57.8	0.23	0.39	-0.06	8	19.0	79.9	37.0	42.92	1.61	14.4	23.3	UnDef	0.16
4.62	93.0	0.35	0.38	1.48	9	19.5	84.7	39.3	45.37	1.56	18.6	29.0	UnDef	0.38
4.88	239.2	2.48	1.04	-6.28	9	19.5	89.6	41.7	47.82	1.51	47.8	72.5	UnDef	0.00
5.12	187.6	2.95	1.57	-7.89	8	19.0	94.4	44.1	50.28	1.47	46.9	69.1	UnDef	0.00
5.38	116.5	1.93	1.66	-7.94	8	19.0	99.1	46.4	52.73	1.44	29.1	41.9	UnDef	0.00
5.62	73.3	1.71	2.34	-8.05	7	18.5	103.8	48.6	55.18	1.40	24.4	34.3	UnDef	0.34
5.88	38.8	0.68	1.74	-3.92	7	18.5	108.4	50.8	57.63	1.37	12.9	17.8	UnDef	0.13
6.12	26.4	0.41	1.55	-2.52	6	18.0	113.0	52.9	60.09	1.35	10.5	14.2	201.8	0.11
6.38	34.8	0.28	0.79	-4.36	7	18.5	117.6	55.0	62.54	1.32	11.6	15.3	UnDef	0.10
6.62	25.0	0.28	1.12	-3.46	6	18.0	122.1	57.1	64.99	1.29	10.0	13.0	190.4	0.10
6.88	19.6	0.23	1.15	-0.13	6	18.0	126.6	59.2	67.44	1.27	7.8	10.0	146.5	0.10
7.12	22.2	0.28	1.27	1.46	6	18.0	131.1	61.2	69.90	1.25	8.9	11.1	167.1	0.10
7.38	34.0	0.29	0.84	-1.66	7	18.5	135.7	63.3	72.35	1.23	11.3	13.9	UnDef	0.10
7.62	13.8	0.23	1.65	4.36	6	18.0	140.2	65.4	74.80	1.21	5.5	6.7	98.8	0.14
7.88	20.8	0.25	1.19	2.95	6	18.0	144.8	67.5	77.25	1.19	8.3	9.9	154.6	0.10
8.12	11.7	0.13	1.12	12.56	6	18.0	149.2	69.5	79.71	1.17	4.7	5.5	81.6	0.11
8.38	10.2	0.11	1.10	14.91	5	18.0	153.8	71.6	82.16	1.16	5.1	5.9	69.1	0.10
8.62	8.1	0.11	1.39	23.73	5	18.0	158.2	73.6	84.61	1.14	4.0	4.6	51.8	0.09
8.88	7.1	0.09	1.33	27.53	5	18.0	162.8	75.7	87.06	1.12	3.5	4.0	43.6	0.09
9.12	5.9	0.05	0.80	37.74	1	17.5	167.2	77.7	89.52	1.11	2.9	3.3	33.6	0.08
9.38	6.1	0.04	0.73	39.56	1	17.5	171.6	79.6	91.97	1.10	3.0	3.3	34.7	0.08
9.62	5.8	0.03	0.59	41.62	1	17.5	175.9	81.5	94.42	1.08	2.9	3.1	32.1	0.08

Run No: 99-0331-0827-3041

CPT File: 141CPN4.COR

Depth (m)	AvgQt (bar)	AvgFs (bar)	AvgRf (%)	AvgUd (m)	SBT	U.Wt. (kN/m <sup>3</sup> )	TStress (kPa)	EStress (kPa)	Ueq (kPa)	Cn	N60 (blows/ft)	(N1)60 (blows/ft)	Su (kPa)	CRR
9.88	6.1	0.04	0.65	41.49	1	17.5	180.3	83.4	96.87	1.07	3.1	3.3	34.5	0.08
10.12	6.3	0.03	0.52	40.52	1	17.5	184.7	85.4	99.33	1.06	3.1	3.3	35.6	0.08
10.38	6.2	0.03	0.50	45.80	1	17.5	189.1	87.3	101.78	1.05	3.1	3.2	34.4	0.08
10.62	6.3	0.04	0.61	43.99	1	17.5	193.4	89.2	104.23	1.04	3.2	3.3	35.3	0.08
10.88	5.8	0.03	0.52	47.45	1	17.5	197.8	91.1	106.68	1.03	2.9	3.0	30.3	0.08
11.12	5.7	0.03	0.56	44.07	1	17.5	202.2	93.1	109.14	1.01	2.9	2.9	29.6	0.00
11.38	7.7	0.03	0.43	52.01	1	17.5	206.6	95.0	111.59	1.00	3.8	3.9	45.0	0.09
11.62	8.4	0.04	0.50	49.36	6	18.0	211.0	97.0	114.04	0.99	3.4	3.3	50.2	0.09
11.88	8.4	0.04	0.53	50.55	6	18.0	215.5	99.0	116.49	0.98	3.3	3.3	49.7	0.09
12.12	8.6	0.04	0.47	50.96	6	18.0	220.0	101.1	118.95	0.97	3.4	3.3	51.0	0.09
12.38	8.0	0.04	0.48	52.15	6	18.0	224.5	103.1	121.40	0.96	3.2	3.1	46.0	0.09
12.62	8.2	0.04	0.48	52.76	6	18.0	229.0	105.1	123.85	0.95	3.3	3.1	47.0	0.09
12.88	8.7	0.07	0.80	49.72	5	18.0	233.5	107.2	126.30	0.95	4.4	4.1	51.1	0.09
13.12	8.7	0.08	0.92	46.21	5	18.0	238.0	109.2	128.76	0.94	4.3	4.1	50.5	0.09
13.38	8.8	0.06	0.73	47.22	6	18.0	242.5	111.3	131.21	0.93	3.5	3.2	50.7	0.09
13.62	8.6	0.06	0.73	49.71	6	18.0	247.0	113.3	133.66	0.92	3.5	3.2	49.4	0.09
13.88	8.9	0.06	0.64	50.32	6	18.0	251.5	115.4	136.11	0.91	3.5	3.2	50.7	0.09
14.12	9.9	0.07	0.73	50.95	6	18.0	256.0	117.4	138.57	0.90	4.0	3.6	58.9	0.09
14.38	11.7	0.17	1.43	34.76	5	18.0	260.5	119.5	141.02	0.90	5.8	5.2	72.7	0.09
14.62	14.0	0.16	1.17	33.45	6	18.0	265.0	121.5	143.47	0.89	5.6	5.0	90.7	0.10
14.88	11.7	0.16	1.36	40.43	5	18.0	269.5	123.6	145.92	0.88	5.9	5.2	72.2	0.09
15.12	20.2	0.25	1.25	19.73	6	18.0	274.0	125.6	148.38	0.87	8.1	7.1	139.7	0.15
15.38	12.5	0.14	1.15	42.34	6	18.0	278.5	127.7	150.83	0.87	5.0	4.3	77.6	0.10
15.62	13.7	0.14	0.99	44.75	6	18.0	283.0	129.7	153.28	0.86	5.5	4.7	86.9	0.10
15.88	21.4	0.18	0.85	26.89	6	18.0	287.5	131.8	155.73	0.85	8.6	7.3	148.0	0.16
16.12	15.2	0.17	1.11	35.81	6	18.0	292.0	133.8	158.19	0.85	6.1	5.1	98.0	0.11
16.38	14.7	0.12	0.84	39.32	6	18.0	296.5	135.9	160.64	0.84	5.9	4.9	93.6	0.10
16.62	15.1	0.15	0.98	39.04	6	18.0	301.0	137.9	163.09	0.83	6.1	5.0	97.0	0.10
16.88	26.6	0.14	0.51	25.39	7	18.5	305.6	140.0	165.54	0.83	8.9	7.3	UnDef	0.10
17.12	22.4	0.19	0.82	23.74	6	18.0	310.1	142.1	168.00	0.82	9.0	7.4	154.8	0.16
17.38	33.0	0.28	0.84	20.34	7	18.5	314.7	144.2	170.45	0.81	11.0	9.0	UnDef	0.12
17.62	25.0	0.27	1.10	22.24	6	18.0	319.2	146.3	172.90	0.81	10.0	8.1	174.6	0.18
17.88	18.9	0.24	1.27	36.16	6	18.0	323.8	148.4	175.35	0.80	7.5	6.1	124.9	0.12
18.12	18.3	0.16	0.85	38.89	6	18.0	328.2	150.4	177.81	0.80	7.3	5.8	120.1	0.12
18.38	19.7	0.09	0.46	46.81	7	18.5	332.8	152.6	180.26	0.79	6.6	5.2	UnDef	0.13
18.62	35.7	0.20	0.55	33.03	7	18.5	337.4	154.7	182.71	0.79	11.9	9.4	UnDef	0.10
18.88	40.2	0.22	0.55	23.56	7	18.5	342.1	156.9	185.16	0.78	13.4	10.5	UnDef	0.10
19.12	54.7	0.33	0.60	21.38	8	19.0	346.8	159.1	187.62	0.78	13.7	10.6	UnDef	0.11
19.38	55.9	0.55	0.98	19.36	7	18.5	351.4	161.4	190.07	0.77	18.6	14.4	UnDef	0.14
19.62	25.1	0.23	0.92	34.85	7	18.5	356.1	163.5	192.52	0.77	8.4	6.4	UnDef	0.17

Run No: 99-0331-0827-3041  
 Job No: 99-141  
 Client: Thurber Engineering  
 Project: Trout Lake By-Pass  
 Site: 99-141 CPT-N4  
 Location: N.INTERCHANGE  
 Cone: 20 TON A 058  
 CPT Date: 99/26/03  
 CPT Time: 14:44  
 CPT File: 141CPN4.COR  
 Northing (m): 0.000  
 Easting (m): 0.000  
 Elevation (m): 0.000

Water Table (m): 0.00 (ft): 0.0  
 Su Nkt used: 12.50  
 Averaging Increment (m): 0.25  
 Phi Method: Robertson and Campanella, 1983  
 Dr Method: Jamiolkowski - All Sands  
 State Parameter M: 1.20

Used Unit Weights Assigned to Soil Zones

Values of 1.0E9 or UnDef are printed for parameters that are not valid for the material type (SBT)

Depth (m)	k (cm/s)	Bq	Qtn	Rfn (%)	SBTn	Qc1N	DeltaQc1N	Qc1Ncs	Fc (%)	Phi (Deg)	Dr (%)	OCR	State Del(n1)60 (N1) Param	
0.12	5.0E-05	0.01	1000.0	0.17	10	31.6	0.0	31.6	0.0	50	80.0	10.0	-0.23	0.0
0.38	1.0E-15	-0.18	42.6	10.00	1	2.2	UnDef	UnDef	100.0	UnDef	UnDef	6.0	UnDef	UnDef
0.62	1.0E-15	-0.23	29.4	10.00	1	2.0	UnDef	UnDef	100.0	UnDef	UnDef	6.0	UnDef	UnDef
0.88	5.0E-04	-0.01	799.4	0.34	10	71.9	0.0	71.9	0.0	50	82.4	1.0	-0.28	0.0
1.12	5.0E-03	-0.02	708.0	0.30	10	95.4	0.0	95.4	0.0	50	84.8	1.0	-0.26	0.0
1.38	5.0E-03	-0.01	573.5	0.43	10	103.8	0.0	103.8	0.0	50	83.0	1.0	-0.27	0.0
1.62	5.0E-03	0.00	474.8	0.41	10	107.9	0.0	107.9	0.0	48	80.8	1.0	-0.25	0.0
1.88	5.0E-03	0.00	383.7	0.40	10	105.1	0.0	105.1	0.0	48	77.4	1.0	-0.23	0.0
2.12	5.0E-03	0.00	352.5	0.49	10	112.9	0.0	112.9	0.4	48	77.2	1.0	-0.24	0.0
2.38	5.0E-03	0.00	381.8	0.47	10	139.8	0.0	139.8	0.0	48	81.4	1.0	-0.24	0.0
2.62	5.0E-03	0.00	384.9	0.59	10	158.7	0.0	158.7	0.7	48	83.4	1.0	-0.26	0.0
2.88	5.0E-03	0.00	315.5	0.65	9	144.9	0.0	144.9	1.7	46	79.2	1.0	-0.25	0.0
3.12	5.0E-02	0.00	595.5	0.55	10	299.8	0.0	299.8	0.0	50	95.0	1.0	-0.30	0.0
3.38	5.0E-02	-0.01	469.8	0.49	10	247.9	0.0	247.9	0.0	48	93.3	1.0	-0.26	0.0
3.62	5.0E-02	-0.01	368.7	0.63	9	203.1	0.0	203.1	1.1	48	87.6	1.0	-0.26	0.0
3.88	5.0E-03	0.00	288.1	0.62	9	165.1	0.0	165.1	1.9	46	81.6	1.0	-0.24	0.0
4.12	5.0E-03	0.00	177.4	0.53	9	105.7	0.0	105.7	3.4	44	68.9	1.0	-0.18	0.0
4.38	5.0E-03	-0.01	154.2	0.40	9	95.1	0.0	95.1	3.2	44	65.8	1.0	-0.15	0.0
4.62	5.0E-02	0.00	234.4	0.38	9	148.3	0.0	148.3	1.1	46	78.6	1.0	-0.18	0.0
4.88	5.0E-02	0.00	571.0	1.04	9	370.3	0.0	370.3	1.9	50	95.0	1.0	-0.36	0.0
5.12	5.0E-03	-0.01	423.3	1.58	9	282.5	1.6	284.1	5.2	48	95.0	1.0	-0.38	0.2
5.38	5.0E-03	-0.01	249.0	1.67	9	171.1	15.4	186.4	8.1	46	82.7	1.0	-0.34	2.3
5.62	5.0E-04	-0.02	148.5	2.37	7	105.1	35.9	140.9	14.5	44	68.7	1.0	-0.34	6.5
5.88	5.0E-04	-0.03	74.3	1.79	7	54.5	29.4	83.9	18.1	40	49.9	1.0	-0.22	5.0
6.12	5.0E-05	-0.03	47.7	1.62	7	36.2	30.8	67.0	22.2	38	38.2	6.0	-0.16	5.7
6.38	5.0E-04	-0.03	61.2	0.82	9	47.0	14.3	61.3	13.8	40	45.6	1.0	-0.13	2.6
6.62	5.0E-05	-0.04	41.7	1.18	7	33.1	24.5	57.6	20.9	38	35.6	6.0	-0.12	4.7
6.88	5.0E-05	-0.04	30.9	1.23	7	25.4	30.4	55.8	25.4	36	30.0	6.0	-0.10	5.1
7.12	5.0E-05	-0.03	34.1	1.35	7	28.4	32.0	60.4	24.9	38	31.2	6.0	-0.11	5.5
7.38	5.0E-04	-0.03	51.5	0.88	7	42.7	17.9	60.6	16.1	38	42.9	1.0	-0.12	3.2
7.62	5.0E-05	-0.03	18.9	1.84	6	17.0	68.0	85.0	37.8	34	30.0	6.0	-0.08	6.7
7.88	5.0E-05	-0.03	28.6	1.28	7	25.3	35.4	60.7	26.9	36	30.0	6.0	-0.09	5.7
8.12	5.0E-05	0.04	14.7	1.29	6	14.0	56.1	70.1	38.3	32	30.0	6.0	-0.02	5.5
8.38	5.0E-06	0.07	12.1	1.30	6	12.0	48.1	60.2	42.3	UnDef	UnDef	3.0	UnDef	5.9
8.62	5.0E-06	0.23	8.8	1.73	6	9.4	37.5	46.9	53.1	UnDef	UnDef	3.0	UnDef	4.6
8.88	5.0E-06	0.34	7.2	1.73	4	8.1	32.5	40.7	57.9	UnDef	UnDef	3.0	UnDef	4.0
9.12	1.0E-07	0.67	5.4	1.12	4	6.7	26.6	33.3	59.7	UnDef	UnDef	1.5	UnDef	3.3
9.38	1.0E-07	0.68	5.4	1.01	4	6.8	27.1	33.9	58.2	UnDef	UnDef	1.5	UnDef	3.3
9.62	1.0E-07	0.78	4.9	0.85	4	6.4	25.6	32.0	58.8	UnDef	UnDef	1.5	UnDef	3.1

Depth (m)	k (cm/s)	Bq	Qtn	Rfn (%)	SBTn	Qc1N	DeltaQc1N	Qc1Ncs	Fc (%)	Phi (Deg)	Dr (%)	OCR	State Del(n1)60 (N1 Param
9.88	1.0E-07	0.72	5.2	0.93	4	6.7	26.8	33.5	58.6	UnDef	UnDef	1.5	UnDef 3.3
10.12	1.0E-07	0.67	5.2	0.74	6	6.8	27.3	34.1	55.9	UnDef	UnDef	1.5	UnDef 3.3
10.38	1.0E-07	0.81	4.9	0.72	6	6.6	26.5	33.1	57.1	UnDef	UnDef	1.5	UnDef 3.2
10.62	1.0E-07	0.74	4.9	0.88	4	6.7	26.9	33.6	59.2	UnDef	UnDef	1.5	UnDef 3.3
10.88	1.0E-07	0.95	4.2	0.79	4	6.0	24.1	30.2	62.9	UnDef	UnDef	1.5	UnDef 3.0
11.12	1.0E-07	0.87	4.0	0.86	4	5.9	23.7	29.7	65.1	UnDef	UnDef	1.5	UnDef 2.9
11.38	1.0E-07	0.71	5.9	0.59	6	7.9	31.6	39.4	50.3	UnDef	UnDef	1.5	UnDef 3.9
11.62	5.0E-05	0.59	6.5	0.67	6	8.5	34.1	42.6	49.3	30	30.0	3.0	0.18 3.3
11.88	5.0E-05	0.61	6.3	0.71	6	8.4	33.6	42.0	50.7	30	30.0	3.0	0.18 3.3
12.12	5.0E-05	0.60	6.3	0.63	6	8.5	34.1	42.7	49.4	30	30.0	3.0	0.19 3.3
12.38	5.0E-05	0.68	5.6	0.66	6	7.9	31.5	39.4	53.0	30	30.0	1.5	0.21 3.1
12.62	5.0E-05	0.67	5.6	0.66	6	8.0	31.9	39.8	52.9	30	30.0	1.5	0.21 3.1
12.88	5.0E-06	0.57	6.0	1.10	6	8.4	33.7	42.1	56.8	UnDef	UnDef	1.5	UnDef 4.1
13.12	5.0E-06	0.51	5.8	1.27	4	8.3	33.2	41.6	59.5	UnDef	UnDef	1.5	UnDef 4.1
13.38	5.0E-05	0.52	5.7	1.01	6	8.3	33.2	41.5	57.0	30	30.0	1.5	0.15 3.2
13.62	5.0E-05	0.57	5.4	1.02	4	8.1	32.5	40.6	58.3	30	30.0	1.5	0.17 3.2
13.88	5.0E-05	0.56	5.5	0.90	6	8.2	33.0	41.2	56.6	30	30.0	1.5	0.17 3.2
14.12	5.0E-05	0.49	6.3	0.98	6	9.2	36.6	45.8	54.2	30	30.0	3.0	0.14 3.6
14.38	5.0E-06	0.22	7.6	1.84	4	10.7	42.8	53.5	57.4	UnDef	UnDef	3.0	UnDef 5.2
14.62	5.0E-05	0.16	9.3	1.44	6	12.7	50.7	63.4	49.2	30	30.0	3.0	0.03 5.0
14.88	5.0E-06	0.28	7.3	1.76	4	10.5	42.2	52.7	57.8	UnDef	UnDef	3.0	UnDef 5.2
15.12	5.0E-05	0.03	13.9	1.45	6	18.0	72.1	90.1	40.8	32	30.0	6.0	-0.02 7.1
15.38	5.0E-05	0.27	7.6	1.48	6	11.1	44.2	55.3	54.5	30	30.0	3.0	0.06 4.3
15.62	5.0E-05	0.26	8.4	1.24	6	12.0	48.1	60.1	49.9	30	30.0	3.0	0.06 4.7
15.88	5.0E-05	0.06	14.0	0.98	6	18.6	74.5	93.1	36.3	32	30.0	6.0	0.01 7.3
16.12	5.0E-05	0.16	9.2	1.38	6	13.1	52.5	65.6	49.1	30	30.0	3.0	0.04 5.1
16.38	5.0E-05	0.19	8.6	1.05	6	12.6	50.3	62.9	47.3	30	30.0	3.0	0.06 4.9
16.62	5.0E-05	0.18	8.8	1.23	6	12.9	51.6	64.4	48.6	30	30.0	3.0	0.05 5.0
16.88	5.0E-04	0.04	16.8	0.58	7	22.5	37.7	60.2	28.4	32	30.0	1.0	0.02 4.7
17.12	5.0E-05	0.03	13.6	0.96	6	18.8	75.3	94.1	36.7	32	30.0	6.0	0.01 7.4
17.38	5.0E-04	0.01	20.7	0.93	7	27.4	48.5	76.0	28.9	34	30.2	1.0	-0.03 6.0
17.62	5.0E-05	0.02	14.9	1.26	6	20.7	82.7	103.4	37.8	32	30.0	6.0	-0.02 8.1
17.88	5.0E-05	0.11	10.5	1.53	6	15.5	61.9	77.4	47.3	30	30.0	3.0	0.01 6.1
18.12	5.0E-05	0.14	10.0	1.04	6	14.9	59.7	74.6	43.9	30	30.0	3.0	0.04 5.8
18.38	5.0E-04	0.17	10.7	0.56	6	15.9	63.7	79.7	36.5	30	30.0	1.0	0.08 5.2
18.62	5.0E-04	0.04	20.9	0.61	7	28.7	34.1	62.8	25.3	34	31.5	1.0	0.00 4.8
18.88	5.0E-04	0.01	23.4	0.60	7	32.1	31.3	63.4	23.5	34	34.7	1.0	-0.01 4.7
19.12	5.0E-03	0.00	32.2	0.64	7	43.3	27.5	70.8	19.5	36	43.3	1.0	-0.05 3.4
19.38	5.0E-04	0.00	32.5	1.05	7	44.0	41.7	85.7	23.2	36	43.7	1.0	-0.08 6.3
19.62	5.0E-04	0.07	13.2	1.07	6	19.6	78.5	98.2	38.4	32	30.0	1.0	0.01 6.4

Interpretation Output - Release 1.00.17

Run No: 99-0331-0827-3514

Job No: 99-141

Client: Thurber Engineering

Project: Trout Lake By-Pass

Site: 99-141 CPT-N5

Location: N.INTERCHANGE

Cone: 20 TON A 058

CPT Date: 99/26/03

CPT Time: 13:39

CPT File: 141CPN5.COR

Northing (m): 0.000

Easting (m): 0.000

Elevation (m): 0.000

Water Table (m): 0.00 (ft): 0.0

Su Nkt used: 12.50

Averaging Increment (m): 0.25

Phi Method: Robertson and Campanella, 1983

Dr Method: Jamiolkowski - All Sands

State Parameter M: 1.20

Used Unit Weights Assigned to Soil Zones

Values of 1.0E9 or UnDef are printed for parameters that are not valid for the material type (SBT)

Depth (m)	AvgQt (bar)	AvgFs (bar)	AvgRf (%)	AvgUd (m)	SBT	U.Wt. (kN/m <sup>3</sup> )	TStress (kPa)	ESTress (kPa)	Ueq (kPa)	Cn	N60 (blows/ft)	(N1)60	Su (kPa)	CRR
0.12	1.9	0.03	1.59	-0.54	1	17.5	2.2	1.0	1.23	2.00	0.9	1.9	14.9	0.00
0.38	2.4	0.07	3.05	-0.27	3	17.5	6.6	2.9	3.68	2.00	2.4	4.8	18.9	0.00
0.62	8.8	0.09	1.07	0.11	5	18.0	11.0	4.9	6.13	2.00	4.4	8.8	69.6	0.00
0.88	48.7	0.07	0.15	-1.68	8	19.0	15.6	7.0	8.58	2.00	12.2	24.3	UnDef	0.17
1.12	62.5	0.18	0.28	0.94	8	19.0	20.4	9.3	11.04	2.00	15.6	31.3	UnDef	0.26
1.38	78.0	0.25	0.31	1.22	8	19.0	25.1	11.6	13.49	2.00	19.5	39.0	UnDef	0.43
1.62	72.6	0.26	0.36	1.59	8	19.0	29.9	13.9	15.94	2.00	18.2	36.3	UnDef	0.36
1.88	55.6	0.23	0.41	2.01	8	19.0	34.6	16.2	18.39	2.00	13.9	27.8	UnDef	0.21
2.12	23.8	0.37	1.57	7.91	6	18.0	39.2	18.4	20.85	2.00	9.5	19.0	187.3	0.10
2.38	19.2	0.11	0.56	22.25	6	18.0	43.8	20.5	23.30	2.00	7.7	15.3	149.9	0.09
2.62	23.0	0.18	0.78	33.78	7	18.5	48.3	22.6	25.75	2.00	7.7	15.3	UnDef	0.09
2.88	23.4	0.16	0.68	49.33	7	18.5	52.9	24.7	28.20	1.97	7.8	15.3	UnDef	0.09
3.12	20.3	0.17	0.84	41.05	6	18.0	57.5	26.8	30.66	1.89	8.1	15.3	157.7	0.09
3.38	14.2	0.13	0.89	38.18	6	18.0	62.0	28.9	33.11	1.82	5.7	10.3	108.7	0.09
3.62	9.7	0.05	0.51	39.62	6	18.0	66.5	30.9	35.56	1.76	3.9	6.8	72.5	0.00
3.88	8.5	0.05	0.55	40.35	6	18.0	71.0	33.0	38.01	1.70	3.4	5.8	62.2	0.00
4.12	7.6	0.03	0.42	32.28	1	17.5	75.4	35.0	40.47	1.65	3.8	6.3	55.1	0.00
4.38	7.8	0.03	0.39	36.35	1	17.5	79.8	36.9	42.92	1.61	3.9	6.3	55.9	0.00
4.62	8.0	0.03	0.39	32.83	1	17.5	84.2	38.8	45.37	1.57	4.0	6.3	57.2	0.00
4.88	7.9	0.03	0.40	26.94	1	17.5	88.6	40.7	47.82	1.53	4.0	6.1	56.2	0.00
5.12	6.3	0.03	0.47	30.71	1	17.5	92.9	42.7	50.28	1.50	3.2	4.7	43.1	0.09
5.38	7.9	0.04	0.52	24.53	6	18.0	97.4	44.6	52.73	1.46	3.2	4.6	55.4	0.08
5.62	7.4	0.03	0.39	29.91	1	17.5	101.8	46.6	55.18	1.43	3.7	5.3	51.3	0.00
5.88	8.9	0.08	0.89	19.86	5	18.0	106.2	48.6	57.63	1.40	4.5	6.3	62.8	0.10
6.12	8.7	0.05	0.54	24.53	6	18.0	110.8	50.7	60.09	1.37	3.5	4.8	61.0	0.09
6.38	13.6	0.18	1.35	1.52	6	18.0	115.2	52.7	62.54	1.35	5.4	7.3	99.2	0.10
6.62	9.2	0.12	1.32	13.56	5	18.0	119.8	54.8	64.99	1.32	4.6	6.1	63.8	0.10
6.88	17.3	0.25	1.46	5.48	6	18.0	124.2	56.8	67.44	1.30	6.9	9.0	128.5	0.11
7.12	10.2	0.12	1.16	21.35	5	18.0	128.8	58.9	69.90	1.28	5.1	6.5	71.6	0.11
7.38	10.3	0.10	0.98	21.14	6	18.0	133.2	60.9	72.35	1.25	4.1	5.2	72.0	0.11
7.62	20.0	0.10	0.50	18.39	7	18.5	137.8	63.0	74.80	1.23	6.7	8.2	UnDef	0.09
7.88	21.2	0.16	0.78	-0.05	6	18.0	142.4	65.1	77.25	1.21	8.5	10.3	158.0	0.09
8.12	16.8	0.16	0.92	5.58	6	18.0	146.9	67.2	79.71	1.19	6.7	8.0	122.7	0.09
8.38	21.0	0.11	0.54	6.18	7	18.5	151.4	69.3	82.16	1.18	7.0	8.2	UnDef	0.09
8.62	17.1	0.12	0.70	14.13	6	18.0	156.0	71.4	84.61	1.16	6.8	7.9	124.2	0.09
8.88	25.7	0.11	0.45	6.15	7	18.5	160.6	73.5	87.06	1.14	8.6	9.8	UnDef	0.08
9.12	31.8	0.06	0.18	12.26	7	18.5	165.2	75.7	89.52	1.13	10.6	11.9	UnDef	0.08
9.38	28.9	0.24	0.81	12.74	7	18.5	169.8	77.8	91.97	1.11	9.6	10.7	UnDef	0.10
9.62	33.3	0.20	0.60	6.75	7	18.5	174.4	80.0	94.42	1.09	11.1	12.1	UnDef	0.09

Run No: 99-0331-0827-3514

CPT File: 141CPN5.COR

Depth (m)	AvgQt (bar)	AvgFs (bar)	AvgRf (%)	AvgUd (m)	SBT	U.Wt. (kN/m <sup>3</sup> )	TStress (kPa)	EStress (kPa)	Ueq (kPa)	Cn	N60 (blows/ft)	(N1)60 (blows/ft)	Su (kPa)	CRR
9.88	44.1	0.15	0.34	-2.83	8	19.0	179.1	82.3	96.87	1.08	11.0	11.9	UnDef	0.09
10.12	21.9	0.17	0.80	7.49	6	18.0	183.8	84.4	99.33	1.07	8.8	9.3	160.6	0.10
10.38	16.8	0.28	1.69	12.46	6	18.0	188.2	86.5	101.78	1.05	6.7	7.1	119.7	0.15
10.62	11.5	0.09	0.78	22.97	6	18.0	192.8	88.5	104.23	1.04	4.6	4.8	76.9	0.10
10.88	26.1	0.19	0.74	9.58	7	18.5	197.3	90.6	106.68	1.03	8.7	9.0	UnDef	0.10
11.12	21.9	0.29	1.33	0.69	6	18.0	201.9	92.7	109.14	1.02	8.8	8.9	158.9	0.15
11.38	32.2	0.22	0.68	-1.11	7	18.5	206.4	94.8	111.59	1.00	10.7	10.8	UnDef	0.10
11.62	28.7	0.09	0.33	14.28	7	18.5	211.1	97.0	114.04	0.99	9.6	9.5	UnDef	0.00



Run No: 99-0331-0827-3514  
 Job No: 99-141  
 Client: Thurber Engineering  
 Project: Trout Lake By-Pass  
 Site: 99-141 CPT-N5  
 Location: N.INTERCHANGE  
 Cone: 20 TON A 058  
 CPT Date: 99/26/03  
 CPT Time: 13:39  
 CPT File: 141CPN5.COR  
 Northing (m): 0.000  
 Easting (m): 0.000  
 Elevation (m): 0.000

Water Table (m): 0.00 (ft): 0.0  
 Su Nkt used: 12.50  
 Averaging Increment (m): 0.25  
 Phi Method: Robertson and Campanella, 1983  
 Dr Method: Jamiolkowski - All Sands  
 State Parameter M: 1.20

Used Unit Weights Assigned to Soil Zones

Values of 1.0E9 or UnDef are printed for parameters that are not valid for the material type (SBT)

Depth (m)	k (cm/s)	Bq	Qtn	Rfn (%)	SBTn	Qc1N	DeltaQc1N	Qc1Ncs	Fc (%)	Phi (Deg)	Dr (%)	OCR	State Del(n1)60 (N1 Param
0.12	1.0E-07	-0.04	194.3	1.61	9	3.8	0.5	4.3	9.3	UnDef	UnDef	10.0	UnDef 0.1
0.38	5.0E-08	-0.03	81.8	3.14	7	4.8	4.6	9.4	23.2	UnDef	UnDef	10.0	UnDef 2.1
0.62	5.0E-06	-0.01	178.7	1.08	9	17.6	1.0	18.6	7.0	UnDef	UnDef	10.0	UnDef 0.3
0.88	5.0E-03	-0.01	688.9	0.15	10	97.3	0.0	97.3	0.0	50	84.7	1.0	-0.19 0.0
1.12	5.0E-03	0.00	667.4	0.28	10	125.1	0.0	125.1	0.0	50	87.8	1.0	-0.25 0.0
1.38	5.0E-03	0.00	668.6	0.31	10	156.1	0.0	156.1	0.0	50	91.0	1.0	-0.25 0.0
1.62	5.0E-03	0.00	519.0	0.36	10	145.2	0.0	145.2	0.0	48	86.3	1.0	-0.25 0.0
1.88	5.0E-03	0.00	340.2	0.41	10	111.1	0.0	111.1	0.0	48	76.5	1.0	-0.22 0.0
2.12	5.0E-05	0.02	127.2	1.59	9	47.6	11.3	58.9	12.2	44	50.4	10.0	-0.26 2.6
2.38	5.0E-05	0.10	91.6	0.58	9	38.3	3.5	41.8	8.1	42	42.7	10.0	-0.12 0.8
2.62	5.0E-04	0.14	99.6	0.80	9	45.9	5.8	51.8	9.2	42	46.4	1.0	-0.15 1.2
2.88	5.0E-04	0.20	92.4	0.69	9	46.8	5.6	52.4	9.0	42	45.6	1.0	-0.13 1.1
3.12	5.0E-05	0.19	73.4	0.87	9	39.2	9.6	48.8	12.4	40	40.4	10.0	-0.13 2.2
3.38	5.0E-05	0.25	47.0	0.93	7	26.4	13.2	39.7	17.5	38	30.0	6.0	-0.08 2.7
3.62	5.0E-05	0.39	29.3	0.55	7	17.5	11.4	28.8	19.7	36	30.0	6.0	0.02 2.2
3.88	5.0E-05	0.46	23.6	0.60	7	14.8	14.3	29.1	23.4	34	30.0	6.0	0.04 2.6
4.12	1.0E-07	0.40	19.7	0.46	7	12.9	0.0	12.9	5.0	UnDef	UnDef	6.0	UnDef 0.0
4.38	1.0E-07	0.45	19.0	0.43	7	12.8	0.0	12.8	5.0	UnDef	UnDef	6.0	UnDef 0.0
4.62	1.0E-07	0.39	18.4	0.43	7	12.8	0.0	12.8	5.0	UnDef	UnDef	6.0	UnDef 0.0
4.88	1.0E-07	0.31	17.3	0.46	7	12.4	0.0	12.4	5.0	UnDef	UnDef	6.0	UnDef 0.0
5.12	1.0E-07	0.47	12.6	0.56	7	9.7	30.0	39.7	33.3	UnDef	UnDef	6.0	UnDef 4.2
5.38	5.0E-05	0.27	15.5	0.59	7	11.8	23.9	35.7	30.1	32	30.0	6.0	0.06 3.3
5.62	1.0E-07	0.37	13.8	0.45	7	10.9	0.0	10.9	5.0	UnDef	UnDef	6.0	UnDef 0.0
5.88	5.0E-06	0.17	16.2	1.01	7	12.8	43.6	56.3	34.0	UnDef	UnDef	6.0	UnDef 5.9
6.12	5.0E-05	0.24	15.0	0.62	7	12.3	27.6	39.8	30.9	32	30.0	6.0	0.05 3.7
6.38	5.0E-05	-0.04	23.5	1.48	7	18.7	44.7	63.4	31.4	34	30.0	6.0	-0.08 5.8
6.62	5.0E-06	0.09	14.6	1.52	6	12.4	49.6	62.0	40.4	UnDef	UnDef	6.0	UnDef 6.1
6.88	5.0E-05	-0.01	28.3	1.58	7	23.0	42.0	64.9	29.2	36	30.0	6.0	-0.11 6.1
7.12	5.0E-06	0.16	15.2	1.33	6	13.3	53.4	66.7	38.0	UnDef	UnDef	6.0	UnDef 6.5
7.38	5.0E-05	0.15	14.8	1.12	6	13.2	53.0	66.2	36.7	32	30.0	6.0	0.00 5.2
7.62	5.0E-04	0.06	29.6	0.54	7	25.2	15.8	41.0	19.4	36	30.0	1.0	-0.02 2.6
7.88	5.0E-05	-0.04	30.3	0.84	7	26.2	22.4	48.7	22.3	36	30.0	6.0	-0.06 4.2
8.12	5.0E-05	-0.02	22.8	1.01	7	20.5	33.0	53.5	28.1	34	30.0	6.0	-0.05 5.0
8.38	5.0E-04	-0.01	28.1	0.59	7	25.2	18.2	43.4	20.7	36	30.0	1.0	-0.03 2.9
8.62	5.0E-05	0.03	21.7	0.77	7	20.2	27.2	47.4	26.5	34	30.0	6.0	-0.02 4.4
8.88	5.0E-04	-0.01	32.8	0.48	7	30.0	0.0	30.0	5.0	36	32.8	1.0	-0.03 0.0
9.12	5.0E-04	0.01	39.8	0.19	9	36.5	0.0	36.5	5.0	38	38.4	1.0	0.03 0.0
9.38	5.0E-04	0.01	34.9	0.86	7	32.8	23.4	56.2	20.6	38	35.3	1.0	-0.08 3.8
9.62	5.0E-04	-0.01	39.4	0.63	7	37.2	17.2	54.4	16.9	38	38.9	1.0	-0.06 3.0

Run No: 99-0331-0827-3514

CPT File: 141CPN5.COR

Depth (m)	k (cm/s)	Bq	Qtn	Rfn (%)	SBTn	Qc1N	DeltaQc1N	Qc1Ncs	Fc (%)	Phi (Deg)	Dr (%)	OCR	State Del(n1)60 (N1) Param	
9.88	5.0E-03	-0.03	51.4	0.35	9	48.6	0.0	48.6	5.0	38	46.6	1.0	-0.04	0.0
10.12	5.0E-05	-0.01	23.8	0.87	7	23.8	31.0	54.8	26.2	34	30.0	6.0	-0.04	5.1
10.38	5.0E-05	0.01	17.3	1.90	6	18.1	72.5	90.6	39.8	32	30.0	6.0	-0.07	7.1
10.62	5.0E-05	0.13	10.9	0.94	6	12.3	49.1	61.3	41.0	30	30.0	3.0	0.04	4.8
10.88	5.0E-04	-0.01	26.7	0.80	7	27.5	27.4	54.9	23.7	36	30.2	1.0	-0.05	4.1
11.12	5.0E-05	-0.05	21.4	1.46	7	22.7	66.5	89.2	32.9	34	30.0	6.0	-0.08	7.8
11.38	5.0E-04	-0.04	31.7	0.73	7	33.0	23.6	56.6	20.6	36	35.5	1.0	-0.06	3.8
11.62	5.0E-04	0.01	27.5	0.36	7	29.2	0.0	29.2	5.0	36	32.0	1.0	0.01	0.0

**Thurber Engineering**

## **APPENDIX C**

### **Summary of Dissipations and Pore Pressure Plots**

**ConeTec Investigations Ltd.**

### Summary of Pore Pressure Dissipation Plots

Test CPT - S1

Depth (m)	$c_h(\text{cm}^2/\text{min})$	t-50 (sec)	Uo (m)
2.5	8.8	115	---
4.5	5.2	196	---
6.5	3.4	295	---
8.5	7.6	134	---
10.5	25.0	41	---
13.0	---	---	13.3

Test CPT - S2

Depth (m)	$c_h(\text{cm}^2/\text{min})$	t-50 (sec)	Uo(m)
3.5	8.3	121	---
5.5	2.4	415	---
6.5	4.5	225	---
8.5	4.8	213	---
9.75	---	---	9.8

Test CPT - S3

Depth (m)	$c_h(\text{cm}^2/\text{min})$	t-50 (sec)	Uo(m)
3.5	13.2	77	---
5.5	3.5	290	---
7.7	2.7	860	---
9.5	5.1	200	---
11.5	17.6	58	---
16.93	---	---	17.1

Test CPT - S4

Depth (m)	$c_h(\text{cm}^2/\text{min})$	t-50 (sec)	Uo(m)
3.5	3.5	290	---
5.5	2.5	398	---
7.5	2.6	383	---
9.5	2.4	423	---
11.3	11.3	90	---

Test CPT - S5

Depth (m)	$c_h(\text{cm}^2/\text{min})$	t-50 (sec)	Uo(m)
7.0	11.2	91	---
15.0	6.4	159	---
22.08	---	---	22.4

Test CPT - N1

Depth (m)	$c_h(\text{cm}^2/\text{min})$	t-50 (sec)	Uo(m)
5.0	3.2	316	---
7.0	4.9	205	---
9.0	27.0	38	---
12.1	---	---	12.4

Test CPT - N2

Depth (m)	$c_h(\text{cm}^2/\text{min})$	t-50 (sec)	Uo(m)
8.0	1.8	560	---
10.0	4.0	253	---
12.0	12.7	80	---
16.0	110	9	---
17.3	---	---	17.2

Test CPT - N3

Depth (m)	$c_h(\text{cm}^2/\text{min})$	t-50 (sec)	Uo(m)
5.0	1.4	703	---
7.0	1.7	603	---
12.0	68.0	15	---
15.12	---	---	15.1

Test CPT - N4

Depth (m)	$c_h(\text{cm}^2/\text{min})$	t-50 (sec)	Uo(m)
7.5	4.4	230	---
10.0	1.9	534	---
13.0	2.5	403	---
20.6	---	---	21.0

Test CPT - N5

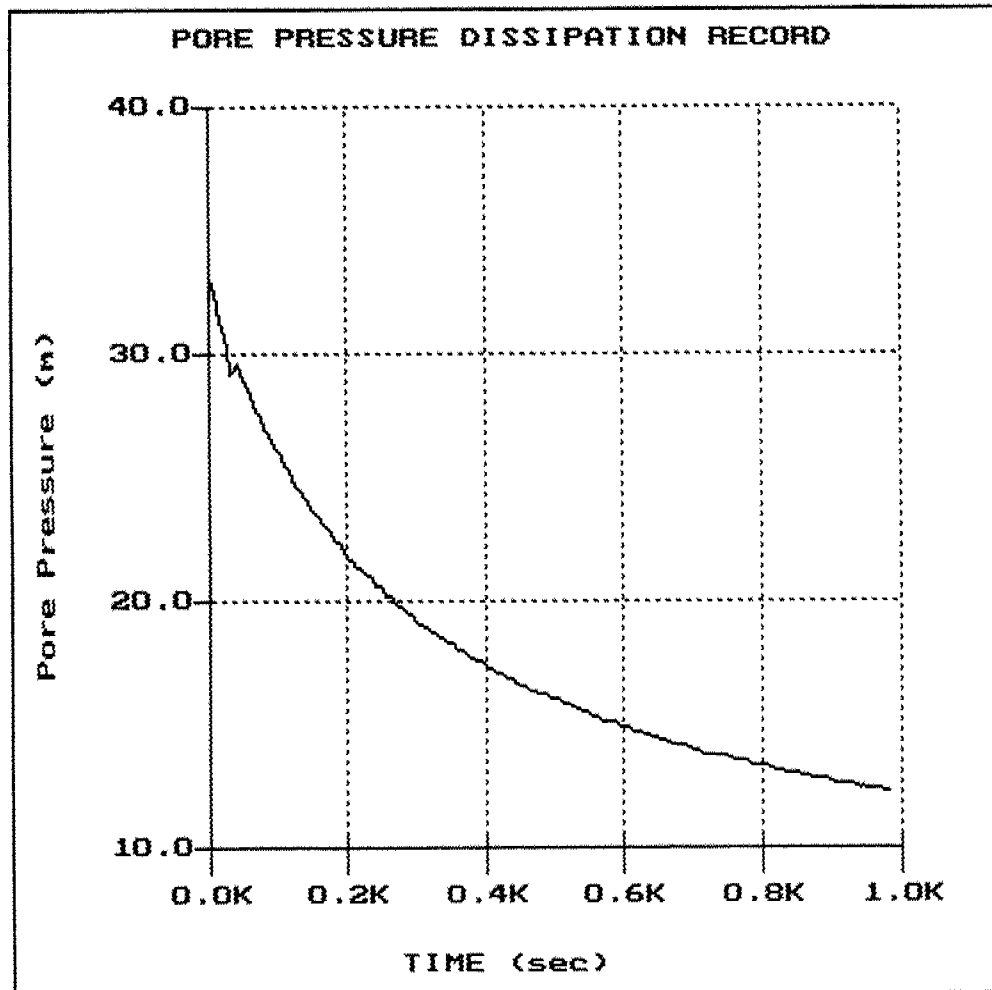
Depth (m)	$c_h(\text{cm}^2/\text{min})$	t-50 (sec)	Uo(m)
4.0	2.7	373	---
9.0	60.3	17	---
12.55	---	---	13.0

Thurber Engineering

Hole: CPTUN1  
Location: N.INTERCHANGE

Cone: 20 TON A 058  
Date: 03:26:99 08:19

File: 141CPN1.PPD  
Depth (m): 5.00  
(ft): 16.40  
Duration: 980.0s

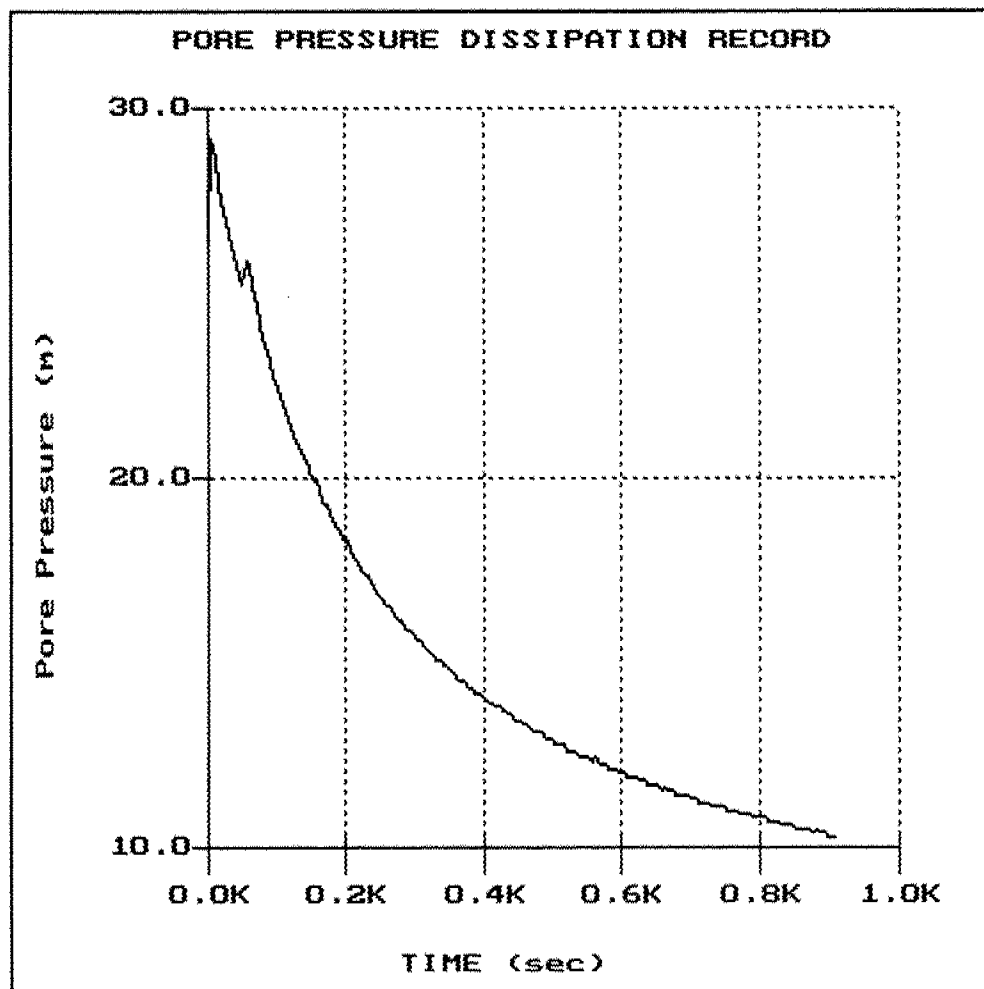


Thurber Engineering

Hole: CPTUN1  
Location: N.INTERCHANGE

Cone: 20 TON A 058  
Date: 03:26:99 08:19

File: 141CPN1.PPD  
Depth (m): 7.00  
(ft): 22.97  
Duration : 905.0s



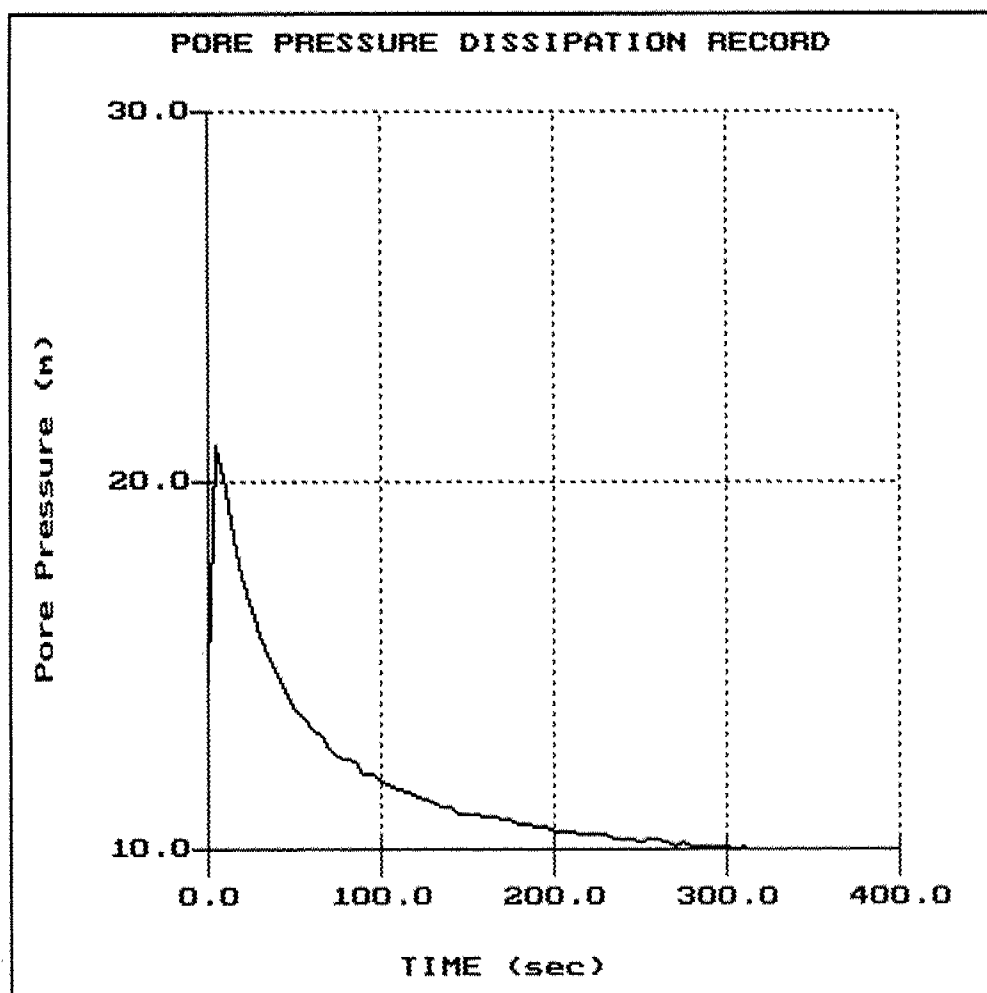


Thurber Engineering

Hole: CPTUN1  
Location: N. INTERCHANGE

Cone: 20 TON A 058  
Date: 03:26:99 08:19

File: 141CPN1.PPD  
Depth (m): 9.00  
(ft): 29.53  
Duration : 310.0s



Thurber Engineering

Hole: CPTUN1

Location: N.INTERCHANGE

Cone: 20 TON A 058

Date: 03:26:99 08:19

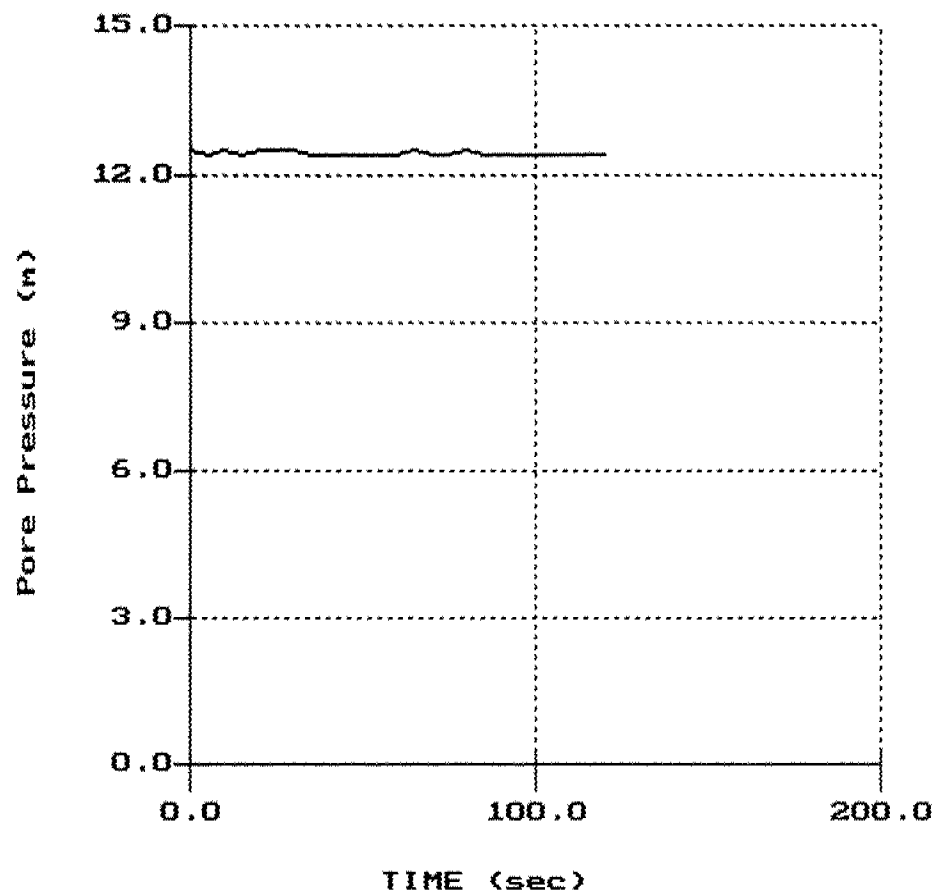
File: 141CPN1.PPD

Depth (m): 12.10

(ft): 39.70

Duration: 120.0s

PORE PRESSURE DISSIPATION RECORD

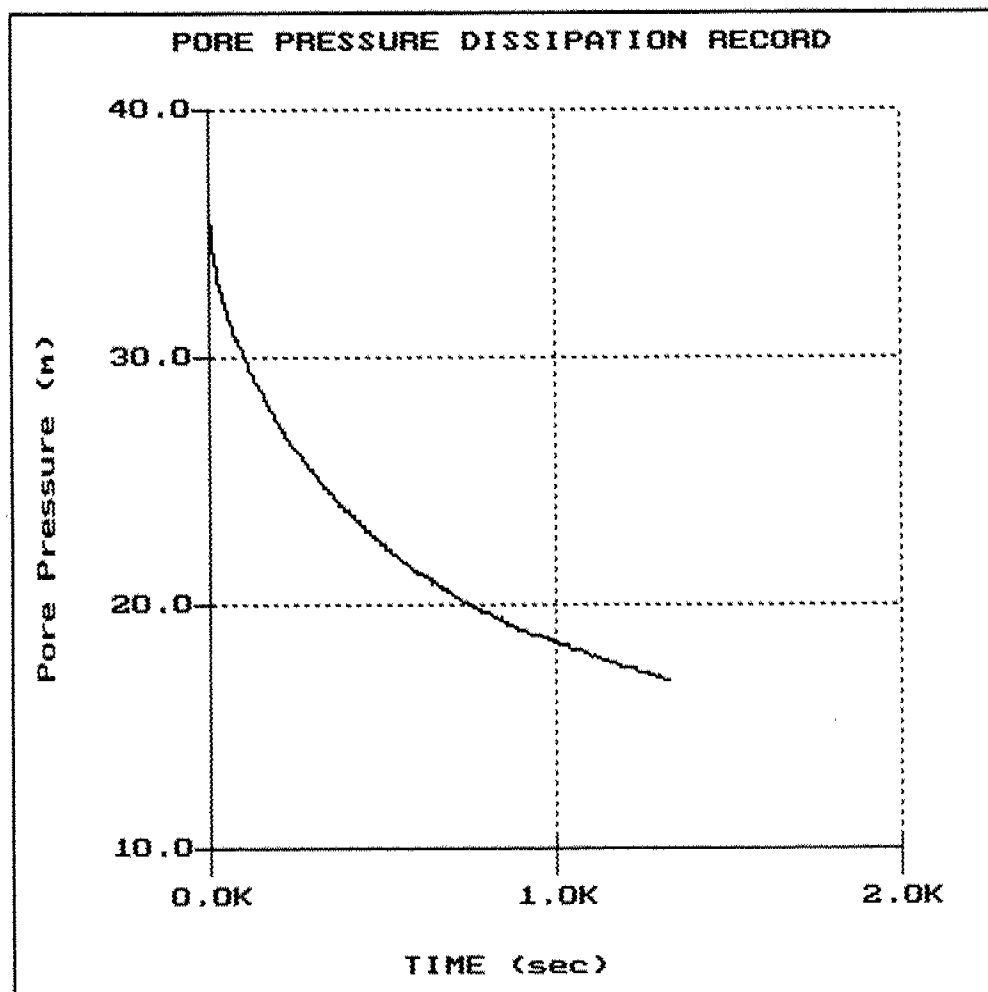


Thurber Engineering

Hole: CPTUN2  
Location: N. INTERCHANGE

Cone: 20 TON A 058  
Date: 03:26:99 09:54

File: 141CPN2.PPD  
Depth (m): 8.00  
(ft): 26.25  
Duration: 1325.0s



Thurber Engineering

Hole: CPTUN2

Location: N. INTERCHANGE

Cone: 20 TON A 058

Date: 03:26:99 09:54

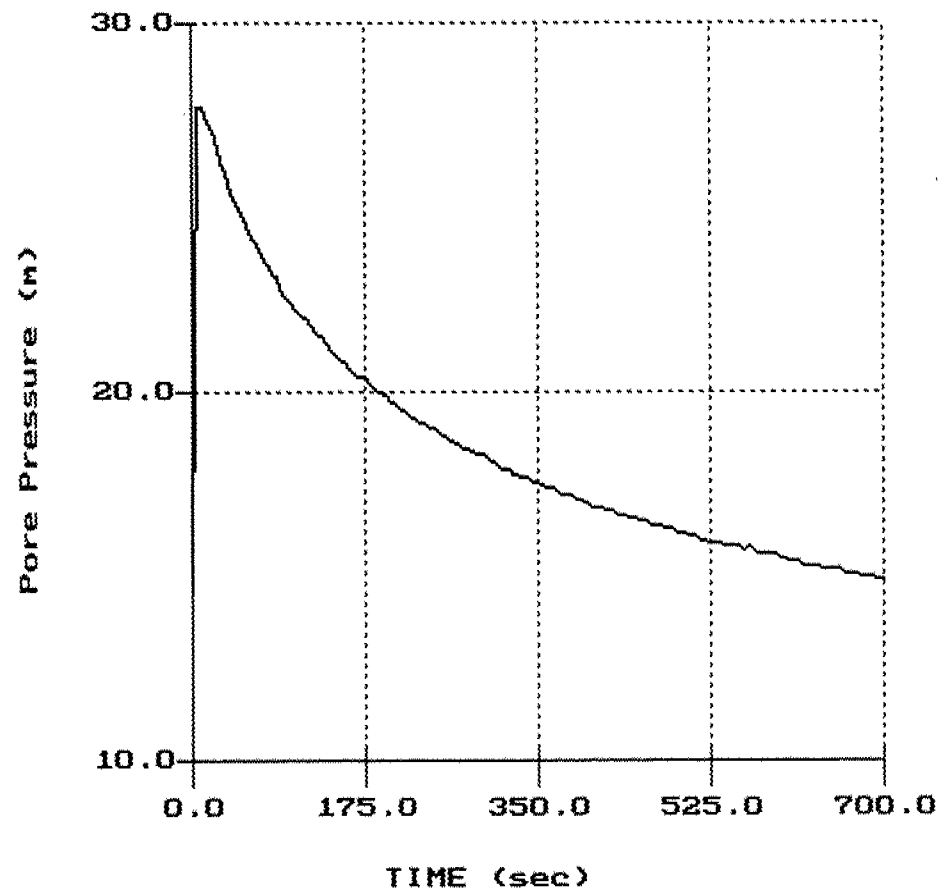
File: 141CPN2.PPD

Depth (m): 10.00

(ft): 32.81

Duration: 700.0s

PORE PRESSURE DISSIPATION RECORD

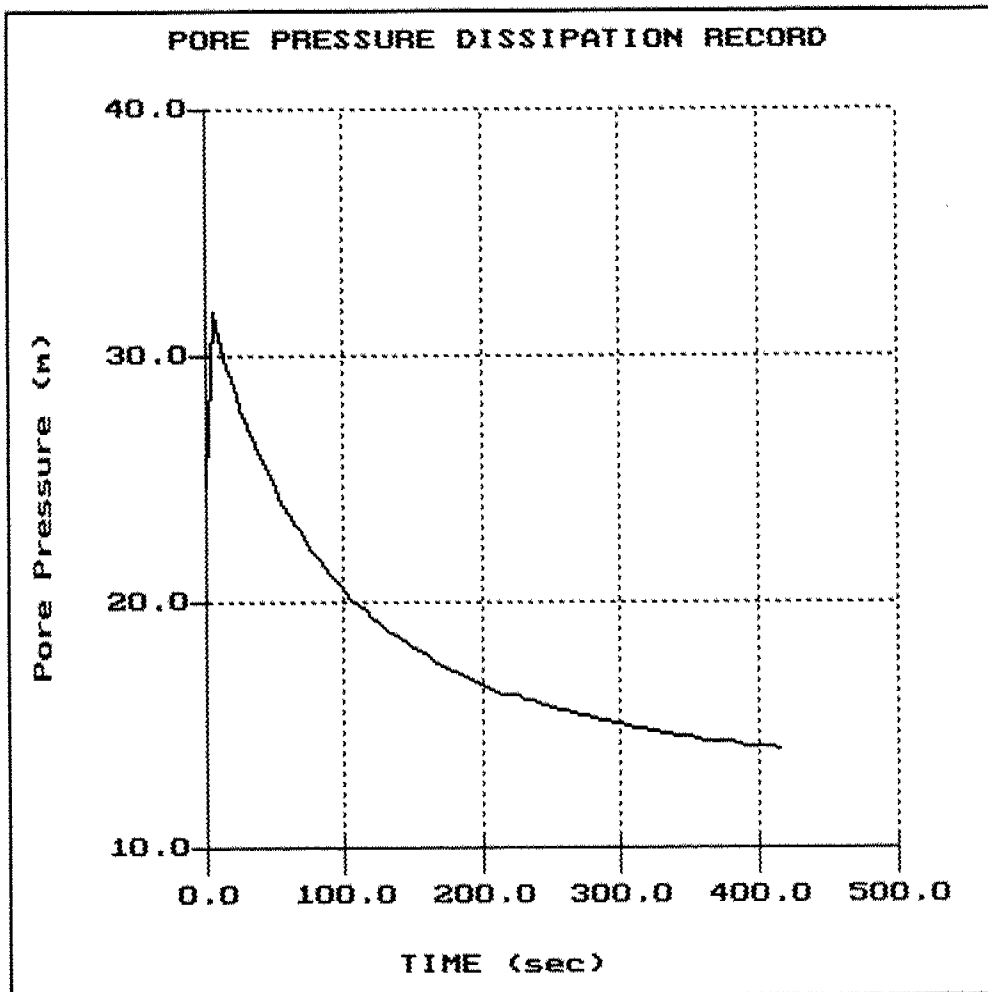


Thurber Engineering

Hole: CPTUN2  
Location: N. INTERCHANGE

Cone: 20 TON A 058  
Date: 03:26:99 09:54

File: 141CPN2.PPD  
Depth (m): 12.00  
(ft): 39.37  
Duration: 415.0s



Thurber Engineering

Hole: CPTUN2

Location: N.INTERCHANGE

Cone: 20 TON A 058

Date: 03:26:99 09:54

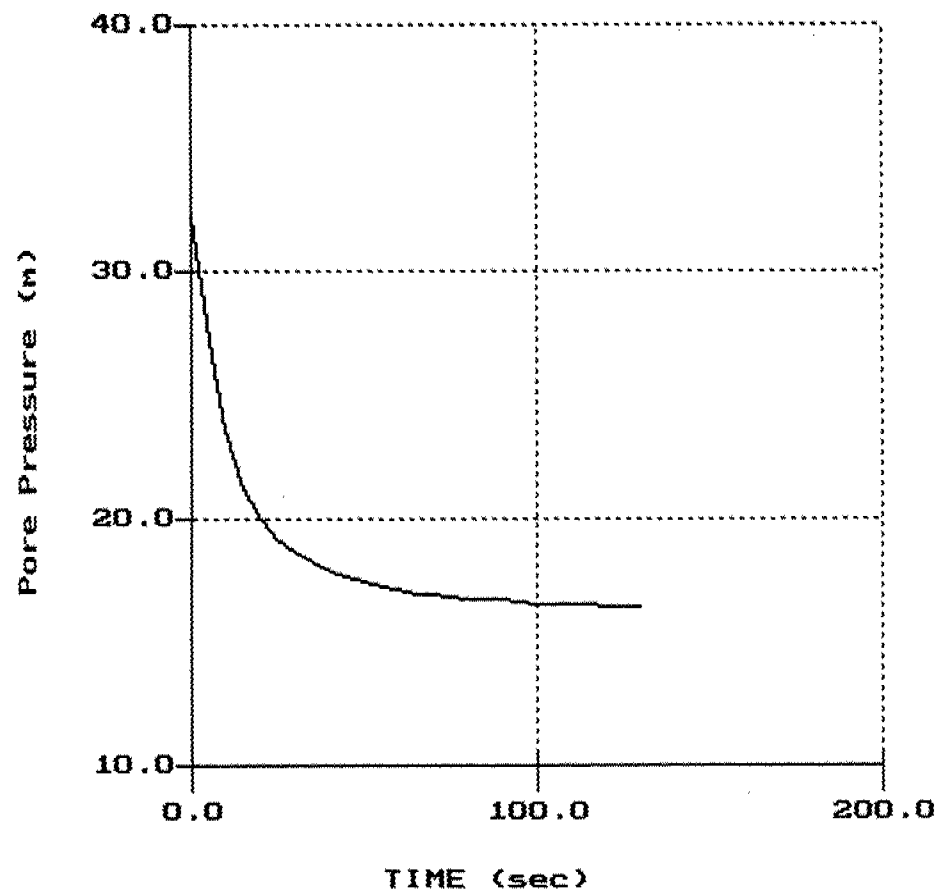
File: 141CPN2.PPD

Depth (m): 16.00

(ft): 52.49

Duration: 130.0s

PORE PRESSURE DISSIPATION RECORD

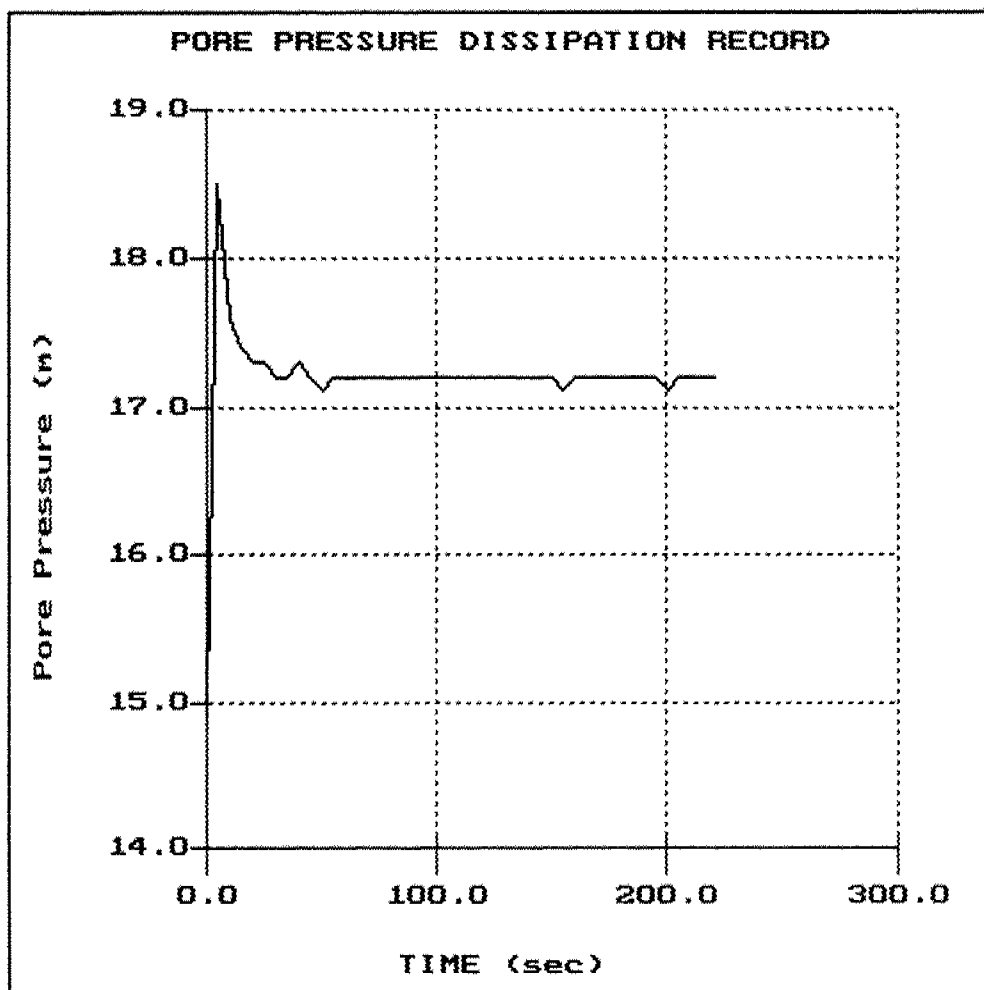


Thurber Engineering

Hole: CPTUN2  
Location: N.INTERCHANGE

Cone: 20 TON A 058  
Date: 03:26:99 09:54

File: 141CPN2.PPD  
Depth (m): 17.33  
(ft): 56.86  
Duration: 220.0s



Thurber Engineering

Hole: CPTUN3

Location: N.INTERCHANGE

Cone: 20 TON A 058

Date: 03:26:99 11:48

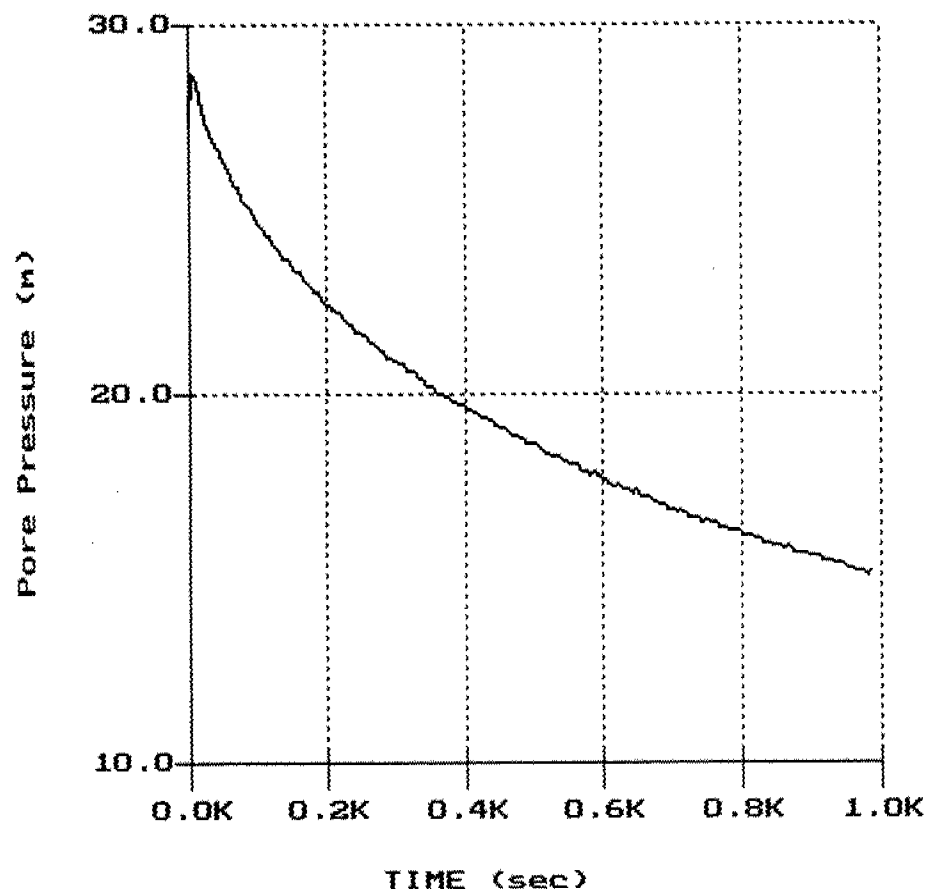
File: 141CPN3.PPD

Depth (m): 5.00

(ft): 16.40

Duration: 985.0s

PORE PRESSURE DISSIPATION RECORD





Thurber Engineering

Hole: CPTUN3

Location: N.INTERCHANGE

Cone: 20 TON A 058

Date: 03:26:99 11:48

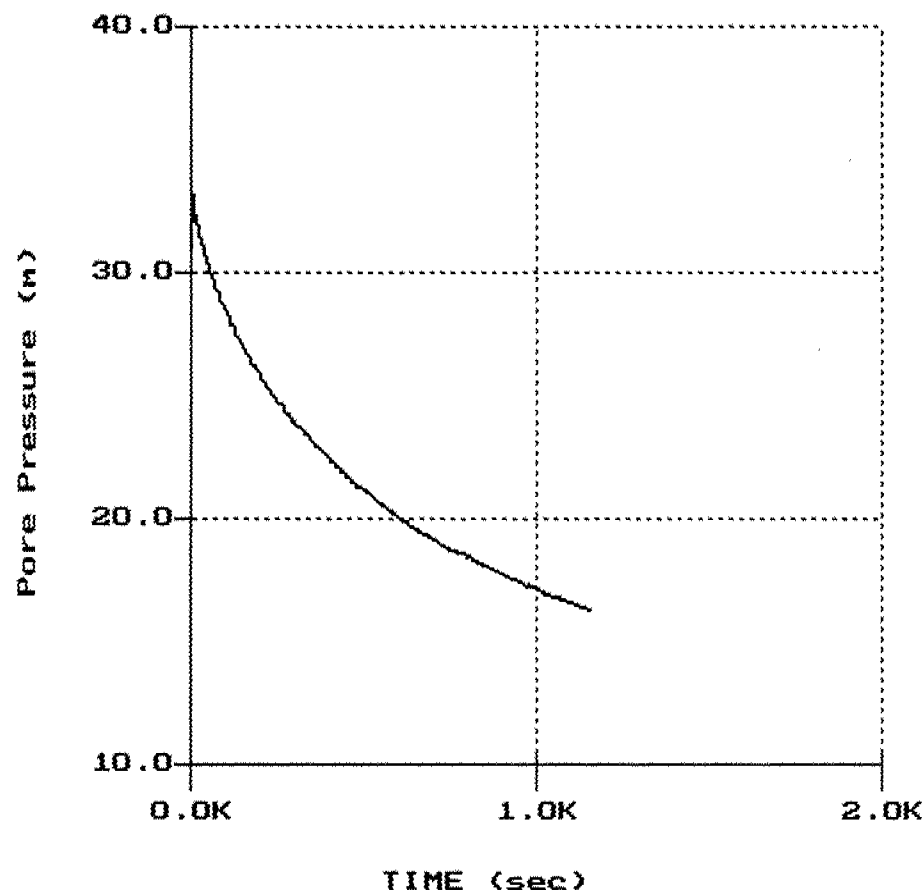
File: 141CPN3.PPD

Depth (m): 7.00

(ft): 22.97

Duration: 1155.0s

PORE PRESSURE DISSIPATION RECORD



Thurber Engineering

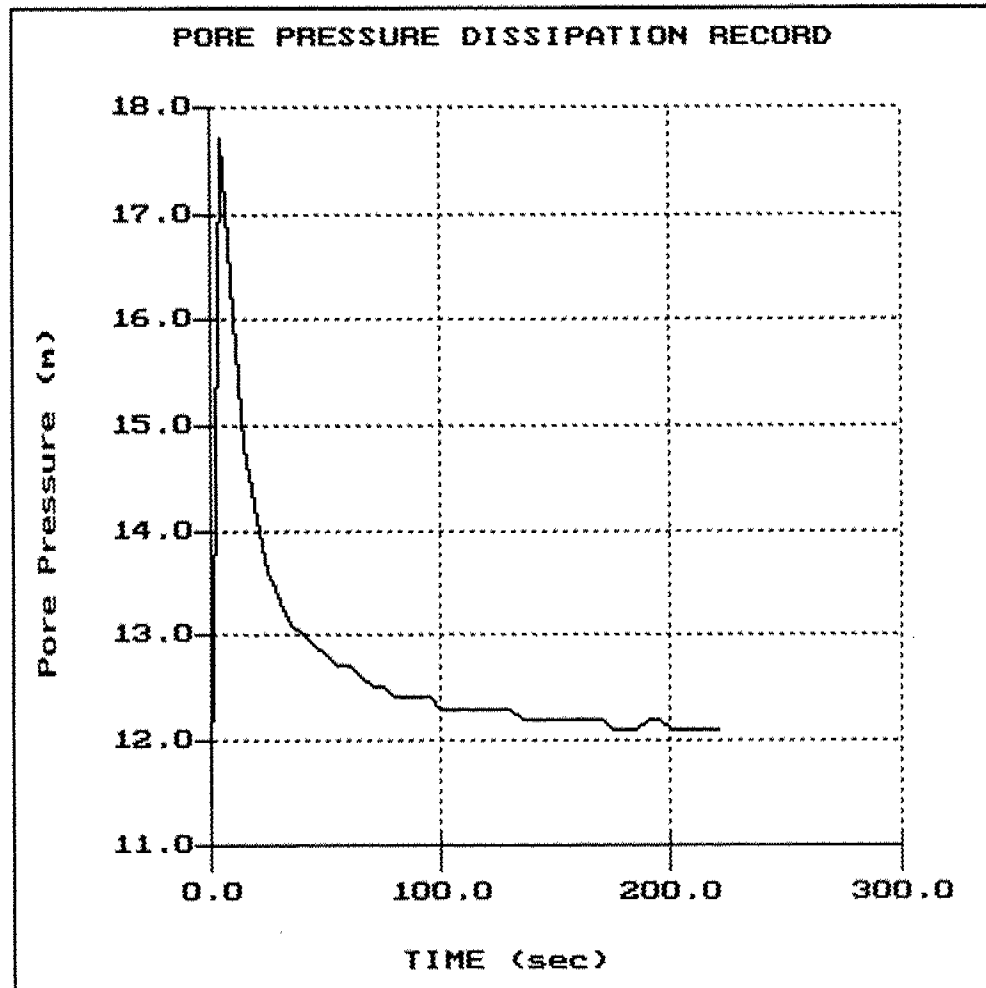
Hole: CPTUN3

Location: N. INTERCHANGE

Cone: 20 TON A 058

Date: 03:26:99 11:48

File: 141CPN3.PPD  
Depth (m): 12.00  
(ft): 39.37  
Duration: 220.0s



Thurber Engineering

Hole: CPTUN3

Location: N.INTERCHANGE

Cone: 20 TON A 058

Date: 03/26/99 11:48

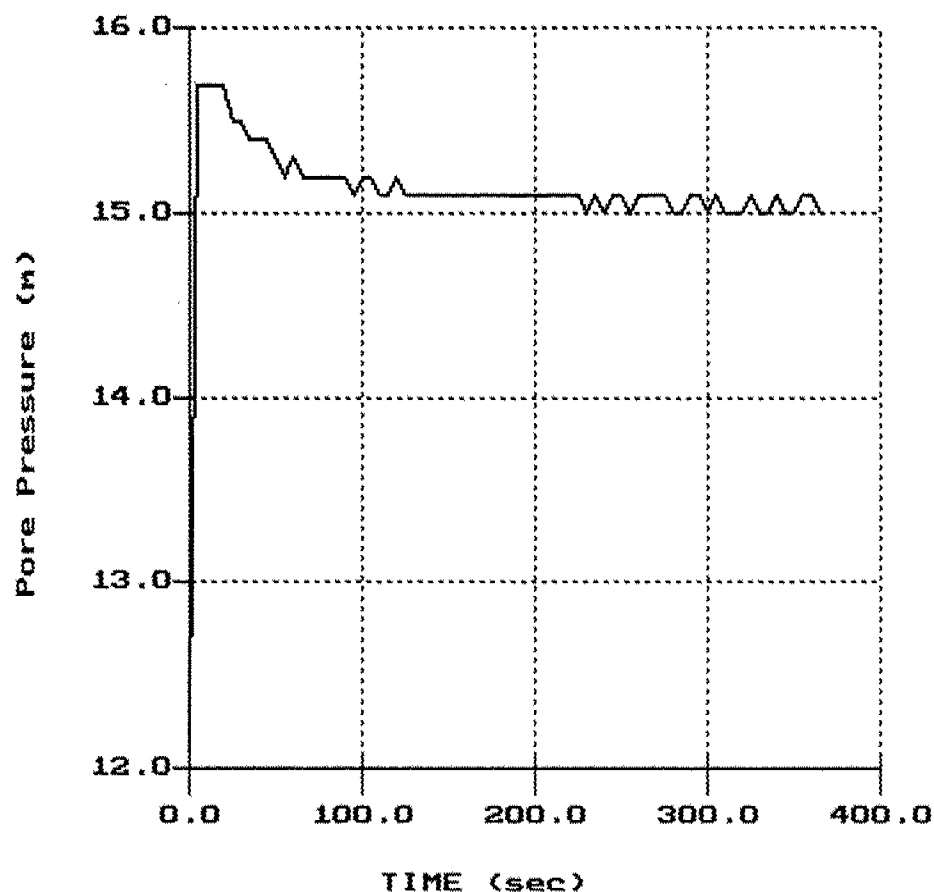
File: 141CPN3.PPD

Depth (m): 15.12

(ft): 49.61

Duration: 365.0s

PORE PRESSURE DISSIPATION RECORD



Thurber Engineering

Hole: CPTUN4

Location: N.INTERCHANGE

Cone: 20 TON A 058

Date: 03:26:99 14:44

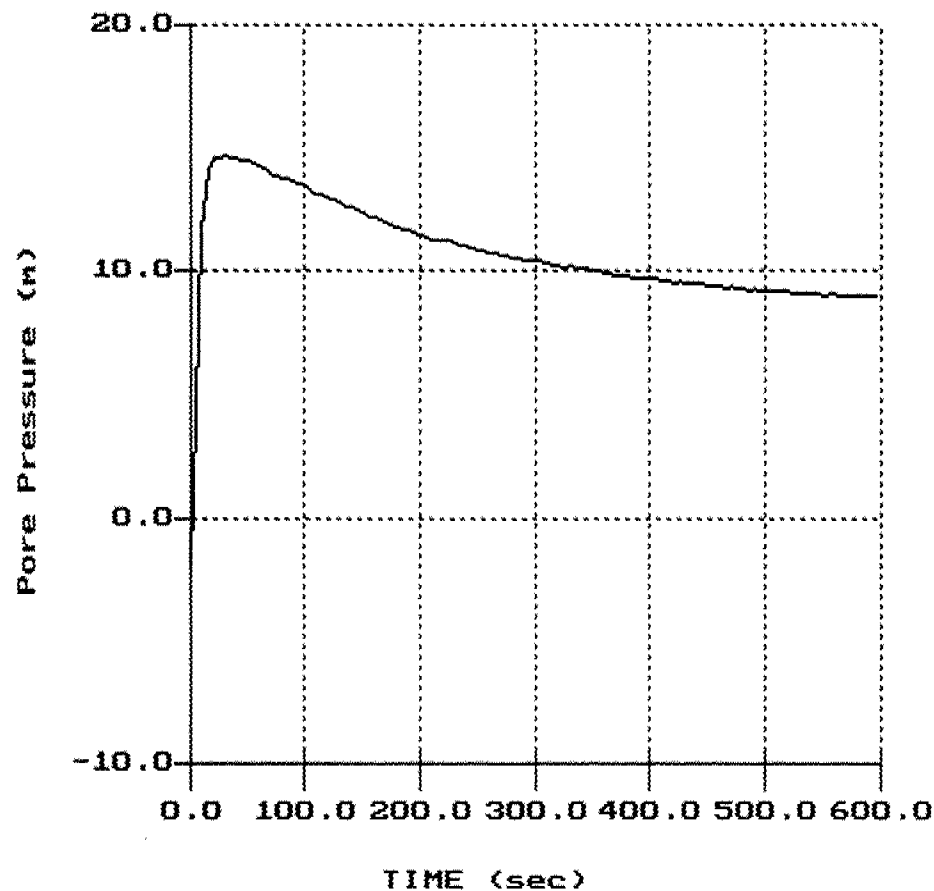
File: 141CPN4.PPD

Depth (m): 7.50

(ft): 24.61

Duration: 595.0s

PORE PRESSURE DISSIPATION RECORD

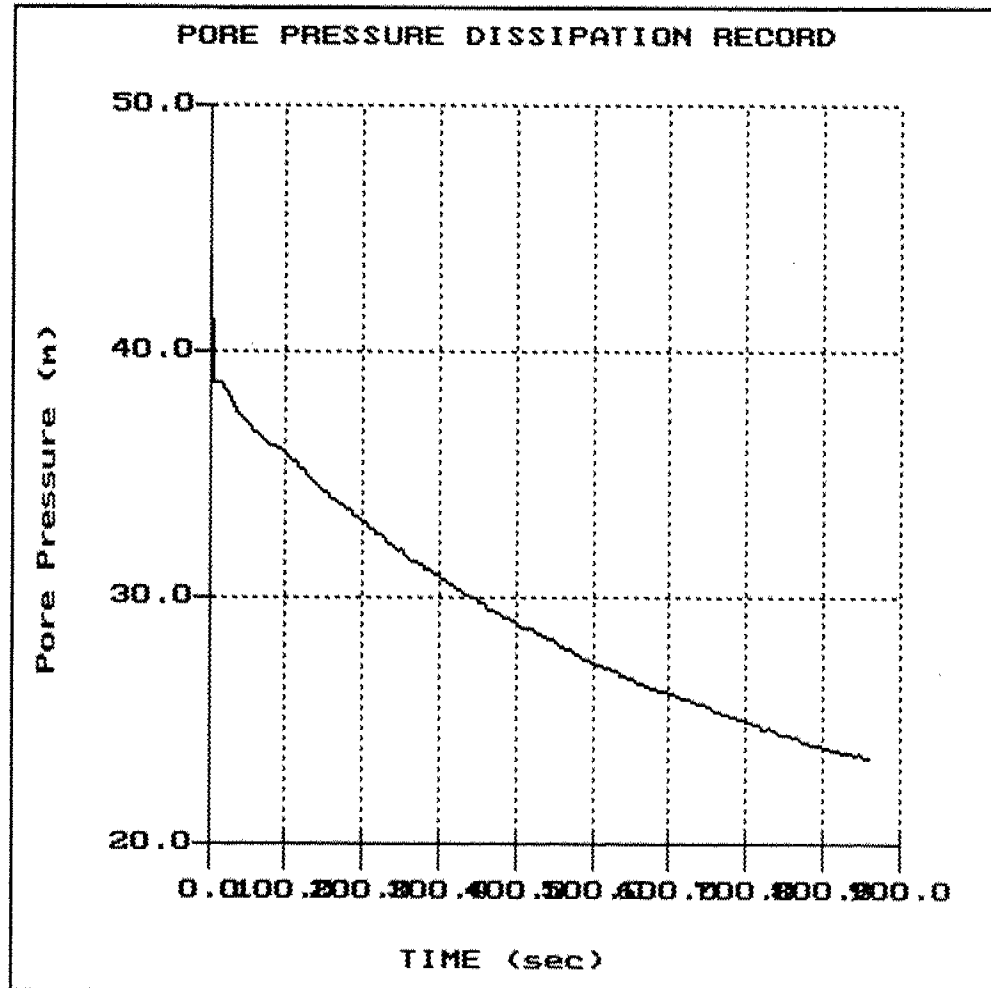


Thurber Engineering

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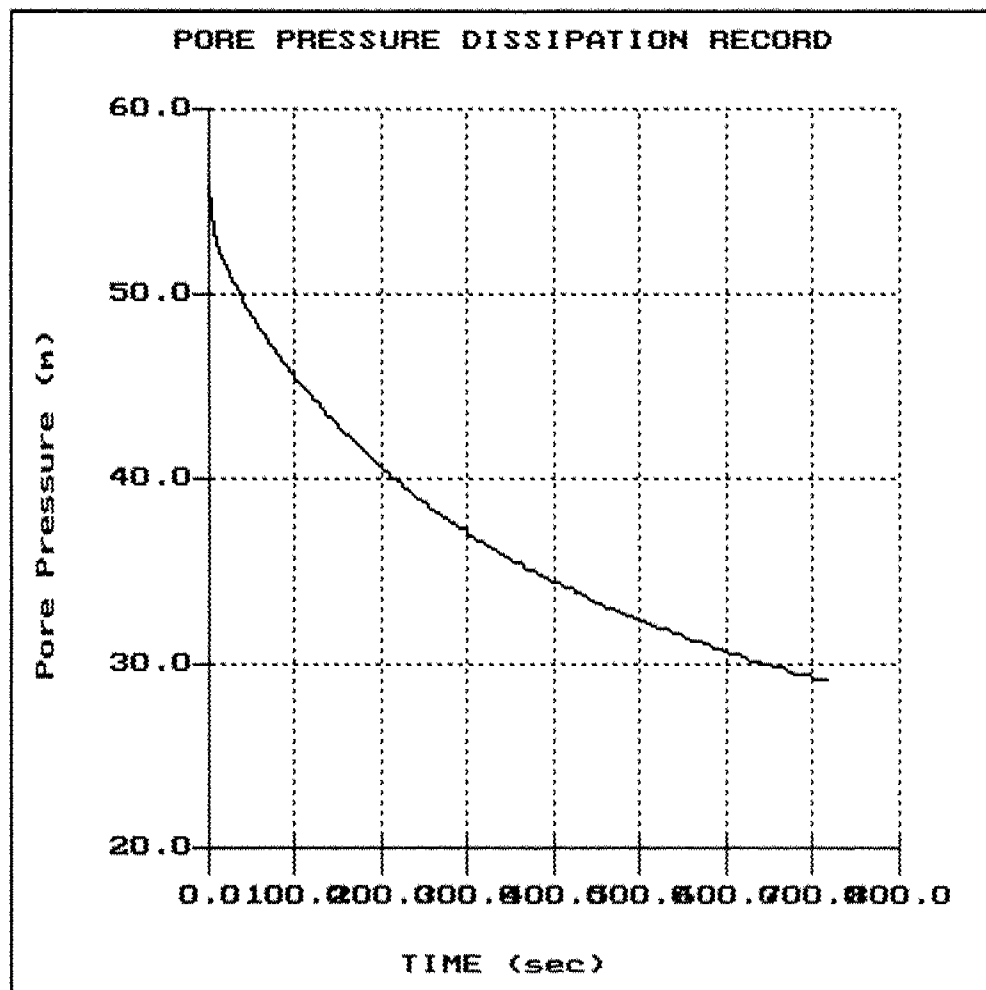


Thurber Engineering

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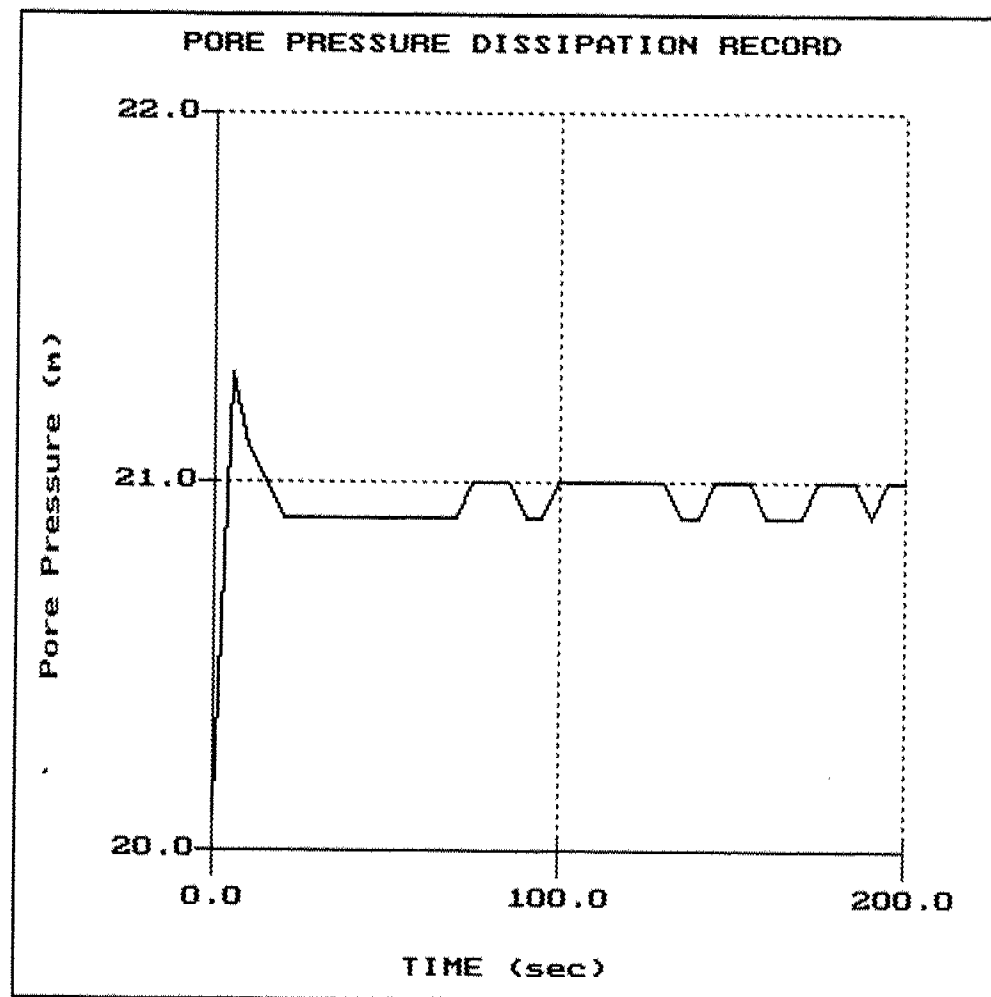


Thurber Engineering

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Thurber Engineering

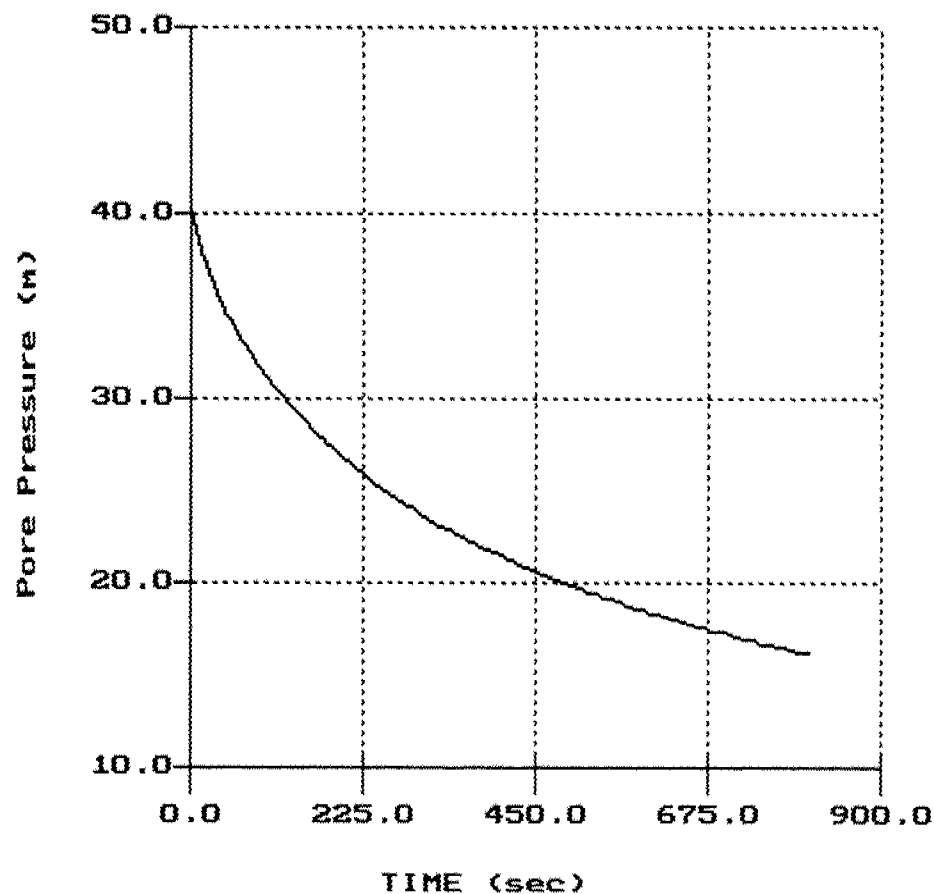
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Location: N.INTERCHANGE

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Date: 03:26:99 13:39

PORE PRESSURE DISSIPATION RECORD



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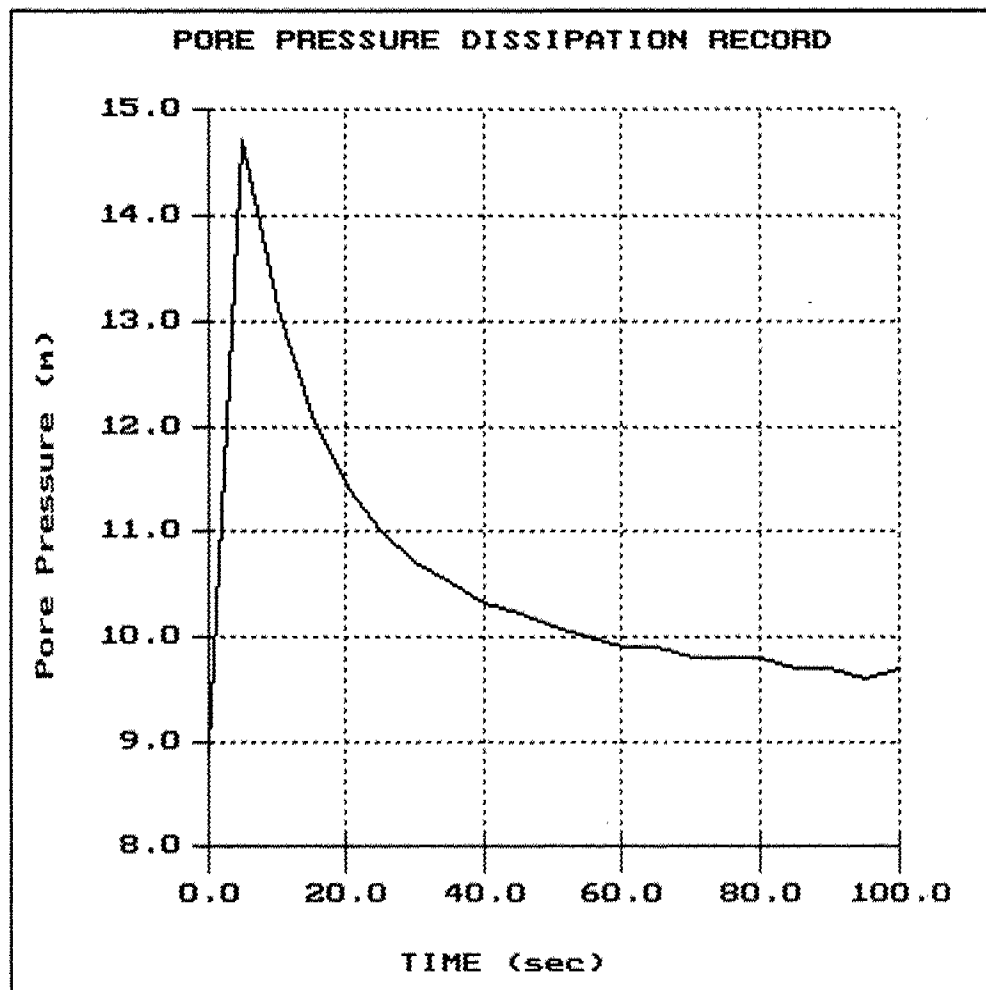


Thurber Engineering

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(ft): 29.53  
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Thurber Engineering

Hole: CPTUN5

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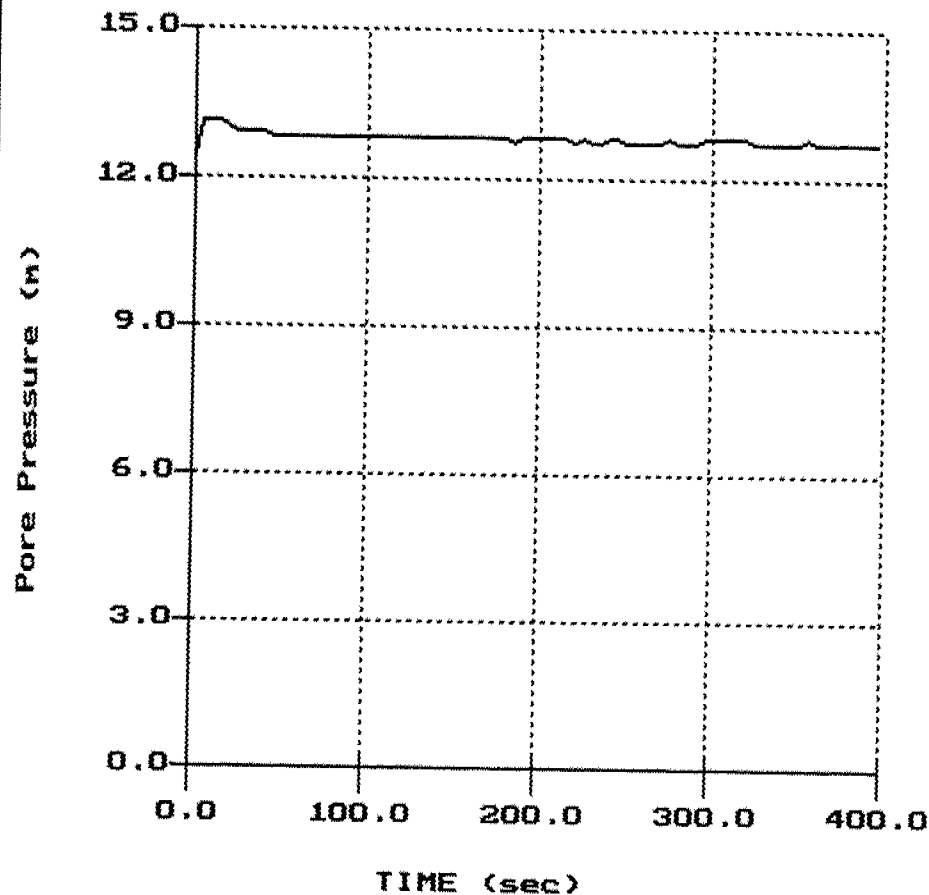
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(ft): 41.17

Duration: 395.0s

PORE PRESSURE DISSIPATION RECORD





**APPENDIX D**

**NON STANDARD SPECIAL PROVISIONS**

**Granular Blanket**

**Wick Drains**

**Special Provision****1.0 SCOPE**

This non-standard special provision specifies the requirements for the surface preparation, supply, placement and compaction of the Granular Filter blanket in connection with the installation of the prefabricated vertical drains.

**2.0 MATERIALS**

The Granular Filter Blanket shall be Granular 'A' material and shall satisfy the physical and gradation requirements as specified in OPSS 1010.

**3.0 CONSTRUCTION**

3.1 The Granular 'A' blanket shall be placed and compacted to the limits and, grades shown on the plans or as directed by the Contract Administrator.

3.2 The Granular 'A' blanket shall be placed subsequent to the required subexcavation.

3.3 The Granular 'A' blanket shall be placed and compacted in lift thicknesses not exceeding 250 mm.

3.4 The Granular 'A' blanket shall be compacted to 90%  $\pm$  2% of its standard proctor density.

**4.0 PAYMENT****4.1 Measurement of Payment**

Measurement of payment shall be by the tonne. The method of determining the mass of materials for payment shall conform to OPSS 102.

#### 4.2 Basis of Payment

Granular 'A' Blanket - Item

Payment at the contract price for the above item shall be full compensation for all labour, equipment and material required to do the, work.

## Special Provision

**1.0 GENERAL****1.1 Scope**

This non-standard special provision specifies the requirements for the supply and installation of wick drains in accordance with the details shown on the plans and with the requirements of these specifications.

**1.2 Qualifications**

This work shall be undertaken by a recognized specialist subcontractor with at least 5 years of proven satisfactory experience in work of this type and magnitude.

**2.0 SITE CONDITIONS**

The Contractor shall refer to the following reports in the Contract Documents for a description of subsurface conditions at this site:

- Foundation Investigation, Bridge Structure, Approaches and Embankment Fills - North Interchange (McCarthy Street) - Trout Creek by-Pass, King's Highway 11 - District 54, Sudbury, Ontario, GWP No.774-93-00 by Trow Consulting Engineers
- Design Report -Trout Creek By-pass - King's Highway 11 -Wick Drain Design and Monitoring Program - North Interchange Embankments - District 54, Sudbury, Ontario - GWP No. 774-93-00 by Thurber Engineering Ltd.

The Record of Borehole sheets are not represented as a complete description of the subsurface conditions, but only present what was found in borings at the indicated locations on the date boreholes were drilled. The subsurface conditions may be variable between the borehole locations.

The Contractor should verify existing surface conditions.

### **3.0 MATERIALS**

- 3.1 The prefabricated drain shall consist of a continuous plastic drainage core wrapped in a non-woven geotextile material. The core configuration should be 'Studded' or 'Grooved' ('Filament ' or 'Cusped' are not acceptable).

The Contractor shall submit samples of the prefabricated drain for evaluation and approval to the Contract Administrator at least one month prior to commencement of work under this item.

Fabricated wick drain material shall meet the minimum Specifications included in the table attached at the end of this text.

- 3.3 The Contractor shall submit a 1 m sample of the vertical drain material to the Contract Administrator prior to usage and shall allow two weeks for the Contract Administrator to evaluate the material. The sample shall be stamped or labelled by the manufacturer as being representative of the drain material having the specified trade name. Documentation indicating the source of the drain shall be provided. Approval of the sample by the Contract Administrator shall be required prior to site delivery of the vertical drain material.

- 3.4 Manufacturer certification shall be provided for all drain material delivered to the project.

- 3.5 All drains supplied shall be free of defects, rips, holes or flaws. During shipment the drain shall be protected from damage. During on-site storage the storage area shall be such that the drain is protected from sunlight, dirt, dust, mud, debris and any other detrimental substances.

### **4.0 EQUIPMENT**

- 4.1 Vertical drains shall be installed with equipment which will minimise disturbance to the granular 'A' blanket or the native subsoil during the installation operation. Static or vibratory methods are considered acceptable. Falling weight impact hammers will not be allowed.



- 4.2 The Contractor is advised that the site is considered as an environmentally sensitive area and therefore the control of any water effluent needs to be carefully planned and organized. Jetting techniques, therefore, shall be subjected to the approval of the Contract Administrator.
- 4.3 The Contractor shall be permitted to use augering equipment to predrill or to loosen the native soils and the granular 'A' blanket if required to facilitate the installation of the wick drains.
- 4.4 Each prefabricated wick drain shall be installed using a mandrel or sleeve which shall be advanced through the underlying soil and the granular blanket. The mandrel shall protect the prefabricated drain material from tears, cuts and abrasions during installation and shall be withdrawn after the installation on the drain. The mandrel shall be provided with an "anchor" rod or plate at the bottom to prevent the soil from entering the bottom of the mandrel during installation of the drain and to anchor the bottom of the drain at the required depth at the time of mandrel removal. The projected cross-sectional area of the mandrel and anchor combination shall not exceed 7700 mm<sup>2</sup>.

## **5.0 INSTALLATION**

### **5.1 Installation Method Proposal Submission**

At least three weeks prior to the installation of the drainage strips, the Contractor shall submit to the Contract Administrator, for review and approval, details of the sequence and method of installation. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

- Size, type, weight, maximum pushing force, and configuration of the installation rig.
- Dimensions and length of mandrel.
- Details of drain anchorage.
- Detailed description of proposed installation procedures.

- Proposed methods for overcoming obstructions.
- Proposed methods for splicing drains.

Approval by the Engineer will not relieve the Contractor of his responsibilities to install vertical drain strips in accordance with the plans and specifications.

## 5.2 Construction Sequence

Vertical drains shall be installed subsequent to the construction of the granular 'A' blanket and prior to installation of monitoring instruments and placement of the embankment material.

## 5.3 Trial Drains

Prior to the installation of prefabricated drains within the areas designated on the plans, the Contractor shall demonstrate that the proposed materials, equipment and installation method produces a satisfactory drain installation in accordance with these specifications. The Contractor will be required to install a total of ten trial drains at locations within the work area as designated by the Contractor Administrator.

Should the ten trial drains be installed to the satisfaction of the Contract Administrator, the trial drains can be incorporated as part of the permanent installation. The Contractor will be compensated for each trial drain if the installation satisfies the requirements of this specification, at the same unit price as the production drains. The Contractor shall not be compensated for unsatisfactory trial drains.

Approval by the Contract Administrator of the method and equipment used to install the trial drains shall not constitute, necessarily, acceptance of the method for the remainder of the project. If, at any time, the Contractor Administrator installation considers that the method of installation does not produce a drain which satisfies the project requirements, the Contractor shall alter his method and/or equipment as necessary to comply with these specifications.

#### 5.4 Layout

Prefabricated drains shall be located and staked out by the Contractor. The location of the drains shall not vary by more than 150 mm from the locations indicated on the drawings.

#### 5.5 Plumbness

Drains shall be installed vertically, within a tolerance of not more than 10 mm per 500 mm. The equipment shall be carefully checked for plumbness, and the Contractor shall provide the Contract Administrator with a suitable means of verifying the plumbness of the mandrel and of determining the depth of the drain at any time.

#### 5.6 Splices

Splices or connections in the vertical drain material shall be done in a professional manner so as to ensure continuity and to avoid any reduction of the flow characteristics of the wick material. Splices shall be a minimum of 150 mm in length.

#### 5.7 Cut-off

The prefabricated drain shall be cut at the surface such that at least a 150 mm length protrudes above the top of the granular blanket at each drain location.

#### 5.8 Obstructions

Where obstructions are encountered below the working surface which cannot be penetrated by the drain installation equipment, the Contractor shall complete the drain from the elevation of the obstruction to the working surface and notify the Contract Administrator. At the direction of the Contract Administrator, the Contractor shall attempt to install a new drain within a 500 mm radius of the obstructed drain. A maximum of two attempts shall be made as directed by the Contract Administrator. The Contractor will be compensated for each obstructed drain unless the drain is improperly completed, in which case no compensation will be allowed.

## 5.9 Preaugering

Preaugering will likely not be required at this site. If however, the Contractor judges that preaugering is required, the drilling shall not extend more than 1m into the Silty Clay deposit at the site. Any additional cost for preaugering, shall be incorporated into the unit price.

## 5.10 Rejected Drains

Prefabricated drains that are installed beyond the plan location by more than 150 mm, or that are damaged or are not installed in accordance with the specifications described above shall be rejected. Rejected drains may be removed at the Contractor's own expense and time. The Contractor shall not be compensated for the materials and work associated with rejected drains.

Replacement drains shall be installed within a 50 cm radius from the location of the rejected drain as directed by the Contract Administrator.

## 5.11 Geotechnical Instrumentation

Installation of the drains should be coordinated with the placement of geotechnical instrumentation as shown on the drawings. Special care should be taken to install drains in such a manner so as not to disturb instrumentation already in place. The replacement of instrumentation damaged as a result of the Contractor's activities will be the responsibility of the Contractor.

## 6.0 **PAYMENT**

### 6.1 Measurement of Payment

Measurement of the item "WICK DRAINS" is by Plan Quantity, as may be revised by Adjusted Plan Quantity shall be by the linear metre for all accepted drains installed including the protruding portion. Properly completed obstructed wick drains and properly installed replacement wick drains and trial drains will be measured for payment.

6.2 Basis for Payment

Item - Wick Drains

Payment at the contract unit price per linear metre for the above item shall be full compensation for all labour, materials and equipment to complete the work in accordance with the Plans and Specifications.

No payment shall be made for unacceptable drains or delays or expenses incurred by the Contractor as a result of improper or unacceptable material or installation.

PRODUCT SPECIFICATIONS			
	TEST METHOD	UNITS	VALUE
PHYSICAL PROPERTIES			
Drain Body Material		Studded or Groved	Polypropylene
Filter Material		Non-Woven	Polypropylene
Weight	ASTM-D-1777	g/m	75
Width		mm	not less than 100
Thickness	ASTM-D-5199	mm	not less than 3
Mass of Filter	ASTM-D-1777	g/m <sup>2</sup>	154
MECHANICAL PROPERTIES			
Drain composite Tensile Strength	ASTM D-4595	kN	0.375 @ 10%
Filter Puncture Strength	ASTM-D-751-68	kN	0.335
Filter Grab Strength	ASTM-D-1682	kN	0.8
Filter Trapezoidal Tear	ASTM-D-1117	kN	0.22
Filter Burst Strength	ASTM-D-751-68	kPa	2000
Discharge Capacity @ 70 kPa	ASTM-D4716	m <sup>3</sup> /s	100x10 <sup>-6</sup>
FOS	CAN/CGSB-148.1 No. 10.2	μm	15 to 100
Minimum elongation at break (%)	CAN/CGSB-148.1 No. 7.3	%	15
Water Permeability	ASTM D-4491	m/s	0.000005

**Foundation Investigation and  
Design Report  
Bridge Structure, Approaches  
and Embankment Fills  
North Interchange (McCarthy Street)  
Trout Creek By-Pass, King's Highway 11  
District 54, Sudbury, Ontario  
GWP No. 774-93-00**

**Prepared for:**

**Marshall Macklin Monaghan  
80 Commerce Valley Drive East  
THORNHILL, Ontario  
L3T 7N4**

**Trow Consulting Engineers Ltd.**

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Chicago, Fargo, Madison, St. Anthony, Tallahassee, Bogotá**

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Facsimile: (705) 674-8271**

**SO7524G/B  
January 7, 1999**

## Preface

Work Project GWP 774-93-00 is one of a series of projects for the four lane expansion of Highway 11. The project involves the four lane design of Highway 11, from 4.0 km south of Highway 522, northerly for 7.9 km. It will result in the construction of a westerly by-pass of the existing Highway 11 and the Town of Trout Creek.

This section is located in the Townships of Laurier and Himsworth South, within the geographic District of Parry Sound. The project requires geotechnical input for the following major components:

- New pavement design for the entire length of the four lane by-pass, including associated service roads.
- Trout Creek South Interchange (underpass), Site 44-372, Foundations and Approach Embankments.
- Trout Creek Northbound Lanes, Site 44-371N, Foundations and Approach Embankments.
- Trout Creek Southbound Lanes, Site 44-371S, Foundations and Approach Embankments.
- Highway 522 (underpass), Site 44-370, Foundations and Approach Embankments.
- Trout Creek North Interchange (underpass), Site 44-369, Foundations and Approach Embankments.

The following report deals with the foundation conditions for the proposed 2 span bridge structure and the approaches and ramps for the proposed Trout Creek North Interchange (McCarthy Street ), Site 44-369. Separate reports will be submitted for the other, above listed components.



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# PART 1: FOUNDATION INVESTIGATION FOR BRIDGE STRUCTURE, APPROACHES & RAMPS

## 1.1 Introduction

**DRAFT**

This report presents the results of a geotechnical foundation investigation undertaken by Trow Consulting Engineers Ltd. (Trow) for the proposed bridge structure and the associated approach and ramp embankments, at the North Interchange to the Town of Trout Creek (the proposed King's Highway 11 Trout Creek By-Pass and McCarthy Street) at Site 44-369.

It is Trow's understanding that a two span bridge structure will be constructed to carry the realigned McCarthy Street traffic over the new section of the four lane Highway 11, with the central pier located in the median of the proposed King's Highway 11. An approximate 2.8 m grade increase of the new four lanes is proposed at the bridge location, requiring up to approximately 11 m grade raises for the realigned section of McCarthy Road at the bridge abutments, in accordance with the grading plan proposed to meet the design objectives.

## 1.2 Site Description and Geological Setting

The site is located in the Township of South Himsforth at the proposed Highway 11, Trout Creek By-Pass, and the realigned McCarthy Street Connection, approximately 1.2 km north of the Town of Trout Creek, as shown on Figure 1-A and 1-B in Appendix A. The bridge structure is located at approximate Station 13+492 along the proposed Highway 11, which corresponds to Station 10+000, along the proposed, realignment of McCarthy Street at this location.

The existing ground level at the site is at approximately elevation 315.2 m. The grade of the proposed four lane, Trout Creek By-Pass is at elevation 318 m, while the McCarthy Street grade (bridge level) is at approximately elevation 326 m. Ramps will be constructed to allow access to and from McCarthy Street to the south and north bound lanes of Highway 11.

The terrain at the site for the proposed bridge structure is relatively flat and poorly drained, as evidenced by surficial ponding water. No bedrock outcroppings are visible at the bridge site.

relocate  
to  
original  
alignment  
Recommendation

Mature trees, with heavy underbrush, are found in the immediate vicinity of the structure and approaches. An easement for TransCanada PipeLines (TCPL) runs in a north-south direction, with the closest edge of the TCPL right-of-way (east side) located approximately 14 m west of the westerly face of the abutment of the proposed structure. It is understood that the existing TCPL right-of-way will be relocated to the west of the proposed interchange, and therefore, will not impact on the design of the proposed interchange.

According to OGS Maps 2544 and 2556, as well as Ontario Geological Survey Map P. 3160 (Quaternary geology, South River area), the site is located in what is known as the Central Gneiss Belt.

The overburden soils have been mapped as comprising relatively shallow, deltaic sands and gravels with some prodeltaic silts, and deposits of glacio-lacustrine silts and clays. A thin layer of basal, stoney, glacial till can be expected immediately over the bedrock.

*Historical Geology (glaciation)*

### **1.3 Investigative Procedures**

#### **1.3.1 General**

Part 1.3 of this report describes the investigative procedures adopted for the geotechnical assessment of the north interchange (McCarthy Street) of the Trout Creek By-Pass, King's Highway 11. Properties of the overburden soils were obtained by insitu, and laboratory testing procedures, as described below.

#### **1.3.2 Field Investigation**

The field work for the geotechnical investigation related to the bridge structure and associated approaches and ramps was carried out between May 25, 1998 to August 9, 1998. Table 1-1, below, provides a summary of the boreholes drilled and the borehole numbering system used for this project.



Table 1-1 Field Work Summary

Interchange Component	Field Work	Date of Fieldwork	Depths Penetrated (m)
Bridge Structure	<b>3 boreholes</b>		
	BH-1BF	May 25, 1998	15.2 (incl. ~3 m of bedrock core)
	BH-2BF	May 26, 1998	15.9 (incl. ~4.6 m of bedrock core)
	BH-3BF	May 27, 1998	17.4 (incl. ~3 m of bedrock core)
	<b>3 Dynamic Cone Tests</b>		
	C-1BF	May 25, 1998	13.7
	C-2BF	May 26, 1998	11.9
	C-3BF	May 27, 1998	14.4
East Interchange Approach	<b>6 boreholes</b>		
	BH-1ABP	May 28, 1998	6.6*
	BH-1BBP	August 5, 1998	12.6
	BH-2BP	July 17, 1998	14.2
	BH-3BP	July 15, 1998	18.1
	BH-4BP	July 15, 1998	18.8
	BH-5BP	July 14, 1998	15.7
			* (terminated prematurely due to nearby advancing forest fire)
East Interchange Ramps	<b>3 boreholes</b>		
	BH-9BP	July 16, 1998	19.7
	BH-10BP	July 17, 1998	14.2
	BH-11BP	July 16, 1998	17.3
West Interchange Approach	<b>9 boreholes</b>		
	BH-12BP	July 17, 1998	3.7
	BH-14BP	July 17, 1998	3.5
	BH-15BP	July 17, 1998	0.3
	BH-16BP	July 17, 1998	2.3
	BH-17BP	July 17, 1998	4.6
	BH-18BP	July 17, 1998	0.5
	BH-19BP	July 17, 1998	3.2
	BH-20BP	July 17, 1998	1.4
	BH-21BP	August 6, 1998	5.6
West Interchange Ramps	<b>3 boreholes</b>		
	BH-13BP	July 17, 1998	1.8
	BH-22BP	August 9, 1998	4.2
	BH-23BP	August 8, 1998	8.1

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} could not locate an plan.

The borehole locations are shown on the attached site plan, Drawing 1B, in Appendix A. These locations, as well as the surface elevations, were established by a survey crew from Marshall Macklin Monaghan, and are referenced to geodetic datum.

Investigations in connection with the bridge foundations consisted of three boreholes (BH-1BF, 2BF, and 3BF) and three dynamic cone penetration tests (C-1BF, 2BF, and 3BF). The dynamic cones were advanced to refusal at depths ranging from 11.9 m to 14.4 m. The three boreholes were advanced to refusal at between 11.3 m and 14.4 m depth, and were then diamond drilled for 3.0 m to 4.6 m to prove and verify bedrock quality.

Investigations in connection with the embankments included a total of 21 fully sampled boreholes. Six boreholes were advanced along the alignment of the East Interchange Approach and 3 boreholes in the vicinity of the East Interchange ramps. Nine boreholes were advanced along the alignment of the West Interchange Approach, and 3 additional boreholes for the West Interchange ramps.

The majority of the boreholes were advanced through the overburden soils using either a track-mounted, CME-55 or Dietric 50 power auger rig, equipped with both solid and hollow stem augers, and supplied by a soils drilling contractor, Master Soil Investigations Limited. The boreholes were advanced to depths ranging from 0.3 m to 19.7 m. Due to accessibility problems, 4 boreholes (1-BBP and 21-BP to 23BP) were advanced using light weight, motorized tripod equipment, in conjunction with standard wash boring techniques. Soil samples were obtained by using a 51 mm O.D. split-spoon sampler in conjunction with standard penetration tests (ASTM D1586), at approximately 0.75 m and 1.5 m intervals. The standard penetration test "N" values were recorded and used to provide an assessment of the compactness or consistency of the overburden soils. Several "undisturbed", 50 mm I.D., thin walled Shelby tube samples were obtained from cohesive soil deposits. As well, field vane shear strength testing was carried out in the boreholes throughout the cohesive soils to measure the *in situ* undrained shear strength. The recovered soil samples were examined and carefully logged in the field, and then shipped to Trow's Sudbury laboratory for further visual and tactile examination, identification, and laboratory testing.

At the bridge structure site, a borehole and dynamic cone test were carried out at each of the three foundation elements. After encountering refusal in each borehole, bedrock was proven by rock coring to depths ranging from approximately 3.0 m to 4.6 m, using standard diamond drilling techniques. A "BQ" size core barrel was used and core samples of the bedrock were retrieved for rock quality determination and classification.

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The boreholes for the approaches and ramps were generally terminated either at practical refusal on either assumed bedrock or a boulder obstruction, or in several cases, in competent granular or cohesive soils beneath any soft clay or loose silt zones. Borehole BH-1ABP was terminated prematurely at 6.5 m depth due to an advancing forest fire, and was subsequently re-drilled (as BH-1BBP) near the same location.

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Details of the soil and bedrock conditions encountered in the boreholes are included on the logs in the attached Appendix B. The additional two standard data sheets, included with the logs, provide further details on soil descriptions for classification purposes. Table B-1 in Appendix B provides the results of our analysis of the recovered rock core samples.

### 1.3.3 Laboratory Testing

The laboratory testing program for selected soil samples consisted of the following:

- Natural moisture content determinations
- Atterberg limits
- Grain size distribution analyses
- Unit Weight determinations
- One dimensional Consolidation Test
- Laboratory Shear Tests

The moisture contents, Atterberg limit test and laboratory shear test results are summarized on the attached borehole logs in Appendix B. The grain size distribution curves for selected soil samples are presented on Figures 1 to 6 in Appendix C. The results of the Atterberg Limit tests are also plotted on the Plasticity Chart on Figure 7 and the Consolidation Test on Figure 8 (also in Appendix C).

## 1.4 Subsurface Conditions

The borehole locations are shown on the site plan, Drawing 1B, in Appendix A. Included in Appendix B are the borehole and dynamic cone penetration test logs. Longitudinal and cross sectional soil sections at the bridge structure site and along the ramp/embankment sections are plotted on Drawings 1C to 1F in Appendix A. Based on the borehole information, the following different soil layers were typically encountered, although they were not necessarily present in each borehole.

- peat/organic topsoil

- silty sand
- sand
- sandy silt
- silty clay with silt layers
- silt, with clay and sand seams
- silty sand and gravel till
- bedrock

The soil conditions are generally similar in the vicinity of the bridge structure site and the approach and ramps to the east of the bridge structure, with the boreholes encountering a layer of compressible silty clay. To the west of the bridge structure, the ground surface rises and there is typically shallow bedrock and absence of the silty clay layer.

A summary of the above soil strata encountered in the boreholes, and inferred from the dynamic cone tests, is presented below.

#### **1.4.1 Peat/Organic Topsoil**

In the flat, poorly drained area to the east of and including the bridge structure, there is a surficial layer of peat overlying the underlying mineral soils. Based on the soil exploration data, it appears that the peat layer is typically very soft, wet, black and dark brown in colour, and generally from 0.5 m to 0.9 m thick. On the higher, better drained ground to the west of the bridge structure, generally shallow depths of predominantly unsaturated organic topsoil were encountered, typically less than 300 mm in thickness.

#### **1.4.2 Silty Sand**

Immediately below the organic veneer, a stratum of wet, brown silty sand was encountered in boreholes BH-2BP, 3BP, and in BH-13BP, 14BP, 19BP and 23BP. The silty sand is typically fine to fine-medium in gradation, with a silt content typically ranging between about 22% to 40%, and occasionally, some gravel sizes. The results of several grain size analyses are provided on Figure 1 in Appendix C. Moisture contents typically vary from 22% to 32%.



The silty sand is also organic-stained with some root inclusions at upper levels. The silty sand deposit extends to depths of approximately 1.4 m to 3.0 m below the existing ground level. The silty sand is typically in a loose to compact condition, with standard penetration resistance "N" values ranging from 9 to 16 blows/300 mm.

#### **1.4.3 Sand**

Immediately below the organic veneer, or beneath the upper silty sand layer, a stratum of wet, brown sand was encountered in boreholes BH-1BF to 3BF, and in boreholes BH-1ABP to 4BP, 9BP to 11BP, 17BP, and 20BP to 23BP. The sand is generally fine in gradation, becoming somewhat coarser with depth, and contains a trace to some silt content. The sand is also organic-stained at upper levels, where it directly underlies the surficial peat stratum. Grain size analyses (see Figure 2 in Appendix C) confirm that the silt fraction is approximately 5% to 15%. Moisture contents typically vary between 15% to 25%.

The sand deposit typically extends to depths of 2.4 to 5.2 m below the existing ground level, except where bedrock is encountered at shallow depth. The sand is generally compact but occasionally compact to loose, with standard penetration resistance "N" values ranging from 6 to 25 blows/300 mm.

#### **1.4.4 Sandy Silt**

A stratum of predominantly sandy silt was encountered in several boreholes either directly beneath the peat stratum (BH-5BP, 12BP, 16BP), or beneath the sand stratum (BH-4BP & 9BP). The sandy silt is brown and wet, with a silt content greater than 50% and the balance being fine sand. The results of several grain size analyses are provided on Figure 3 in Appendix C, and indicate that the silt content of the material ranges from about 56% to 77%. Moisture contents vary from 22% to 32%.

The sandy silt deposit, where encountered, extends to depths of between 1.5 to 7.0 m below the existing ground surface. The sandy silt is typically compact, but occasionally loose or dense, with standard penetration resistance "N" values ranging from 7 to 31 blows/300 mm.

### **1.4.5 Silty Clay with Silt Layers**

#### **1.4.5.1 Bridge Structure Site**

Beneath the sand stratum at the proposed bridge structure site, a stratum of layered silty clay was intercepted beneath the centre pier (BH-1BF) and the east abutment (BH 3-BF). At the west abutment (BH 2-BF), the silty clay stratum is absent. The silty clay is thickest at borehole 1-BF (pier), i.e. approximately 4.5 m thick (extending down to 7.3 m depth), while at borehole 3-BF (east abutment), the silty clay layer reduces in thickness to about 2 m (extending down to 5 m depth). The silty clay material contains numerous wet, thin silt seams, which typically increase in frequency with depth. In-situ field vanes and laboratory shear tests confirm that the silty clay has a stiff crust with undrained shear strengths exceeding 100 kPa. The silty clay, however, at depth, reduces to a soft/firm consistency with undrained shear strengths of 28 kPa to 33 kPa. Sensitivity, the measure of peak shear strength to remoulded shear strength, ranged from 5 to 7, indicating that the silty clay is moderately sensitive.

#### **1.4.5.2 Eastern Approach & Ramps**

The silty clay stratum was also encountered in all of the boreholes underlying the proposed approach and ramps to the east of the bridge structure (i.e., boreholes BH-1ABP to -11BP), with thicknesses ranging from 3.4 m in BH-1BBP to 7.5 m in BH-11BP. The silty clay material contains numerous, wet, thin silt seams, which typically increase in frequency and in thickness with depth. In-situ field vanes indicated undrained shear strengths ranging from 23 kPa to 48 kPa in soft to firm zones within the clay. A stiff upper clay crust was not identified in any of these boreholes. The shear strength of the silty clay typically increases with depth to stiff or very stiff, partially as a result of increasing silt content. The sensitivity ranged from 3 to 7, but is typically 4, indicating that the silty clay is moderately sensitive. The undrained shear strength profile, which includes shear strength data from all of the boreholes, is provided on Figure 8 in Appendix C.

Standard Penetration Test "N" values obtained throughout the silty clay deposit ranged from less than 1 to 5 blows/300 mm, thereby inferring a soft to very stiff consistency.

The moisture content of the clay varies from 32% to 66%, with the higher values associated with weaker, more compressible zones and the lower values associated with less compressible, higher silt content material. The results of Atterberg limits tests (summarized on Figure 7 in Appendix C and on the Moisture Content-Atterberg Limit Profile on Figure 8, as well as on the individual borehole logs) indicate Liquid Limits typically ranging between 29% to 48%, Plastic Limits from 20% to 22%, and corresponding Plasticity Indices ranging from 8 to 25%. The silty clay material, therefore, is considered to be an inorganic clay of low to medium plasticity, i.e. a "CL" soil (in accordance with the Unified Soil Classification System).

*CL - low plasticity  
CI - med plasticity*

Several grain size analyses carried out on representative samples of the silty clay material (see Figure 4) indicate that the material contains approximately 20% to 22% clay sizes (i.e. finer than 2  $\mu\text{m}$ ), 65% to 78% silt sizes, and 0% to 14% fine sand sizes.

The results of a one-dimensional consolidation test, carried out on a sample of representative, soft/firm silty clay extruded from an "undisturbed" Shelby tube from a depth of 7.7 m in borehole BH-10BP, are presented graphically on the void ratio vs pressure diagram on Figure 9 in Appendix C. The consolidation test results are summarized in Table 1-2, below. The data indicates that the silty clay material is lightly overconsolidated.

Table 1-2 Measured and Calculated Consolidation Test Parameters	
Parameter	BH-10, TW-5, 7.7 m Depth
Estimated Overburden Pressure ( $P'_o$ )	50 kPa
Estimated Max Pressure with Surcharge Loading ( $P'_1$ )	250 kPa
Estimated Pre-Consolidation Pressure ( $P'_c$ )	100 kPa
Coefficient of Volume Change ( $m_v$ )	$0.52 \text{ MPa}^{-1}$
Compression ratio, $C_c' = C_c / (1 + e_o)$	0.23
Recompression Ratio, $C_r' = C_r / (1 + e_o)$	0.012
Coefficient of Consolidation, $C_v$	$4.7 \text{ m}^2/\text{yr}$
Coefficient of Permeability, $k = c_v m_v \gamma_w$	$7.7 \times 10^{-8} \text{ cm/s}$

*$P'_c - P'_o$   
= 50 kPa*

#### 1.4.5.3 Western Section of Interchange

An isolated, thin 0.8 m thick layer of stiff silty clay was encountered in borehole BH-16BP at a depth of 1.5 m below existing grade beneath the upper sandy silt layer. The silty clay is underlain by probable bedrock at a depth of 2.3 m. Silty clay was not encountered in any of the other boreholes in the west section of the interchange.

#### **1.4.6 Silt**

A stratum of silt was encountered directly beneath the layered silty clay (boreholes BH1-BF and BH3-BF), and directly beneath the sand (borehole BH2-BF) at the bridge structure site. Silt was also encountered in all boreholes to the east of the bridge, generally underlying the silty clay, except in BH-5BP, where a 2.9 m thick layer of silt was encountered above the silty clay stratum. The silt stratum was also encountered in BH-21BP and 23BP along the western section of the interchange, beneath the N-EW ramp. The thickness of the silt stratum varies from approximately 2.4 m to greater than 7 m.

The silt contains seams of fine sand throughout, as well as occasional thin silty clay seams. Grain size analyses (see Figure 5) indicate that the silt material contains up to approximately 5% clay sizes (i.e. finer than 2  $\mu$ m), 85% to 90% silt sizes, and 5% to 10% sand sizes. The moisture content typically ranges from 19% to 33%.

The standard penetration resistance "N" values were reasonably consistent and ranged from 4 to 28, indicating a generally loose to compact condition.

#### **1.4.7 Silty Sand and Gravel Till**

A basal zone of silty sand and gravel till was encountered in boreholes BH-2BF and 3BF at the bridge site, 1BBP, 3BP, 9BP beneath the east section of the interchange, and in boreholes 12BP, 13BP, 14BP and 21BP, 22BP and 23BP beneath the west section of the interchange. although at borehole 1BF, it is absent. The deposit is reasonably well-graded with 17% to 30% silt sizes, 53% to 73% sand sizes and 10% to 18% gravel sizes (see Figure 6). The till also contains larger gravel sizes, cobbles and boulders, which were not sampled. The till appears to be weakly cemented, indicating a "till-like" structure. The moisture content ranged from 6% to 14%.

The standard penetration resistance "N" values ranged from 47 to 101 blows/300 mm at the bridge site, indicating a dense to very dense condition. At the embankment sites, the "N" values ranged between 19 and 60, indicating a compact to very dense, but generally compact to dense condition.

### 1.4.8 Bedrock

Bedrock was proven at the bridge structure site by drilling "BQ" size cores commencing at auger refusal depth in each of the 3 boreholes (BH's 1BF, 2BF and 3BF), i.e. at one borehole beneath each of the three foundation elements. Based on the borehole and dynamic cone penetration tests, the bedrock was established at the following depths:

- East Abutment (BH-3BF and cone C-3BF)  
14.3 m (~El. 300.9 m) to 14.4 m (~El. 300.8 m)
- Centre Pier (BH-1BF and cone C-1BF)  
12.2 m (~El. 303.1 m) to 13.7 m (~El. 301.5 m)
- West Pier (BH-2BF and cone C-2BF)  
11.3 m (~El. 304.1 m) to 11.9 m (~El. 303.5 m)

*West Abutment not  
West Pier*

Detailed descriptions of the rock cores obtained are presented in Table B-1 in Appendix B. Generally, the bedrock can be described as a pinkish, light grey, Biotite-Hornblende gneiss and, for the most part, is strong and unweathered. In the upper approximately 3.0 m, the bedrock at borehole BH-2BF (west abutment) is weathered and fractured.

Rock core recovery was 100% and the Rock Quality Designation (RQD) values exceed 80% in BH-1BF (centre pier) and BH-3BF (east abutment). However, the bedrock in the upper approximately 3.0 m in borehole BH-2BF is weathered and fractured and the core recovery was between 80% to 90%, with corresponding RQD values of 20% to 33%. Beneath this weathered zone, the core recovery increases to 100% with RQD values of approximately 63%.

## 1.5 Groundwater Conditions

Information regarding the groundwater levels at the site was obtained by measuring the water levels in the open boreholes after completion of drilling. The measured groundwater levels are shown on the borehole logs and stratigraphic sections. In general, the groundwater table, at the time of the field work, was established at or slightly below grade (i.e. within 300 mm) in the lower lying areas in the vicinity of the bridge site, the eastern section of the interchange, and in the ramp areas of the western interchange. In several areas, ponding water, up to 600 mm deep, was evident above the ground surface in random, local depressions over the poorly drained, flat

terrain. At the time of our investigation, a large beaver pond was located to the northeast of the N-EW Ramp, covering both TransCanada pipelines.

On the higher ground to the west of the bridge structure, the groundwater table was either at or near ground surface, to about 1.0 to 1.5 m below grade.

## Part 2 Engineering Discussions and Recommendations

**DRAFT**

### 2.1 Introduction

The following subsections address geotechnical considerations pertaining to the proposed two-span bridge for the McCarthy Street bridge structure underpass and the embankment construction for the approaches and associated ramps for the North Interchange of the Trout Creek By-Pass (King's Highway 11).

A two-span bridge is proposed to carry McCarthy Street traffic over the four lanes of the new By-Pass. The central pier will be located in the By-Pass median, with the abutments located on the west and east sides of the north and south bound lanes of Highway 11. The proposed pavement grade of the Highway 11 underpass is approximately 3 m above existing ground surface while the overpass pavement grade is approximately 11 m above ground surface. Embankment fills of up to about 11 m height above existing grade will be required to achieve the design objectives for the realigned Highway 11, McCarthy Road, and the various associated ramps.

span  
lengths  
provide  
Elevations

It is understood that the project is scheduled for completion within a two year time frame. Accordingly, some of the discussions and design recommendations presented herein are based on this time frame.

provide  
privileges  
descriptions  
beyond  
bridge

### 2.2 Foundations

Based on the borehole explorations, the upper loose/compact sandy soils, overlying loose silt or compressible silty clay, are considered to be too weak for supporting the proposed bridge foundations on spread footings, where the available allowable bearing pressures would be about 75 kPa. As such, it is recommended that the structure be supported on end-bearing, steel H piles, driven to refusal on the underlying bedrock. Integral abutments, incorporating the piles, can be considered at the abutments. Bedrock was encountered at average depths of approximately 11.6 m (west abutment), 13 m (central pier) and 14.4 m (east abutment).

✓

Current design practice requires that consideration be given to drag loads that may be generated as a result of soil consolidation due to placement of fills, potential groundwater table fluctuations, and the soil stresses induced by pile installation. We are assuming that the piles would be installed after the approach fills have been in place and allowed to almost completely consolidate the silty clay soils.

## 2.2.1 Pile Capacity and Length

Piles driven to bedrock can be designed based on the following ultimate limit state (ULS) design values, and should not exceed the factored axial resistance values, as shown in Table 2-1, below.

Table 2-1 Design H-Pile Capacities (kN)					
			HP 310x79	HP 310x110	HP 310x132
					HP 310x152
Factored (MTO*)	Axial	Resistance	1430	2000	2300 (est)
					2600 (est)
Notes: MTO* = Structural Office Policy Memo 98-01, April 15, 1998					

For design purposes, the modulus of horizontal subgrade reaction for steel H piles can be taken as  $8,000 \text{ kN/m}^2$  for the cohesive silty clay and  $40,000 \text{ kN/m}^2$  for the cohesionless soils.

Table 2-2 below provides a summary of the approximate bedrock elevations at the test locations at which piles would be expected to be founded, based on the attached borehole logs in Appendix B. Drawings 1C and 1D in Appendix A show interpreted soil and rock subsurface profiles at the two abutments and centre pier locations.

It should be noted that the elevations provided in Table 2-2 are approximate. Although not indicated in the borings advanced at this site, the bedrock surface elevation in Northern Ontario is known to be variable and may change rapidly over a very short distance.

reduction in axial capacity for inclined loads  
as per OHBDC



Table 2-2 Location and Estimated Elevation of Bedrock Foundation			
Location	Boreholes & Cone Tests	Approximate Overburden Thickness (m)	Approximate Bedrock Elevation (m)
West Abutment	Borehole BH-2BF Cone C-2BF	11.3	304.1
		11.9	303.5
Centre Pier	Borehole BH-1BF Cone C-1BF	12.2	303.1
		13.7	301.5
East Abutment	Borehole BH-3BF Cone C-3BF	14.3	300.9
		14.4	300.8

Pile caps should be provided with a minimum of 2.0 m of earthcover, or an equivalent thickness of soil and rigid polystyrene insulation, for frost protection.

The above elevations are for preliminary design purposes and were estimated based on the results of the boreholes and dynamic cones drilled near the abutment and pier locations. Interpolation between boreholes and probe holes is approximate, and as such, actual bedrock elevations may vary somewhat from those indicated above.

### 2.2.2 Construction

As discussed in the following subsections of this report, there is a potential for significant settlements of the approach fills on the east side of the bridge structure, as well as from the Highway 11 embankments on either side of the central pier. There also will be associated lateral strains. Accordingly, piles should not be installed until after the majority of the fill settlement has occurred, based on monitoring during construction. This is particularly important for inclined piles, that would be subjected to bending stresses.

*preload  
rec'd  
duration*

All piles should be driven to practical refusal on bedrock. Since the boreholes indicate that the bedrock elevations are relatively uniform, the potential for irregular steeply sloping bedrock at the foundation locations is considered to be low to moderate. The bedrock in Northern Ontario, however, is known to be highly variable, and, as such, some minor problems may arise during pile seating. At some locations, the piles may have a tendency to skip over the bedrock surface, resulting in alignment problems and deeper penetration. In the event that this problem occurs, somewhat longer piles may be required and, in some cases, piles may have to be added or replaced.

To minimize seating difficulties, rock injector points may be considered to facilitate proper seating. All piles must be fitted with reinforcing plates welded to the flanges as per OPSA 3301, in order to minimize pile damage. It is recommended that, during pile driving and upon initial contact with the bedrock, the pile driving energy should be reduced and subsequently increased incrementally until the piles have been sufficiently seated.

## 2.3 Backfill

Backfill to abutments or retaining walls should consist of free-draining granular materials, such as OPS Granular "A", Granular "B" or rock fill. Computation of earth pressures shall be in accordance with Section 6.7.4 of the Ontario Highway Bridge Design Code. Unfactored properties for backfill materials are provided in the following Table 2-3.

Table 2-3 Fill Material Types and Unfactored Geotechnical Properties					
Material	Friction Angle, $\phi'$	$\gamma$ (kN/m <sup>3</sup> )	$K_a$	$K_p$	$K_o$
Granular A	35 degrees	22.0	0.27 0.32	3.7	0.43 0.478
Granular B	30 degrees	21.0	0.33 0.38	3.0	0.50 0.55
Rock Fill	42 degrees	20.0	0.20 0.24	5.0	0.33 0.38

Note:  $K_a$  is the earth pressure coefficient corresponding to the active state.  
 $K_p$  is the earth pressure coefficient corresponding to the passive state.  
 $K_o$  is the earth pressure coefficient at rest.

} applicable

If rock fill is used as backfill behind abutments, the particle size should be limited to no greater than 300 mm and the backfill must be placed carefully in a manner that does not cause damage to the abutments or other structural components of the bridge. *reference OASD 3505*

## 2.4 Excavations

Minimal excavations for the pile caps will be required, since the grades will be raised. The upper organic zone (~350 mm to 1000 mm thick) will have to be removed from the entire construction area, and any additional excavations into the underlying silty sands and sands will be shallow. In accordance with the Occupational Health and Safety Act Regulations for Construction Projects, excavation procedures for Type 3 soils will be adequate, provided the groundwater in the overburden soils is removed. If appropriate dewatering is not done, the saturated soil would have to be considered as a Type 4 soil, and any excavation greater than 1.2 m in depth should then be sloped at 3 horizontal to 1 vertical, starting from the base of the excavation, or alternatively, appropriate shoring provided.

## 2.5 Bridge Approach Fills And Embankments

The construction of the north interchange, including the bridge, will require embankment fill heights of up to about 11.7 m height, with the embankments to the east of the bridge structure located over areas with varying thicknesses of soft, compressible, silty clay soils. This creates two principal design and construction considerations: embankment stability and consolidation settlement. These two issues are discussed in the following subsections; however, the implications of the two issues are inter-related.

In all of the following discussions, it is assumed that all organic material is removed beneath the rock fill embankments and that they are constructed directly on the native, mineral soils, with fill heights measured from the top of the native mineral soil.



### 2.5.1 Embankment Stability

Highway embankments in Northern Ontario are typically constructed using rock fill as the principal component. A typical MTO section will consist of a minimum (steepest) slope of 1.25H to 1V to a maximum height of 6 m. For embankment sections greater than 6 m height, a 2 m wide horizontal bench is required before the next 6 m high, 1.25H to 1V lift is constructed. For the stability and settlement analyses, typical geometric profiles for the embankments were assumed, with the embankments constructed of blast rock fill, with a crest width of approximately 17 m and average, uniform side slopes of 1.5H:1V. The width of the McCarthy Street roadway is expected to be approximately 13 m. However, the crest width of the rock fill will be overbuilt by 2.0 m on either side to compensate for the anticipated long term settlement of the rock fill embankment, and possible future pavement overlays, which will require additional grade raises.

Is it  
6 or  
10 metres  
geometry  
simplified  
why?

The embankments for McCarthy Street and the associated ramps to the west of the bridge structure are considered to be stable under both the short term construction and long term loading conditions. The highest embankments, i.e. up to 11.1 m, are located over areas without any soft, compressible silty clay, and where bedrock is relatively shallow.

global  
stability  
surface  
stability

Slope stability analyses were performed on the proposed embankments to the east of the bridge structure, which are underlain by varying depths of soft/firm silty clay, using SLOPE/W (ver. 3.x), based on Bishop's Simplified Method, using both total stress and effective stress parameters. The undrained shear strength profile indicated on Figure 9 and the shear strength values at each borehole location were used to provide the shear strength parameters for the silty clay soils at various stations along the embankment sections. Table 2.4 below lists the unit weights and strength parameters used. Effective stress parameters were established from multi-stage, consolidated, undrained, triaxial shear strength tests, performed on representative silty clay samples from the site of the South Interchange project, as well as from various geotechnical literature. The silty clay material at the South Interchange site is considered to have similar characteristics to the silty clay soils beneath the eastern section of the North Interchange

← site

Table 2.4 Geotechnical Parameters for Slope Stability Analyses				
	$\gamma_{total}$ (kN/m <sup>3</sup> )	$c_u$ (kPa)	$\phi'$	$\sigma'$ (kPa)
Rockfill	20.0	0	42°	0
Silty Sand/Sand	20.0	0	35°	0
Upper Silty Clay	16.5	20 - 25	28° - 26°	10 - 6
Lower Silty Clay	17.5	50 - 75	28° - 26°	10 - 6
Lower Silt	19.5	0	31°	0
Silty Sand and Gravel Till	21.5	0	37°	0

Notes: Embankment crest width 17 m average side slopes - 1.5H:1V

*high angle of internal friction*

### 2.5.1.1 Total Stress Analysis

The results of the slope stability analyses, in terms of total stresses using undrained shear strengths for the silty clay, as would apply for rapid construction (i.e., short term stability), are provided in Table 2-5, below. A summary of the slope stability analyses carried out near specific borehole locations is provided in Table 2.6, below. Selected graphical printouts, depicting the various slope stability analyses performed and discussed below, are provided in Appendix D.

Table 2-5 Factors of Safety for Total Stress (Short Term) Stability Analyses					
Embankment Height	Soft/Firm Silty Clay Thickness				
	2 m	3 m	4 m	5 m	6 m
5 m	1.63	1.63	1.63	1.51	1.51
6 m	1.37	1.37	1.36	1.33	1.33
7 m	1.36	1.36	1.36	1.22	1.22
8 m	1.28	1.28	1.28	1.10	1.10
9 m	1.26	1.26	1.22	1.04	1.04
10 m	1.19	1.19	1.13	< 1	< 1

Notes: i) Embankment crest width = 17 m, average side slopes at 1.5H:1V  
 ii) Assumed Average Shear Strength of Soft/Firm Silty Clay = 25 kPa

Embankments up to, and including, 6 m in height can be constructed, based on the analyses, with a resulting factor of safety against foundation failure greater than the MTO accepted value of 1.3. The potential problem areas for the short term "undrained" conditions occur in a triangular shaped area, where the embankment fills are greater than 6 m height over 5 - 6 m of soft/firm silty clay, and include the McCarthy Street embankment, east of the bridge crossing, from approximately stations 10+120 to 10+230, and on the elevated sections of the EW-N and S-W/S-E ramps, extending approximately 150 m to the south of the McCarthy Street intersection. Unacceptably low factors of safety (i.e. approaching unity) were calculated for the full height of the embankment. These higher embankments require special consideration, such as staged construction, or other measures, as discussed below.

**Table 2.6 Summary of Results of Slope Stability Analyses**

Station/ Borehole	Height of Embankment Fill (m)	Soft/Firm Silty Clay Layer		Undrained F.S.	Drained F.S.
		thickness d (m)	$c_u$ (kPa)		
10+046 McCarthy Street BH-3BF (East Abutment)	11.5	1.1	20	1.32	1.53
10+102 McCarthy Street BH-1ABP & 1BBP	10.0	3.4	50	1.37	1.49
10+160 McCarthy Street BH-2BP, 3BP, 9BP	9.0	6.3	25	0.99	1.54
	6.0	6.3	25	1.36	-
	9.0 m (2 stage construction)	6.3	38	1.32	-
	9.0 m with 3m high x 10 m wide toe berm	6.3	25	1.17	2.14
13+270 E/W-N Ramp BH-10BP	17.5	6.0	25	1.36	1.72

### 2.5.1.2 Total Stress Analyses With Stabilizing Berms

In areas where embankment heights are greater than 6 m to 7 m height and overlie soft/firm silty clays, placement of fills will require special consideration to prevent short term instability problems. Stabilizing berms may be considered to allow safe, rapid construction of the embankments to full height, while maintaining a minimum factor of safety of 1.3 against foundation failure. Typical berm heights would generally be in the order of about one-third the embankment height. Typical berm dimensions are provided in Table 2-7.

**Table 2.7 Typical Stabilizing Berm Configurations for Short Term FS of 1.3**

Embankment Height	Soft/Firm Silty Clay Thickness	Berm Height	Berm Length
7 m	5 - 6 m	2 m	7 m
8 m	5 - 6 m	3 m	10 m
9 m	5 - 6 m	3.5 m	12 m
10 m	5 - 6 m	3.5 m	14 m

Although the stabilizing berms would enable the embankments to be constructed to full height in one stage, while maintaining adequate factors of safety against foundation failure, the berms may not be the preferred option, not only because of the additional construction costs and potential problems with property allowances, but, more importantly, settlement issues, as discussed in section 2.5.2 below.

### 2.5.1.3 Effective Stress Analyses

The results of our effective stress analyses are provided in Table 2-6 above. The analyses are based on the assumption that the embankments have been constructed to full height and that most of the excess porewater pressures that would have developed in the silty clay stratum have dissipated. Over the assumed construction period of at least 3 months at a steady ramp construction rate, some dissipation of excess porewater pressures will have occurred. However, based on the consolidation test data, in areas of deeper deposits of soft/firm silty clay ( $> 5$  m) and high embankment heights ( $> 7$  m), less than 40% of consolidation will have taken place over the first 3 months ( $\sim 50\%$  after 6 months). The higher embankments will not be sufficiently stable

(i.e. F.S. >1.3) until excess pore water pressures have dissipated to levels of approximately 1/2 of the surcharge weight of embankment fill. Thus, these embankments will require special consideration, such as staged construction or a slower rate of construction. Alternatively, the use of vertical sand or wick drains could also be considered to improve the foundation soils through consolidation in the problem areas for embankment stability and to shorten the construction time by relieving excess pore water pressures.

### 2.5.2 Consolidation Settlement of Embankments

Consolidation settlement calculations have been performed using the results of the consolidation test carried out on a highly compressible sample of silty clay from BH-10BP, along with the results from two consolidation tests conducted on less compressible silty clay material from the South Interchange site. The parameters used for the analyses are summarized in Table 2.8, below.

Table 2.8 Geotechnical Parameters for Settlement Calculations				
	$\gamma_{total}$ (kN/m <sup>3</sup> )	$p_c$ (kPa)	$m_v$ (MPa <sup>-1</sup> )	$c_v$ (m <sup>2</sup> /yr)
Soft-Firm Silty Clay (highly compressible)	16.5	50	0.52	4.7
Firm/Stiff Silty Clay and Silty Clay with Silt layers	18.0	45	0.30	~10.0
Notes: Embankment crest width 17 m, average side slopes = 1.5H:1V				

The results of calculations for embankment centreline settlement using rock fill to construct the embankments, as a function of embankment height and thickness of compressible silty clay, are provided in Table 2-9 below.



**Table 2-9 Estimated Embankment Centreline Consolidation Settlement (mm)**

Embankment Height (m)	Surcharge (kPa)	Silty Clay Thickness (m)			
		2	4	6	8
2	40	30	60	90	110
4	80	60	110	170	230
6	120	90	170	260	340
7	140	100	200	300	400
8	160	110	230	340	460
9	180	130	260	390	510
10	200	140	290	430	570
11	220	160	310	470	630
Proportion of Soft/Firm Silty Clay		67%			
Unit Weight of Fill		20 kN/m <sup>3</sup>			
$m_v$ for soft/firm Silty Clay		0.52 MPa <sup>-1</sup>			
$m_v$ for firm/stiff Silty Clay		0.30 MPa <sup>-1</sup>			
$\mu$ (Skempton-Bjerrum coefficient - $s_s = \mu s_{oed}$ )		0.8			

Specific settlement calculations at various stations along McCarthy Street and the eastern ramps are provided in Table 2-10, below. The silty clay stratum at various locations along the McCarthy Street alignment and eastern ramps was divided into sections considered to be highly compressible and low to moderately compressible, based on the available shear strength, Atterberg Limit results, and water content data. The estimated changes in effective stress on the clay layers, due to the varying heights & weights of fill material, were then applied at various stations to establish the anticipated settlements. Estimated long term settlements (in the areas underlain by compressible silty clays) are approximately 140 mm at the east bridge abutment, and range from 210 mm at station 10+102 (BH-1ABP) to about 400 mm at station 10+160 (BH-2BP) along McCarthy Street, where the combination of fill height (~8.9 m) and the thickness of compressible clay (~5.0 m) is greatest. Settlement of the adjacent EW-N and S-EW ramps varies with the height of fill from less than 200 mm to approximately 370 mm near the McCarthy Street intersections.

In addition to total consolidation settlements, some differential settlement between the embankment centreline and crest will occur, and the design should take these into account by specifying overbuilding of the embankments.

If consideration is to be given to the use of light weight fill, for preliminary design purposes, the total settlements can be reduced approximately by the proportion of rock fill replaced by lightweight polystyrene (i.e., 30% replacement of fill height with polystyrene will result in about 70% of the indicated settlement).

*No - about  
expansion  
10% of  
fill  
slay*

#### 2.5.2.2 Time Rate of Consolidation Settlement

The time rate of consolidation settlement has been calculated for the various cases of embankment height and compressible silty clay thickness at various stations along the McCarthy Street realignment and the eastern ramps, assuming vertical drainage only. The estimated time for 90% consolidation for vertical drainage only was calculated using the average calculated coefficient of consolidation ( $c_v = 4.7 \text{ m}^2/\text{yr}$ ), obtained from the consolidation-time test results obtained from the compressible silty clay sample from borehole BH-10BP. The time for 90% consolidation, after completion of construction of the embankment (assumed to be over a 3 month period), was calculated to range from as little as 1.7 months for shallow deposits of silty clay, to between 20 to over 24 months for soft/firm clay thicknesses of approximately 5.0 m. The estimated remaining settlement after 6 months, 12 months and 18 months was determined at all borehole locations where the silty clay was encountered, with the worst case being at station 10+160 on McCarthy Street, where the remaining settlement was calculated as 167 mm, 87 mm, and 45 mm, respectively (see Table 2.10).

Elastic settlements in the non-plastic sandy and silty soils will occur as the loadings from the fill placement are applied, and therefore will not affect long term performance of the embankments.

**Table 2-10 Summarized Borehole, Embankment Construction & Settlement Data -  
Trout Creek North Interchange Bridge Structure & Approaches**

Borehole & Station Location					Embankment Design				Silty Clay Conditions				Long Term Settlement Estimates				
Borehole	Station/Location	Depth (m)	Refusal	Ground Surf. Elev (m)	Prop. Base EL of Rock Fill (m)	New Pymt Grade (m)	Height of Fill Above O.G (m)	Total Ht of Fill Above Base EL(m)	Thickness of Clay Layer (m)	Est Depth of Soft-Firm Compressible Clay (m)	Est Depth of Stiff or Very Silty, Less Compressible Clay (m)	Equivalent Depth of Soft-Firm Clay (m)	Estimated Change in Effective Stress ( $\Delta \sigma_v$ ) kPa	Est Long Term Settlement (mm)	Estimated Time for 90% Consolidation (months)	Est Remaining Longterm Settlement After 6 Months (mm)	Est Remaining Settlement After 6 Months with Sand/Wick Drains (mm)
Bridge Structure																	
1BF	9+985.5 CL McCarthy Street	12.2		315.3	314.6	317.7	2.4	3.1	4.6	2.0	2.6	3.9	63	80	8	10	5
2BF	9+956.3 O.S. 5.2m R CL McCarthy St	11.3		315.4	315.1	326.7	11.3	11.7	-	-	0.0		233	0	-		
3BF	10+046 O.S. 6.0m L CL McCarthy St	14.3	Y	315.2	314.4	325.6	10.4	11.2	2.1	1.0	1.1	1.8	225	140	2	0	0
Approaches & Ramps - East Interchange																	
1ABP	10+102 CL McCarthy Street	6.6	Y	315.1	314.4	324.5	9.4	10.1	3.3	2.0	1.3	2.9	202	220	5	10	10
1BBP	10+102 CL McCarthy Street	12.6	Y	315.1	314.6	324.5	9.4	9.9	4.5	2.0	2.5	3.8	198	260	8	40	20
2BP	10+160 CL McCarthy Street	14.2	Y	314.7	314.0	322.9	8.2	8.9	6.3	5.0	1.3	5.9	178	410	19	170	30
3BP	10+200 CL McCarthy Street	18.1	Y	314.8	313.9	321.7	6.9	7.8	6.4	5.0	1.4	6.0	157	360	20	150	20
4BP	10+250 CL McCarthy Street	18.8	Y	314.7	313.8	320.3	5.6	6.5	6.7	4.5	2.2	6.1	130	300	20	130	20
5BP	10+300 CL McCarthy Street	15.7	Y	314.6	313.8	318.7	4.1	4.9	7.3	4.0	3.3	6.4	97	220	22	100	10
9BP	13+150 CL S-E/W Ramp	19.7	Y	314.6	313.7	322.4	7.8	8.7	5.8	5.0	0.8	5.6	174	380	17	150	20
10BP	13+270 CL East/West-North Ramp	14.2	Y	315.1	314.3	319.7	4.6	5.4	5.9	5.0	0.9	5.7	108	240	17	90	10
11BP	13+350 CL East/West-North Ramp	17.3		314.9	314.1	318.2	3.3	4.1	7.5	4.6	2.9	6.7	83	200	24	100	10
Approaches & Ramps - West Interchange																	
12BP	9+850 CL McCarthy Street	3.73		316.8	316.5	327.2	10.4	10.7	-	-	0.0		213	0	-		
13BP	13+430 O.S.5m L, North-East Ramp	1.8	Y	316.8	316.6	326.6	9.8	10.0	-	-	0.0		200	0	-		
14BP	9+800 CL McCarthy Street	3.5	Y	317.8	317.5	327.5	9.7	10.0	-	-	0.0		199	0	-		
15BP	9+750 CL McCarthy Street	0.3	Y	318.6	318.3	327.7	9.1	9.4	-	-	0.0		188	0	-		
16BP	9+700 CL McCarthy Street	2.3	Y	319.4	318.8	328.0	8.6	9.2	0.8	-	0.8		184	30	-		
17BP	9+650 CL McCarthy Street	4.57	Y	321.1	321.0	328.2	7.1	7.2	-	-	0.0		144	0	-		
18BP	9+595 O.S. 2m L McCarthy Street	0.5	Y	322.1	321.7	328.5	6.4	6.9	-	-	0.0		137	0	-		
19BP	9+550 CL McCarthy Street	3.2	Y	322.8	322.5	328.7	5.9	6.2	-	-	0.0		124	0	-		
20BP	9+500 CL McCarthy Street	1.37	Y	323.9	323.3	329.0	5.1	5.7	-	-	0.0		114	0	-		
21BP	9+910 CL McCarthy Street	5.55		315.8	315.8	326.9	11.1	11.1	-	-	0.0		222	0	-		
22BP	13+540 O.S.10m L, N-E/W Ramp	4.17	Y	316.5	319.2	324.4	7.9	5.2	-	-	0.0		104	0	-		
23BP	13+600 O.S.18m L, N-E/W Ramp	8.08	Y	319.4	319.3	323.2	3.8	3.9	-	-	0.0		78	0	-		

### 2.5.3 Sand or Wick Drain Design and Construction Considerations

The above time rate of consolidation settlements have been calculated for the various cases of embankment height and clay thickness for vertical drainage only. Analyses have also been carried out assuming the installation of vertical sand or wick drains, spaced in a triangular pattern. It was also assumed for the purposes of calculation, that all embankments are raised steadily over a 3 month construction period. A coefficient of consolidation,  $c_v$ , of  $4.7 \text{ m}^2/\text{year}$  applies for the compressible silty clays underlying the eastern section of the North Interchange, based on the results of our consolidation tests and geotechnical literature. For radial drainage, a ratio of horizontal to vertical coefficient of consolidation of 3 has been assumed, based on horizontal consolidation test data from the South Interchange site. Detailed vertical sand/wick drain design is beyond the scope of this phase of the project.

*C<sub>h</sub> = 3 C<sub>v</sub>*

The use of sand or wick drains under the more critical embankments (i.e. in the triangular shaped problem area in the vicinity of Station 10+160 on McCarthy Street and the adjacent ramps) would increase the rate of consolidation by a factor of approximately 3 to 4. The length of pre-loading time, such that remaining long term settlements are less than 25 mm, could be reduced from 18 to 24 months to approximately 6 months. With respect to stability, by increasing the rate of pore water pressure dissipation and consolidation of the silty clays, it is estimated that installation of sand/wick drains would also increase the shear strength of the silty clays, thus allowing construction of the embankments to full height within about 6 months, while maintaining an adequate factor of safety.

The optimum grid spacing for a wick or sand drain has been calculated as approximately 3.0 m for a triangular pattern, assuming a 300 mm diameter sand drain or equivalent capacity wick drain.

It is expected that larger wick drain spacings (and perhaps no drains) could be used in some of the non-critical areas, such as near to the toes of the embankments, as well as where total settlements can confidently be completed in a relatively short period of time. We expect that by optimizing the drain installation pattern once the final embankment configuration for the project is established, along with scheduling constraints, a significant savings in the drain installation costs will result, compared to specifying a single spacing for the entire project.

It is expected that if vertical drainage is to be implemented on this project, drainage wicks will be selected over sand drains, primarily due to cost. It is noted that either should perform similarly, provided that in the case of drainage wicks (or prefabricated vertical drains), their discharge capacity is not impaired by kinking that will reduce their efficiency. Vertical strains of up to about 6% have been calculated due to consolidation settlement. Systems such as those with a filter fabric covered ribbed core construction should be used, since these appear to better maintain their hydraulic properties than the corrugated or grooved types, for example, when kinking or folding occurs, or due to the high lateral pressures at depth within the consolidating soil.

Additionally, these drains should fully penetrate the clay and underlying silt, and be terminated a minimum of 0.5 m into the underlying silty sand and gravel till to ensure adequate drainage.

#### **2.5.4 Secondary Compression of Clays**

Calculations have been performed to estimate the magnitudes of secondary compression of the foundation clays, following the completion of primary consolidation. For purposes of analysis and discussion, the primary consolidation is assumed to be essentially complete within about 2 years from the start of construction of the embankments. For this assumption to be valid, vertical sand drains or drainage wicks will be required for much of the project.

The stress increases in the clay foundation soils due to embankment loading will, in most cases, exceed the preconsolidation pressures when the embankment height exceeds about 2.5 m. The anticipated embankment centerline settlement due to secondary compression is estimated to be approximately 10% of the total settlement and would be expected to be virtually completed in about 15 years to 20 years. The calculations are based on use of a coefficient of secondary compression,  $C_{\alpha}$ , of 0.004, based on the results of the consolidation tests and the geotechnical literature.

As an example, the anticipated secondary silty clay compression at Stations 10+160 on McCarthy Road, where the silty clay thickness is approximately 6.7 m, after 5, 10 and 20 years may be about 10 mm, 20 mm and 40 mm, respectively.

### **2.5.5 Rockfill and Rockfill Settlement**

Rockfill used in the embankments and approaches should consist of hard, durable, angular quarried rock. Rockfill should be tested for acid drainage potential and checked against the appropriate regulatory requirements for acceptability.

For the bridge abutments that are to be supported by driven steel H-piles, as discussed in a previous section, these will have to be installed after all the embankment fill has been placed. The driving of steel piles through rockfill will be difficult, if not impossible, unless care is taken to ensure that the rockfill meets a tight size range, generally between 50 mm and 150 mm. For the majority of the embankments and approach fills, however, larger rockfill can be used.

The use of large rockfill, loosely placed or end dumped and compacted by conventional construction traffic only, may result in long term settlements of up to about 2% of the fill height. Accordingly, a 10 m high rockfill may settle up to 200 mm following completion of construction. This is primarily due to the rockfill adjusting under highway service conditions.

In order to minimize the long term settlements associated with the rockfill itself, consideration can be given to specifying the use of a finer graded rockfill, with a maximum size of 400 mm. This finer graded rockfill should be placed in lifts limited to about 800 mm and compacted with heavy vibratory rollers. While the production of the finer graded rockfill and its placement, as described above, carries with it a cost premium, the long term performance of the finished highway pavement will be improved.

## **2.6 Culverts**

While we have not specifically been requested to address culverts, it is expected that these will be required as part of the site drainage plan. The number of culverts and their locations are not presently known. Because of the settlement issues discussed previously, any centerline culverts installed prior to the embankment construction will settle with the embankment. Differential settlements between the embankment centerline and the toe may be as high as several hundred millimetres. This must be considered in the design and several options are presented for your consideration.

Preferably, culverts should be located in areas where smaller settlements are expected, such as under the lower height embankments and thinner clay areas.

If culverts must be located in areas of greater potential settlement, they can be designed with a reverse camber to accommodate some of the differential settlement. In this case, they should be oversized and standing water at the inlets and/or outlets should be tolerable, since it will be impossible to predict (ensure) a positive final grade along the length of the culvert.

Alternatively, culverts can be supported on piled foundations. It is expected, however, that this technique will result in differential settlements between the pile supported culvert (essentially zero settlement) and the highway on either side, resulting in a pronounced "bump".

Finally, culverts located in higher settlement areas may be installed following completion of the majority of the associated embankment settlement. In some cases, this may require deep excavations into the embankment fills.

Not  
Practical

## 2.7 Instrumentation and Construction Monitoring

It is recommended that monitoring of the embankments be carried out during and after construction to verify the design recommendations using a properly designed and effective system of instrumentation, consisting primarily of piezometers installed within the silty clay, as well as settlement points. Lateral slope movements should also be monitored both during and following construction of the embankments. Monitoring and continual interpretation of the data is essential in order to control the rate of excess porewater pressure generation and dissipation within the silty clay soils by controlling the rate of embankment fill placement. Excess porewater pressures must be kept below calculated threshold values that may indicate impending failure. This may require a slowing or stoppage of construction until pore pressures dissipate sufficiently to maintain stability and allow construction to resume. The monitoring may, on the other hand, indicate that construction can proceed faster than originally planned, resulting in potential cost savings.

Notwithstanding the above, if stabilizing berms are designed to allow rapid construction of portions of the embankments, a monitoring program is still recommended. The use of berms is based on total stress analyses and design, and does not require the construction monitoring of excess porewater pressures. However, some porewater pressure monitoring, as well as settlement monitoring, is recommended to allow for modifications to the final grading design, if necessary. This is primarily related to the amount of overbuilding of the final grades, in anticipation of the calculated settlements.

Detailed design of a construction monitoring program is beyond the scope of this phase of the project, but can be provided when the final design is nearing completion.

## 2.8 Closing Comments

The recommendations made in this report are in accordance with our present understanding of the project and are provided solely for the use of Marshall Macklin Monaghan and their design team for the design of the bridge foundations, approach fills and embankments for the North Interchange of the Highway 11 Trout Creek By-Pass Project. We request that we be retained to review the design and our recommendations as the design proceeds, to ensure that the final design is in general agreement with our recommendations and that our recommendations have been interpreted as intended.

A subsurface investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those reported at the test locations, we require that we be notified immediately in order to allow reassessment of our recommendations. It may then be necessary to carry out additional field work and analyses.

Contractors bidding on or undertaking the works should, relative to the subsurface conditions, decide on their own investigations, if deemed necessary, as well as their own interpretations of the factual results provided herein, so that they may draw their own conclusions as to how the subsurface conditions may affect them.



The information presented in this report is based on a limited investigation designed to provide information to support an overall assessment of the current geotechnical conditions at the site of the proposed McCarthy Road and King's Highway 11 (Trout Creek By-Pass North Interchange). The conclusions presented in this report reflect site conditions existing at the time of the investigation. It is noted that the soil boundaries indicated on the logs are inferred from discontinuous sampling and observations during drilling. These boundaries are intended to reflect transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change.

This report has been prepared by Mr. A.J. Schell, P.Eng. and was reviewed by Messrs. S.E. Gonsalves, P.Eng., and E. Gonneau, P.Eng. The field investigation was performed by Messrs. I. Dumpis, C.E.T. and R. Moore, C.E.T., and was supervised by Mr. E.A. Gonneau, P.Eng.

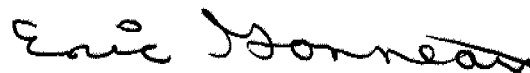
We trust this report is satisfactory for your purposes. Should you have any questions, please do not hesitate to contact us.

All the foregoing and attachments respectfully submitted,

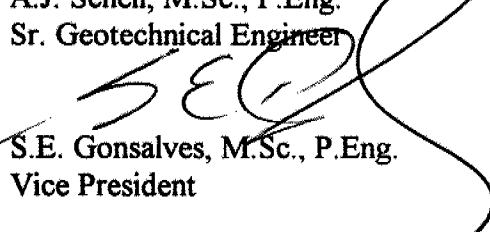
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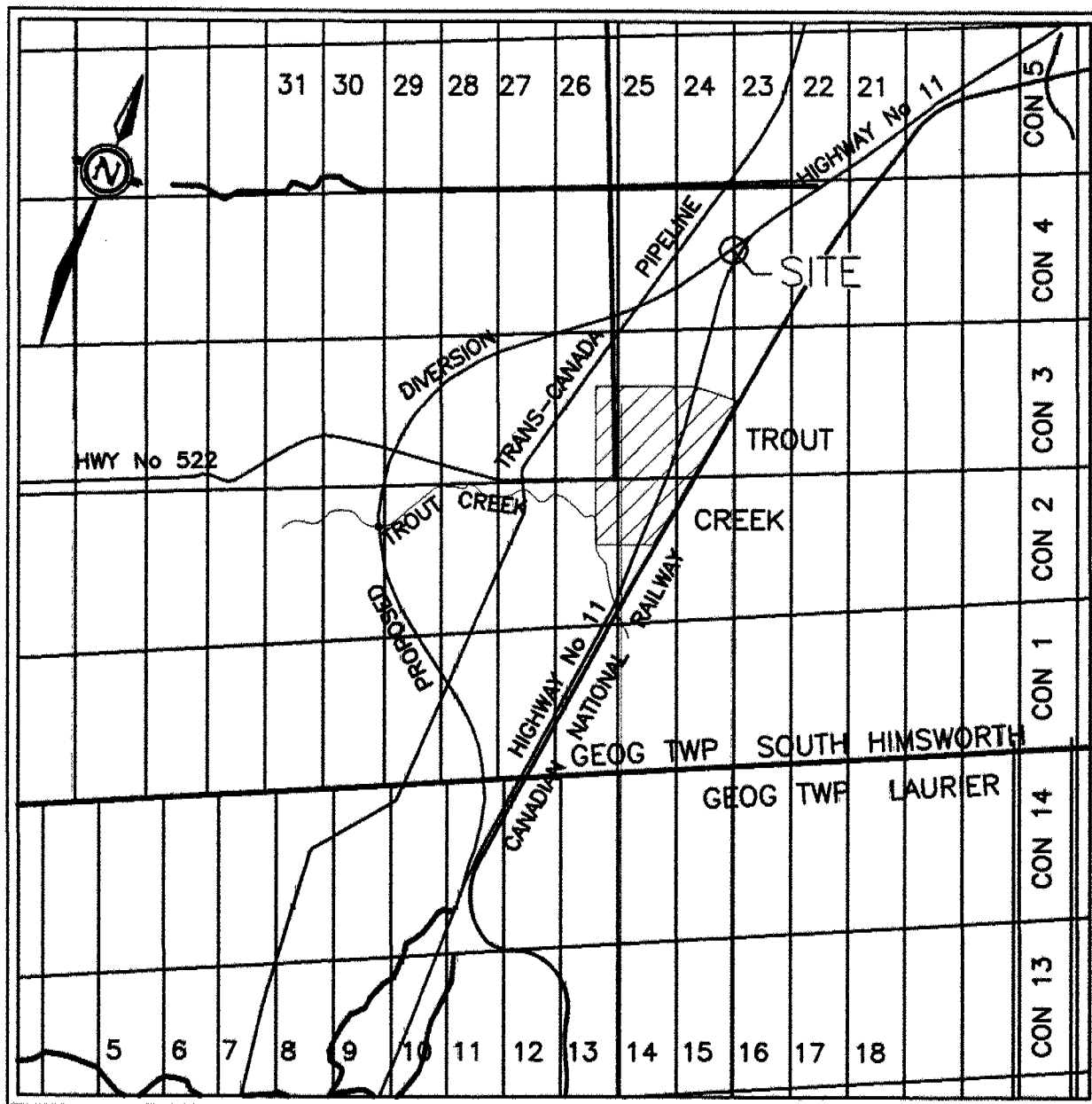
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**APPENDIX A**  
**DRAWINGS AND CROSS SECTIONS**

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## KEY PLAN

1 km 0 1 km

SCALE 1:50000

**Trow**

GEOTECHNICAL EVALUATION

**TROUT CREEK NORTH INTERCHANGE**

**KEY PLAN**

**TROUT CREEK BYPASS, W.P. 774-93-00**

PROJ. No. S075246/B

DATE: JANUARY 1999

DWG. No. 1A

# OVERSIZE DRAWING(S)

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## **APPENDIX B**

### **BOREHOLE LOGS AND ROCK DESCRIPTIONS**

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**NOTES ON SAMPLE DESCRIPTIONS**

1. All descriptions included in this report follow the I.S.S.M.F.E. as suggested in the Canadian Foundation Manual. The laboratory grain-size analysis also follows this classification system. Others may designate the unified classification system as their source; a comparison of the two is shown for your information. Please note that, with the exception of those samples where the grain-size analysis has been carried out, all samples are classified visually and the accuracy of visual examination is not sufficient to differentiate between the classification systems or exact grain sizing.

UNIFIED SOIL CLASSIFICATION											
	Fines (silt or clay)				Sand			Gravel		Cobbles	
					Fine	Medium	Coarse	Fine	Coarse		
I.S.S.M.F.E. SOIL CLASSIFICATION	Clay	Silt			Sand			Gravel			Cobbles
		Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse	
Sieve Sizes											
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2. **FILL:** Where fill is designated on the borehole log, it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of the site fill material. All fills should be expected to contain obstructions such as large concrete pieces of subsurface basements, floors, tanks, etc.; none of these may have been encountered in the borehole. Since boreholes cannot accurately define the contents of fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact and correct composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant on-going and future settlements. Some fill material may be contaminated by toxic waste that renders it unacceptable for deposition in any but designated land fill sites. Unless specifically stated, the fill on this site has not been tested for contaminants that may be considered hazardous. This testing and a potential hazard study can be carried out if you so request. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common but are not detectable using conventional geotechnical procedures.
3. **TILL:** The term till on the borehole logs indicate that the material originates from a geological process associated with glaciation. As a result of this geological process, the till must be considered heterogeneous in composition and, as such, may contain pockets and/or seams of material such as sand, gravel silt or clay. As till often contains cobbles (60 to 200 mm) or boulders (over 200 mm), contractors may encounter them during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size, or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited areas; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till material.

# NOTES ON SAMPLE DESCRIPTIONS (Cont'd)



Project No: S07524G/B

Drawing No: 2B

4. The following table gives a description of the soil based on particle sizes. With the exception of those samples where grain-size analyses have been performed, all samples are classified visually. The accuracy of visual examination is not sufficient to differentiate between this classification system or exact grain size.

Soil Classification		Terminology	Proportion
Clay	< 0.002 mm	"trace" (eg. trace sand)	1% - 10%
Silt	0.002 to 0.06 mm	"some" (eg. some sand)	10% - 20%
Sand	0.06 to 2 mm	adjective (eg. sandy)	20% - 35%
Gravel	2 to 60 mm	and (eg. and sand)	> 35%
Cobbles	60 to 200 mm	noun (eg. boulders)	> 35% and main fraction
Boulders	> 200 mm		

Classification system as suggested in the Canadian Foundation Engineering Manual, 3rd Edition, unless otherwise noted.

The compactness of cohesionless soils and the consistency of cohesive soils are defined by the following:

Cohesionless Soil		Cohesive Soil	
Compactness	Standard Penetration Resistance "N" Blows/0.3 m	Consistency	Undrained Shear Strength (kPa)
Very Loose	0 to 4	Very Soft	< 12
Loose	4 to 10	Soft	12 - 25
Compact	10 to 30	Firm	25 - 50
Dense	30 to 50	Stiff	50 - 100
Very Dense	Over 50	Very Stiff	100 - 200
		Hard	> 200

## 5. Rock Coring

Where rock drilling was carried out, the term RQD (Rock Quality Designation) is used. The RQD is an indirect measure of the number of fractures and soundness of the rock mass. It is obtained from the rock cores by summing the length of core recovered, counting only those pieces of sound core that are 100 mm or more in length. The RQD value is expressed as a percentage and is the ratio of the summed core lengths to the total length of core run. The classification based on the RQD value is given below.

RQD Classification	RQD
Very poor quality	< 25
Poor quality	25 - 50
Fair quality	50 - 75
Good quality	75 - 90
Excellent quality	90 - 100

$$\text{Recovery Designation \% Recovery} = \frac{\text{Length of Core Per Run}}{\text{Total Length of Run}} \times 100$$

# RECORD OF BOREHOLE BH-1BF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 090 923.9 N, 320 159.7 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow Stem Augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE May 25, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20    40    60    80				wp ——— w ——— wl				
								SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)				
								UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR							
						20    40    60    80				10    20    30    40		GR    SA    (SI & CL)				
315.26	GROUND SURFACE															
0.00	PEAT (very soft)															
314.56	SAND, brown, wet, organic staining in upper ~1.0 m depth. (loose to compact)		1	SS	8									0% 95% 5%		
0.70			2	SS	12											
			3	SS	8											
312.56	SILTY CLAY, numerous wet SILT seams, grey. (stiff to ~5.0 m depth then soft to firm)		4	SS	7											
2.70			5	SS	2											
			6	SS	5											
	SILT, grey, wet, layers of fine sand throughout, with occasional thin clay seams. (loose)		7	SS	5									0% 11% 89%		
			8	SS	8											
			9	SS	8											
	A few gravel sizes at base. (possible TILL layer)		10	SS	26											
303.07	BIOTITE HORNBLENDE GNEISS, good to excellent rock quality, unweathered.		11	BQ										Rec 100% RQD 90%		
12.19			12	BQ												
300.02	END OF BOREHOLE															
15.24	Note: 1) This borehole forms part of the North Interchange Foundation Investigation. 2) Borehole located at station 9+998.5, on centreline as referenced to McCarthy Street. 3) Water level was at ~0.4 m & hole was open to ~0.5 m depth on completion.															





# RECORD OF BOREHOLE BH-2BF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 090 964.9 N, 320 170.8 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow Stem Augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE May 26, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION		
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20      40      60      80				wp      —      w      —      wl						
								SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)						
								UNCONFINED QUICK TRIAXIAL      FIELD VANE LAB SHEAR										
						20      40      60      80				10      20      30      40			kN/m <sup>3</sup> GR      SA      (SI & CL)					
315.41	GROUND SURFACE																	
312.08 0.36	PEAT (very soft)																	
	SAND, brown, wet, organic staining in upper ~1.0 m depth. (loose)		1	SS	8											0%	94% 6%	
312.41																		
3.00	SILT, grey, wet, occasional seams of stiff clay in upper ~2.0 m depth, layers of fine sand throughout. (loose)		2	SS	14													
			3	SS	6											0%	4% 96%	
			4	SS	7											0%	0% 100%	
			5	SS	7													
306.41 9.00	SILTY SAND & GRAVEL TILL, some cobble sizes & possible boulders, grey, wet. (very dense)		6	SS	75													
			7	SS	101											11%	72% 17%	
304.13 11.28	BIOTITE HORNBLende GNEISS, very poor/poor quality & weathered to ~14.0 m depth, then changing to fair quality, slightly weathered rock.		8	BQ												Rec	90% RQD 32%	
			9	BQ												Rec	80% RQD 20%	
			10	BQ												Rec	100% RQD 63%	
289.56 15.85	END OF BOREHOLE																	
<p>Note: 1) This borehole forms part of the North Interchange Foundation Investigation. 2) Borehole located at station 9+956.3, ~5.2 m right of centreline as referenced to McCarthy Street. 3) Borehole caved wet at ~0.3 m depth on completion.</p>																		



# RECORD OF BOREHOLE BH-3BF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 090 878.9 N, 320 147.0 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow Stem Augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE May 27, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			SHEAR STRENGTH: Cu, KPa		WATER CONTENT (%)				
								UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR	wp	wl			
315.17	GROUND SURFACE												GR SA (SI & CL)	
0.00	PEAT (very soft)													
314.36	SAND, brown, wet, organic staining in upper ~1.0 m depth. (loose to compact)		1	SS	6							380	0% 95% 5%	
0.81			2	SS	14									
312.27	SILTY CLAY, stratified with wet SILT seams, grey. (stiff to ~4.0 m depth then soft to firm)		3	SS	8								0% 0% 100%	
2.90			4	SS	4									
310.17	SILT, grey, wet, layers of fine sand throughout, with occasional thin clay seams. (loose)		5	SS	5								0% 0% 100%	
5.00			6	SS	4									
			7	SS	6									
			8	SS	6									
303.17			9	SS	47									18% 52% 30%
12.00		SILTY SAND & GRAVEL TILL, some cobble sizes & possible boulders, grey, wet. (very dense)		10	SS	57								
300.87	BIOTITE HORNBLENDE GNEISS, good to excellent rock quality, unweathered.		11	BQ									Rec. 100% RQD 99%	
14.30			12	BQ										
297.74														Rec. 100% RQD 80%
17.43	END OF BOREHOLE													
Note: 1) This borehole forms part of the North Interchange Foundation Investigation. 2) Borehole located at station 10+046.0, ~6.0 m left of centreline as referenced to McCarthy Street.														



1 OF 1

METRIC

W.P. 774-93-00

LOCATION 5 090 921.2 N, 320 158.6 E

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Dynamic Cone / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE May 25, 1998

CHECKED BY I.G.

[illegible]

1 OF 1

METRIC

W.P. 774-93-00

LOCATION 5 090 960.7 N, 320 181.2 E

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Dynamic Cone / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE May 26, 1998

CHECKED BY I.G.

[illegible]

# RECORD OF BOREHOLE C-3BF

1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 090 881.9 N. 320 137.2 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Dynamic Cone / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE May 27, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)				CONE PENETRATION TEST			WATER CONTENT (%)	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	UNCONFINED QUICK TRIAXIAL	SHEAR STRENGTH: Cu, KPa	FIELD VANE LAB SHEAR	PLASTIC LIMIT	NATURAL MOISTURE CONTENT			
315.17	GROUND SURFACE														
0.00	Dynamic cone penetration test only.														
314															
313															
312															
311															
310															
309															
308															
307															
306															
305															
304															
303															
302															
301															
300.74	END OF CONE TEST BOUNCING REFUSAL ON BEDROCK OR BOULDER														
14.43	Note: 1) This cone test forms part of the North Interchange Foundation Investigation. 2) Cone test located at station 10+046.0, ~5.2 m right of centreline as referenced to McCarthy Street.														



# RECORD OF BOREHOLE BH-1ABP 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION 5 095 303.7 N, 315 324.9 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow Stem Augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE May 28, 1998 CHECKED BY A.S.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20					
315.08	GROUND SURFACE												
0.00	PEAT, black, roots, wet. (very soft)												
314.38	SAND, trace to some SILT, brown, wet. (loose)		1	SS	9								0% 95% 5%
0.70			2	SS	14								
			3	SS	13								
			4	SS	9								
311.78	SILTY CLAY, with SILT layers and fine SAND seams, grey, wet. (firm/stiff)		5	SS	3								
3.30													
308.53	END OF BOREHOLE		6	SS	2								
6.55	<p>Note:</p> <p>1) This borehole forms part of the North Interchange Approach Foundation Investigation.</p> <p>2) Borehole located at station 10+102, on centreline as referenced to McCarthy Street.</p> <p>3) Water level was at ~0.3 m &amp; hole was open to ~0.9 m depth on completion.</p> <p>4) Borehole was terminated prematurely due to a nearby advancing forest fire.</p>												



# RECORD OF BOREHOLE BH-1BBP 1 OF 1

## METRIC

W.P. 774-93-00	LOCATION 5 095 303.7 N, 315 324.9 E	ORIGINATED BY R.M.
DIST 54 HWY 11	BOREHOLE TYPE Wash boring / Motorized tripod	COMPILED BY M.D.
DATUM Geodetic	DATE July 5, 1998	CHECKED BY A.S.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20 40 60 80		wp — w — wl				
								SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL    FIELD VANE LAB SHEAR		WATER CONTENT (%) 10 20 30 40				
315.08	GROUND SURFACE													
0.00	PEAT, black, wet. (very soft)		1	SS	3									
314.58	SAND, trace to some SILT, brown to grey brown, moist. (compact to loose)		2	SS	10									
0.50			3	SS	29									
			4	SS	6									
312.48			5	TW										
2.60	SILTY CLAY, with SILT layers, grey, moist. (stiff to very stiff)		6	SS	2									
			7	TW										
307.98	SILT, with thin CLAY seams, fine SAND seams below ~10 m depth, grey, wet. (loose)		8	SS	7									
7.10			9	TW										
302.98	SILTY SAND & GRAVEL TILL, grey brown, wet. (compact)		10	SS	7									
12.10			11	SS	17									
302.51	END OF BOREHOLE DUE TO REFUSAL TO CASING ON BEDROCK OR BOULDER													
12.57														
Note: 1) This borehole forms part of the North Interchange Approach Foundation Investigation. 2) Borehole located at station ~10+102, on centreline as referenced to McCarthy Street. 3) Water level was at surface & hole was open to ~12.0 m depth on completion.														

**Note:**  
 1) This borehole forms part of the North Interchange Approach Foundation Investigation.  
 2) Borehole located at station 10+102, on centreline as referenced to McCarthy Street.  
 3) Water level was at surface & hole was open to 12.0 m depth on completion.



# RECORD OF BOREHOLE BH-2BP 1 OF 1

METRIC

W.P. 774-93-00

LOCATION 5 095 281.8 N, 315 378.6 E

ORIGINATED BY S.M.

DIST 54 HWY 11

BOREHOLE TYPE Hollow Stem Augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE July 17, 1998

CHECKED BY A.S.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20 40 60 80	20 40 60 80	wp — w — wl	WATER CONTENT (%) 10 20 30 40		
314.74	GROUND SURFACE											
0.00	PEAT, brown to black, roots, wet. (very soft)											
313.98												
0.76	SILTY SAND, grey, wet. (loose)		1	SS	9							0% 61% 39%
312.74												
2.00	SAND, fine, trace of SILT, brown, wet. (compact)		2	SS	13							
310.94												
3.80	SILTY CLAY, stratified with wet SILT layers, grey, wet. (soft to firm)		3	SS	0							0% 4% 96%
			4	TW								
			5	SS	1							
			6	SS	8							
304.64												
10.10	SILT, some clay layers in upper sections, occasional sand seams in lower sections, grey. (loose to compact)		7	SS	7							0% 3% 97%
			8	SS	9							
			9	SS	13							
300.57	END OF BOREHOLE											
14.17	Notes: 1) This borehole forms part of the North Interchange Approach Foundation Investigation. 2) Borehole located at station ~10 + 160, on centreline as referenced to McCarthy Street. 3) Water level was at surface & hole was open to ~4.5 m depth on completion.											





# RECORD OF BOREHOLE BH-3BP 1 OF 1

METRIC

W.P. 774-93-00

LOCATION 5 095 266.7 N, 315 415.7 E

ORIGINATED BY S.M.

DIST 54 HWY 11

BOREHOLE TYPE Hollow Stem Augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE July 15, 1998

CHECKED BY A.S.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT KN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20	40	60	80					
314.77	GROUND SURFACE														
0.00	PEAT, black to brown, wet. (very soft)														
313.86															
0.91	SILTY SAND, brown, wet. (loose)		1	SS	9										
312.27															
2.50	SAND, fine to medium, some SILT, brown, wet. (compact)		2	SS	13										
309.57															
5.20	SILTY CLAY, with SILT seams increasing in thickness below ~9.4 m depth. (firm to very stiff)		3	SS	25										0% 84% 16%
303.17															
11.60	SILT, trace of fine sand, some thin clay seams, grey, wet. (loose to compact)		4	TW											
			5	SS	0										
			6	SS	3										
			7	SS	5										
			8	SS	9										0% 9% 91%
			9	SS	10										
			10	SS	10										
299.62															
16.15	SILTY SAND & GRAVEL TILL, with cobbles. (compact to dense)		11	SS	29										
296.63															
18.14	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER														
Notes: 1) This borehole forms part of the North Interchange Approach Foundation Investigation. 2) Borehole located at station ~10+200, on centreline as referenced to McCarthy Street. 3) Water level was at surface & hole was open to ~3.8 m depth on completion.															



# RECORD OF BOREHOLE BH-4BP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 095 247.6 N, 315 461.9 E ORIGINATED BY S.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow Stem Augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE July 15, 1998 CHECKED BY A.S.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20	40	60	80					
314.69	GROUND SURFACE															
0.00	PEAT, black brown, wet. (very soft)															
313.78							314									
0.91	SAND, fine, some SILT, with occasional roots, brown, wet. (compact)		1	SS	12		313									
			2	SS	12		312									0% 85% 15%
310.69							311									
4.00	SANDY SILT, grey brown to grey, wet. (compact to loose)		3	SS	26		310									
			4	SS	7		309									
307.69							308									0% 24% 76%
7.00	SILT, grey, wet. (loose)		5	SS	9		307									
306.19							306									
8.50	SILTY CLAY, with SILT layers, increasing SILT content with depth, grey, wet. (firm to stiff)		6	TW			305									
			7	SS	5		304									
			8	SS	2		303									
			9	SS	9		302									
299.49							301									
15.20	SILT, with thin CLAY layers (decreasing frequency with depth), increasing fine SAND layers at depth. (loose to compact)		10	SS	7		300									
			11	SS	10		299									
			12	SS	17		298									
295.94							297									
18.75	END OF BOREHOLE						296									
Notes: 1) This borehole forms part of the North Interchange Approach Foundation Investigation. 2) Borehole located at station -10+250, on centreline as referenced to McCarthy Street. 3) Water level was at surface & hole was open to -4.8 m depth on completion.																



# RECORD OF BOREHOLE BH-5BP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 095 225.0 N, 315 506.4 E ORIGINATED BY S.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow Stem Augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE July 14, 1998 CHECKED BY A.S.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20 40 60 80		wp — w — wl			
								SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL    FIELD VANE LAB SHEAR		WATER CONTENT (%)			
314.60	GROUND SURFACE												
0.00	PEAT, black to dark brown, wet. (very soft)												
313.84													
0.76	SANDY SILT, grey brown to grey, wet. (compact to dense)		1	SS	12								
			2	SS	14								
			3	SS	31								
309.10													
5.50	SILT, trace of fine SAND, grey, wet. (loose to compact)		4	SS	7							0% 26% 74%	
			5	SS	11								
306.20													
8.40	SILTY CLAY, homogeneous to layered with increasing SILT content at depth, grey, wet. (firm to stiff)		6	SS	0								
			7	SS	0								
			8	SS	5								
			9	SS	7								
298.90			10	SS	7								
15.70	END OF BOREHOLE												
<p>Notes:</p> <p>1) This borehole forms part of the North Interchange Approach Foundation Investigation.</p> <p>2) Borehole located at station ~10+300, on centreline as referenced to McCarthy Street.</p> <p>3) Water level was at surface &amp; hole was open to ~2.2 m depth on completion.</p>													



# RECORD OF BOREHOLE BH-9BP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 095 070.1 N, 315 014.3 E ORIGINATED BY S.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow Stem Augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE July 16, 1998 CHECKED BY A.S.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER TYPE			20 40 60 80	20 40 60 80					
314.59	GROUND SURFACE											
0.00	PEAT, black to brown, wet. (very soft)											
313.68												
0.91	SAND, fine to medium, trace to some SILT, grey brown, wet. (compact)		1 SS 14									
312.09												
2.50	SANDY SILT, grey brown, wet. (compact to loose)		2 SS 11									
			3 SS 7									
			4 TW									
307.79												
6.80	SILTY CLAY, homogeneous to layered with SILT and some fine SAND seams, grey, wet. (firm to stiff)		5 SS 0									0% 44% 56%
			6 TW									
			7 SS 2									0% 74% 26%
301.99			8 SS 7									
12.60	SILT, with thin, soft, grey layers, decreasing with depth, SAND layers near base, grey, wet. (loose to compact)		9 SS 9									
			10 SS 9									
			11 SS 14									
			12 SS 19									
295.59												
19.00	SILTY SAND & GRAVEL TILL, with cobbles.											
294.93												
19.66	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER											
Notes: 1) This borehole forms part of the North Interchange Approach Foundation Investigation. 2) Borehole located at station -13+150, on centreline as referenced to the S/E-W Ramp. 3) Water level was at surface & hole was open to -3.8 m depth on completion.												



# RECORD OF BOREHOLE BH-10BP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 095 154 2 N, 315 275.8 E ORIGINATED BY S.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow Stem Augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE July 17, 1998 CHECKED BY A.S.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80	20 40 60 80	20 40 60 80	20 40 60 80					
315.07	GROUND SURFACE						315									
0.00	PEAT, black to brown, wet. (very soft)															
314.31																
0.78	SAND, fine to medium, trace to some SILT, brown, wet. (loose to compact)		1	SS	9		314									
							313									
			2	SS	14		312									
							311									
310.97							310									
4.10	SILTY CLAY, with SILT layers. (soft to firm then stiff below ~9.0 m depth)		3	SS	0		309									
							308									
			4	SS	0		307									
							306									
			5	TW			305									
							304									
			6	SS	4		303									
							302									
305.07							301									
10.00	SILT, with thin CLAY layers, changing to SILT, with fine sandy layers below ~12.0 m depth, grey, wet. (loose to compact)		7	SS	5											
			8	SS	10											
			9	SS	11											
300.90	END OF BOREHOLE															
14.17	Notes: 1) This borehole forms part of the North Interchange Approach Foundation Investigation. 2) Borehole located at station ~13+270, on centreline as referenced to the E/W-N Ramp. 3) Water level was at surface & hole was open to ~6.1 m depth on completion.															



# RECORD OF BOREHOLE BH-11BP 1 OF 1

METRIC

W.P. 774-93-00

LOCATION 5 095 196.2 N, 315 212 2 E

ORIGINATED BY S.M.

DIST 54 HWY 11

BOREHOLE TYPE Hollow Stem Augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE July 16, 1998

CHECKED BY A.S.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20    40    60    80	20    40    60    80	wp    ——— w    ——— wl	WATER CONTENT (%) 10    20    30    40			
314.95	GROUND SURFACE													
0.00	PEAT, black to brown, wet. (very loose)													
314.05														
0.90	SAND, fine to medium, trace to some SILT, brown, wet. (compact)		1	SS	13									
			2	SS	19									
310.45														
4.50	SILTY CLAY, with SILT layers, grey, wet. (soft to firm then stiff to very stiff below ~9.1 m depth)		3	SS	0									
			4	SS	0									
			5	SS	2									
			6	SS	5									
			7	SS	7									
			8	SS	7									
			9	SS	10									
			10	SS	18									
			11	SS	83									
302.95														
12.00	SILT, with CLAY layers in upper stratum, increasing fine SAND seams/layers with depth. (compact to dense at depth)													



# RECORD OF BOREHOLE BH-12BP 1 OF 1

**METRIC**

W.P. 774-93-00 LOCATION 5 095 398.8 N, 315 091.6 E ORIGINATED BY R.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow Stem Augers / D-50 COMPILED BY M.D.  
 DATUM Geodetic DATE July 17, 1998 CHECKED BY A.S.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE		20	40	60	80	wp	w	wl			
316.83	GROUND SURFACE														
316.50	PEAT, black, moist.														
0.30	SANDY SILT, brown to grey, wet. (compact)		1	SS	12										0% 28% 72%
313.83															
3.00	SILTY SAND TILL (dense)		2	SS	41										
313.10			3	AS											
3.73	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER														
Notes: 1) This borehole forms part of the North Interchange Approach Foundation Investigation. 2) Borehole located at station "9 + 850, on centreline as referenced to McCarthy Street. 3) Water level was at "0.3 m & hole was open to "2.4 m depth on completion.															



# RECORD OF BOREHOLE BH-13BP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 095 448.6 N, 315 071.6 E ORIGINATED BY R.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow Stem Augers / D-50 COMPILED BY M.D.  
 DATUM Geodetic DATE July 17, 1998 CHECKED BY A.S.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT wp	NATURAL MOISTURE CONTENT w	LIQUID LIMIT wl	UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20	40	60	80					
316.75	GROUND SURFACE														
0.00	TOPSOIL, ~150 mm over SILTY SAND, brown, moist to wet at ~1.4 m depth.					316									
315.25	SILTY SAND & GRAVEL TILL.		1	SS	50	315									
1.50	brown, wet.														
314.95	(very dense to dense)														
1.80	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER														
	Notes: 1) This borehole forms part of the North Interchange Approach Foundation Investigation. 2) Borehole located at station ~13+430, offset ~2 m left of centreline as referenced to the N-E Ramp. 3) Water level was at ~0.3 m & hole was open to ~2.4 m depth on completion.														





# RECORD OF BOREHOLE BH-14BP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 095 417.7 N, 315 045.3 E ORIGINATED BY R.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow Stem Augers / D-50 COMPILED BY M.D.  
 DATUM Geodetic DATE July 17, 1998 CHECKED BY A.S.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20	40	60	80					
317.84	GROUND SURFACE														
0.00	TOPSOIL, ~300 mm over SILTY SAND, some GRAVEL, brown, wet. (compact)		1	SS	16										
314.84															
3.00															
314.33	SILTY SAND & GRAVEL TILL, brown, moist. (dense)		2	SS	40										
3.51	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER														
	Notes: 1) This borehole forms part of the North Interchange Approach Foundation Investigation. 2) Borehole located at station +9+800, on centreline as referenced to McCarthy Street. 3) Borehole caved wet at ~0.3 m depth on completion.														






# RECORD OF BOREHOLE BH-16BP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 095 455.4 N, 314 952.7 E ORIGINATED BY R.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow Stem Augers / D-50 COMPILED BY M.D.  
 DATUM Geodetic DATE July 17, 1998 CHECKED BY A.S.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) 				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			CONE PENETRATION TEST				wp — w — wl					
319.40	GROUND SURFACE						20	40	60	80						
0.00	PEAT, black to brown, wet. (very soft)															
318.79																
0.61	SANDY SILT, grey, moist.															
317.90																
1.50	SILTY CLAY, grey. (stiff)		1	SS	5											
317.11																
2.29	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER															
<div>Notes: 1) This borehole forms part of the North Interchange Approach Foundation Investigation. 2) Borehole located at station 9+700, on centreline as referenced to McCarthy Street. 3) Borehole caved wet at surface on completion. 4) Drill moved 0.9 m down chainage &amp; met auger refusal at 1.7 m depth.</div>																



# RECORD OF BOREHOLE BH-17BP 1 OF 1

METRIC

W.P. 774-93-00

LOCATION 5 095 474.3 N, 314 906.4 E

ORIGINATED BY R.M.

DIST 54 HWY 11

BOREHOLE TYPE Hollow Stem Augers / D-50

COMPILED BY M.D.

DATUM Geodetic

DATE July 17, 1998

CHECKED BY A.S.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA	NUMBER			TYPE	BLOWS/0.3m	SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)			
						UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR	20	40	60	80	10	20	30	40
321.14	GROUND SURFACE														
0.00	TOPSOIL, ~150 mm over SAND, trace to some SILT, some GRAVEL below ~2.1 m depth, brown, moist then wet below ~1.5 m depth. (compact to loose)		1	SS	14										
			2	SS	6										
316.57	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER														
4.57	Notes: 1) This borehole forms part of the North Interchange Approach Foundation Investigation. 2) Borehole located at station 9+650, on centreline as referenced to McCarthy Street. 3) Water level was at ~1.5 m & hole was open to 1.7 m depth on completion.														



# RECORD OF BOREHOLE BH-18BP 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION 5 095 497.1 N, 314 856.3 E ORIGINATED BY R.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow Stem Augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE July 17, 1998 CHECKED BY A.S.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m	20	40	60	80	wp	w	wl	WATER CONTENT (%)		
322.11	GROUND SURFACE														
0.00 321.65 0.46	PEAT, black to brown, wet.														
	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER														
	Notes: 1) This borehole forms part of the North Interchange Approach Foundation Investigation. 2) Borehole located at station +9+595, offset ~2 m left centreline as referenced to McCarthy Street. 3) Water level was at surface & hole was open to full depth on completion.														



# RECORD OF BOREHOLE BH-19BP 1 OF 1

METRIC

W.P. 774-93-00

LOCATION 5 095 514.8 N, 314 815.0 E

ORIGINATED BY R.M.

DIST 54 HWY 11

BOREHOLE TYPE Hollow Stem Augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE July 17, 1998

CHECKED BY A.S.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA	NUMBER	TYPE	BLOWS/0.3m	20	40	60	80	wp	w	wl			
322.82	GROUND SURFACE														
322.92	PEAT, black to brown, moist.														
0.30	SILTY SAND, some GRAVEL, brown, wet. (compact)		1	SS	16										16% 63% 21%
319.62			2	SS	25										
3.20	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER														
Notes: 1) This borehole forms part of the North Interchange Approach Foundation Investigation. 2) Borehole located at station +9 + 550, on centreline as referenced to McCarthy Street. 3) Water level was at +0.3 & hole was open to +0.8 m depth on completion.															



# RECORD OF BOREHOLE BH-20BP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 095 543.2 N, 314 774.0 E ORIGINATED BY R.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow Stem Augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE July 17, 1998 CHECKED BY A.S.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20	40	60	80					
323.90	GROUND SURFACE															
0.00	PEAT, black to brown, wet. (very soft)															
323.29																
0.61	SAND, brown, wet.		1	AS			323									
322.53																
1.37	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER															
Notes: 1) This borehole forms part of the North Interchange Approach Foundation Investigation. 2) Borehole located at station +9+500, on centreline as referenced to McCarthy Street. 3) Water level was at surface & hole was open to ~0.6 m depth on completion. 4) Drill made 2 more attempts adjacent BH-20BP & met auger refusals at ~0.6 m depths both times.																



# RECORD OF BOREHOLE BH-21BP 1 OF 1

METRIC

W.P. 774-93-00

LOCATION 5 095 376.2 N, 315 147.1 E

ORIGINATED BY R.M.

DIST 54 HWY 11

BOREHOLE TYPE Wash boring / Motorized tripod

COMPILED BY M.D.

DATUM Geodetic

DATE August 6, 1998

CHECKED BY A.S.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m	20	40	60	80	wp	w	wl			
315.81	GROUND SURFACE														
0.00	TOPSOIL, ~125 mm over SAND, fine to fine-medium, trace to some SILT, brown to grey brown, moist. (very loose to compact)		1	SS	3										
			2	SS	13										
			3	SS	20										
313.21			4	SS	12										
2.60	SILT, with some thin CLAY seams, fine SAND seams below ~3.5 m depth, grey, wet. (loose to compact)		5	SS	19										
310.81			6	SS	29										
5.00	SILTY SAND & GRAVEL TILL, grav. wet.		7	SS	18										
310.26															
5.55	(compact)														
	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER														
	Note: 1) This borehole forms part of the North Interchange Approach Foundation Investigation. 2) Borehole located at station 9+910, on centreline as referenced to McCarthy Street. 3) Water level was at surface & hole was open to ~3.7 m depth on completion.														





# RECORD OF BOREHOLE BH-22BP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 095 548.2 N, 315 123.8 E ORIGINATED BY R.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Wash boring / Motorized tripod COMPILED BY M.D.  
 DATUM Geodetic DATE August 9, 1998 CHECKED BY A.S.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80				wp — w — wl					
								SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL X FIELD VANE LAB SHEAR 20 40 60 80				WATER CONTENT (%) 10 20 30 40					
319.43	GROUND SURFACE																
0.00	TOPSOIL, ~200 mm over SAND, fine to fine-medium, brown, moist. (very loose to compact)		1	SS	2		319.2										
			2	SS	16												
			3	SS	16		318										
317.03							317										
2.40	SILTY SAND & GRAVEL TILL, some cobbles, brown, wet. (compact to dense)		4	SS	50												
			5	SS	20		316										
315.26																	
4.17	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER  Note: 1) This borehole forms part of the North Interchange Approach Foundation Investigation. 2) Borehole located at station ~13+540, offset ~10 m left as referenced to N-E/W ramp . 3 ) Water level was at 0.2 m & hole was open to ~3.0 m depth on completion.																



# RECORD OF BOREHOLE BH-23BP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 095 577.2 N, 315 182.7 E ORIGINATED BY R.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Wash boring / Motorized tripod COMPILED BY M.D.  
 DATUM Geodetic DATE August 8, 1998 CHECKED BY A.S.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA	NUMBER	TYPE		20	40	60	80	wp	w	wl			
319.43	GROUND SURFACE														
0.00	TOPSOIL, ~100 mm over SILTY SAND, orange brown to brown, moist. (loose to compact)		1	SS	6										
			2	SS	10										
317.63			3	SS	25										
1.80	SILT, with SAND seams, occasional CLAY seams, grey, wet. (very loose to loose)		4	SS	12										
			5	SS	3										
			6	SS	5										
			7	SS	6										
312.43															
7.00	SILTY SAND & GRAVEL TILL, brown, moist. (very loose)														
311.35			8	SS	59										
8.00	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER														
Note: 1) This borehole forms part of the North Interchange Approach Foundation Investigation. 2) Borehole located at station ~13+600, offset ~18 m left as referenced to N-E/W ramp. 3) Water level was at ~0.9 m & hole was open to ~4.6 m depth on completion.															



**S07524G/B**

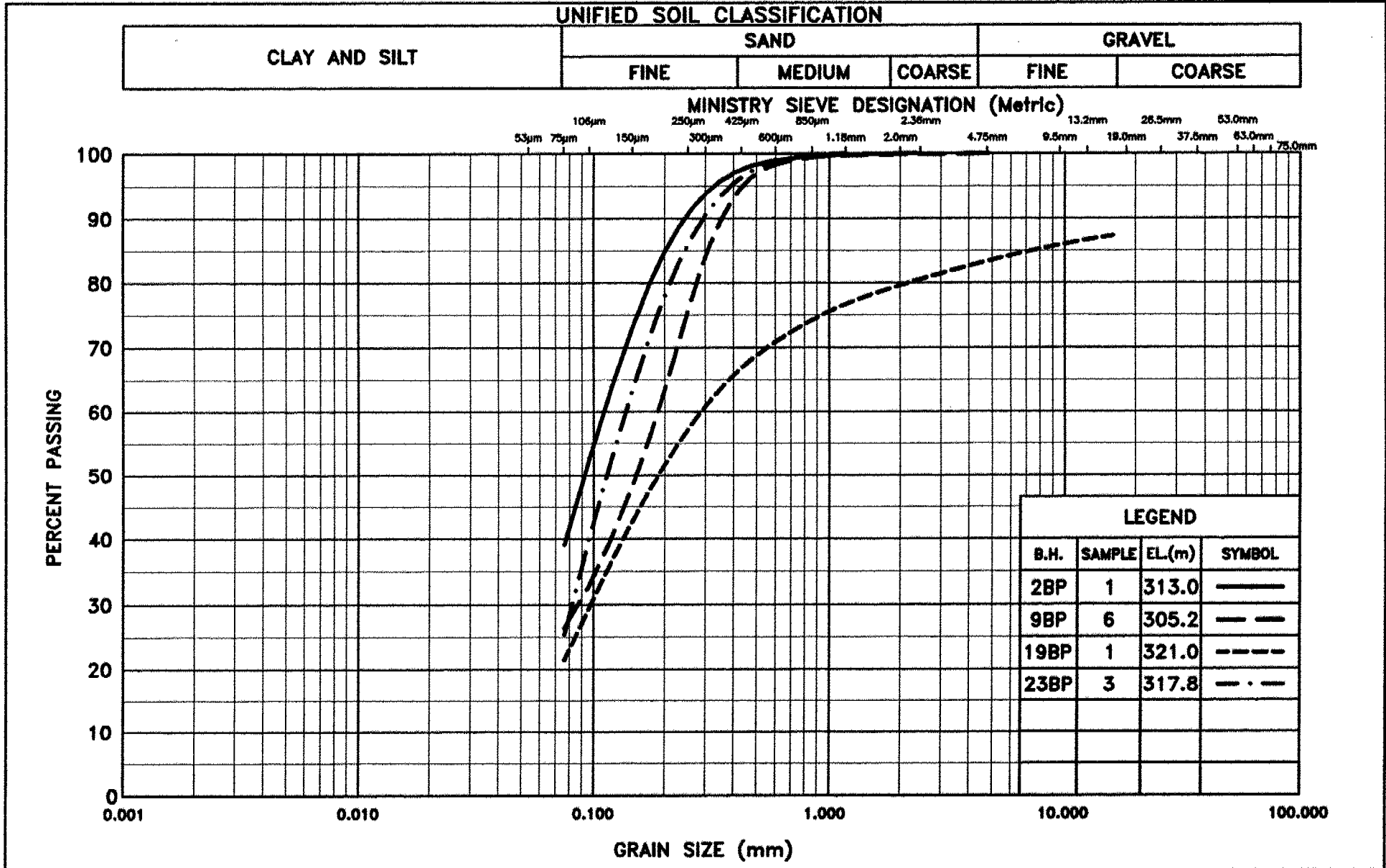
### TABLE B-1 ROCK CORE DESCRIPTION

BH#	Core Recovery				Core Description	
	Rock Core No.	Depth (m)	% CR*	% RQD**	Depth (m)	Description
<b>NORTH INTERCHANGE FOUNDATIONS</b>						
1BF	11	12.28 to 13.72	100	90	12.28 to 15.24	<b>Biotite Hornblende Gneiss (Garnetiferous)</b> , light grey to pinkish white, medium to coarse grained, unweathered, fractures very widely spaced, dipping at 0 to 10° and 45° to 90° from vertical, planar, smooth
	12	13.72 to 15.24	100	88		
2BF	8	11.28 to 12.80	90	32	11.28 to 15.85	<b>Biotite Hornblende Gneiss</b> , white to grey-pink, medium to coarse grained, weathered, fractures, closely spaced to 14 m, dipping at 0 to 45° from vertical, planar, smooth
	9	12.80 to 14.33	80	20		
	10	14.33 to 15.85	100	63		
3BF	11	14.42 to 15.85	100	99	14.42 to 17.43	<b>Biotite Hornblende Gneiss (Garnetiferous)</b> , white to grey-pink, medium to coarse grained, unweathered, fractures widely spaced, dipping at 80° to 90° from vertical, planar, rough
	12	15.85 to 17.43	100	80		

\*CR = Core Recovery  
\*\*RQD = Rock Quality Designation

## **APPENDIX C**

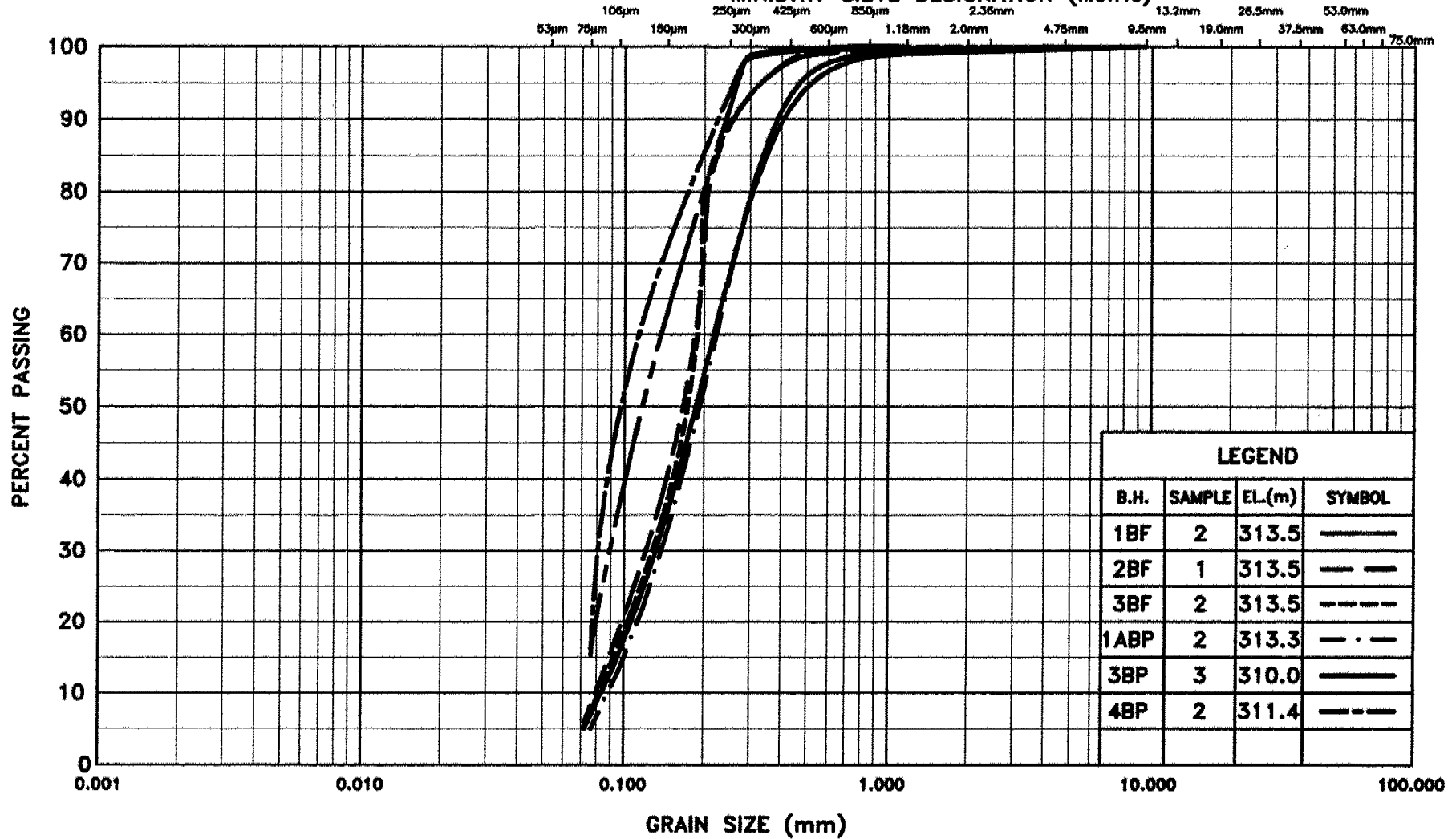
### **RESULTS OF LABORATORY TESTING**



# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION

SAND

FIGURE 2

W.P 774-93-00

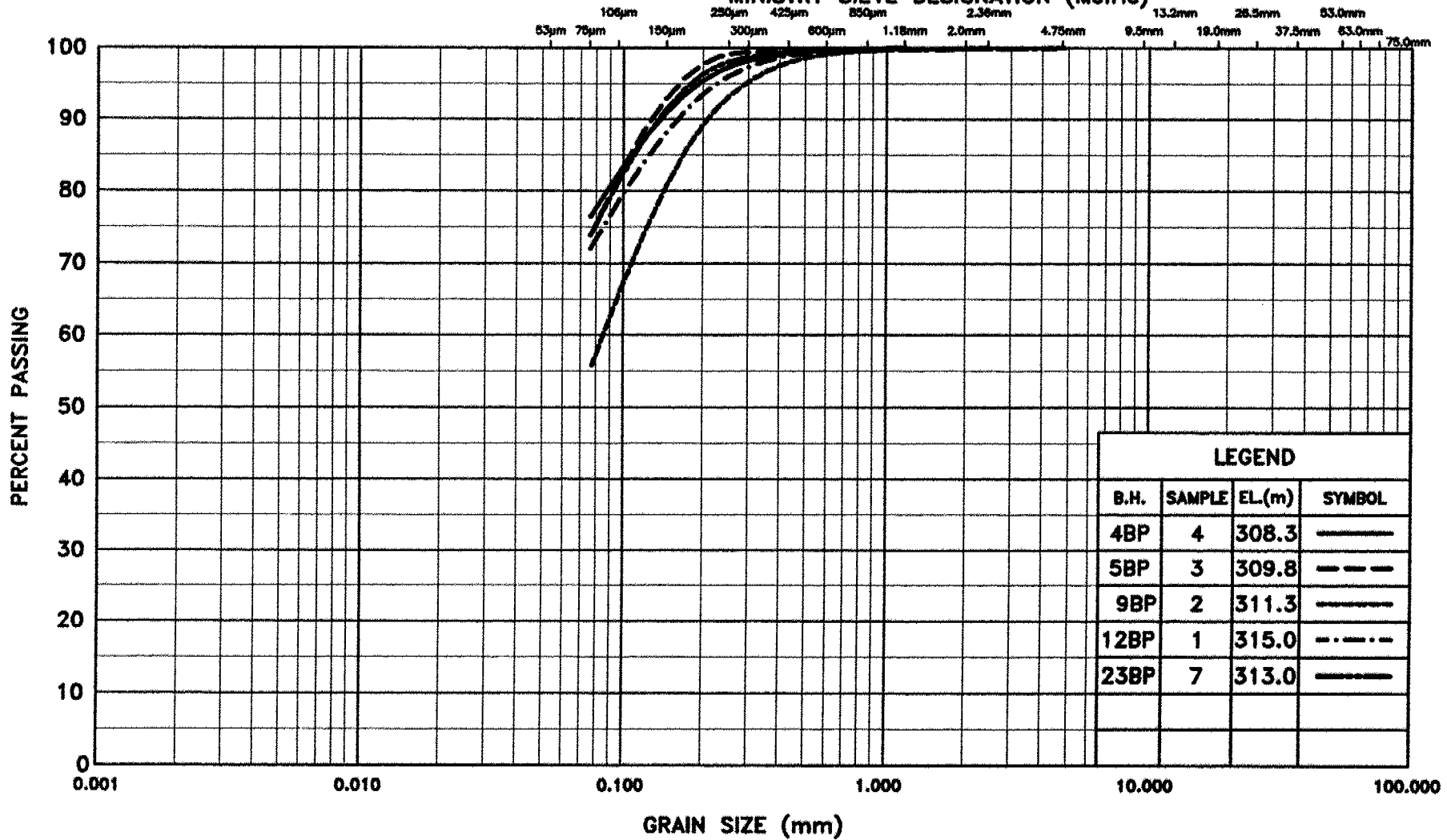


PROJ. No. S07524GB

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION  
SANDY SILT

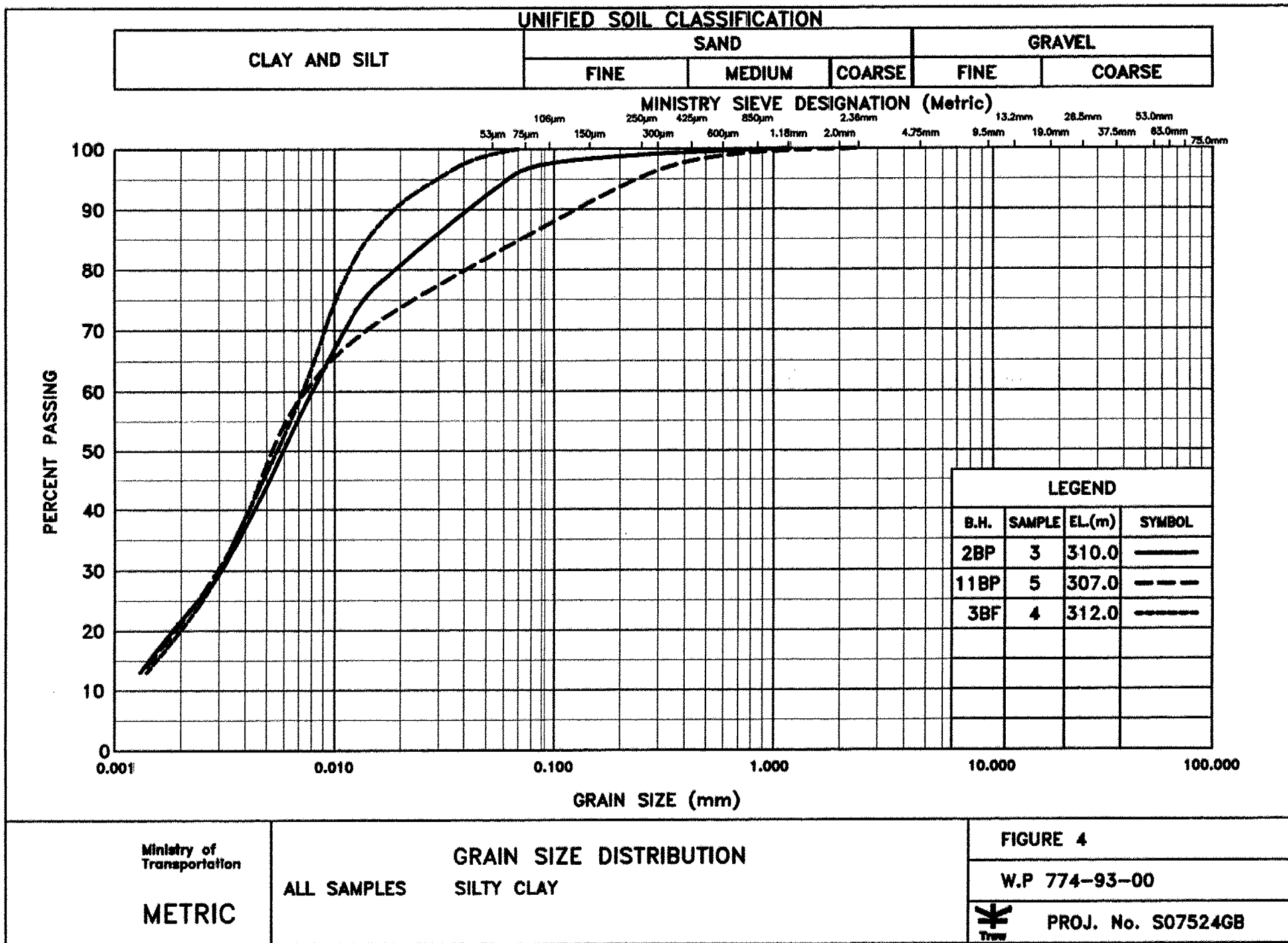
FIGURE 3

W.P 774-93-00



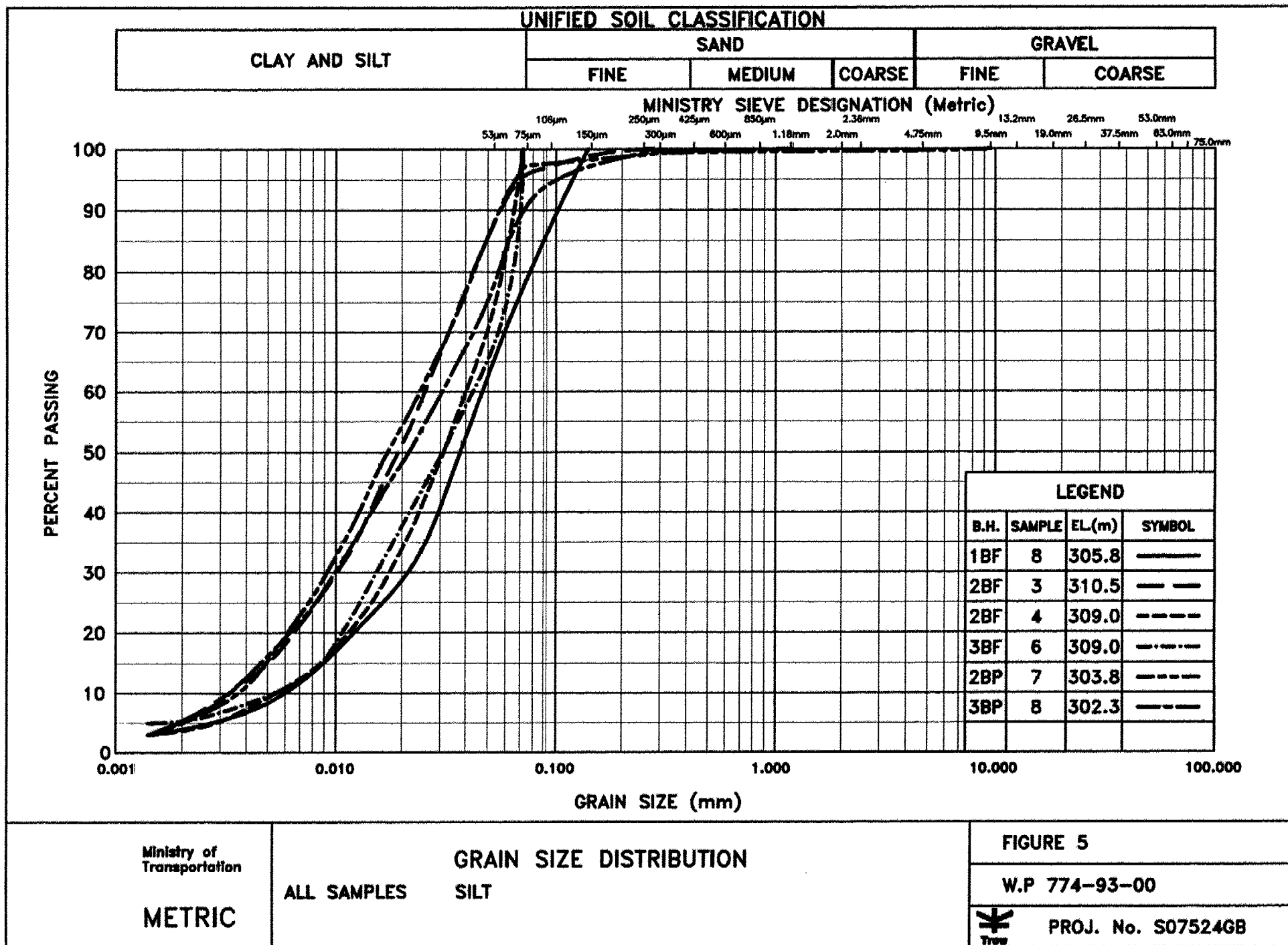
PROJ. No. S07524GB

C:\SU\5246\SU\5246B4-GHAU.dwg Thu Jan 07 16:12:26 1999 IPW SUDOURY





C:\S07524GB\S07524GB5-GBAD.dwg Thu Jan 07 16:09:56 1999 Trow Sudbury

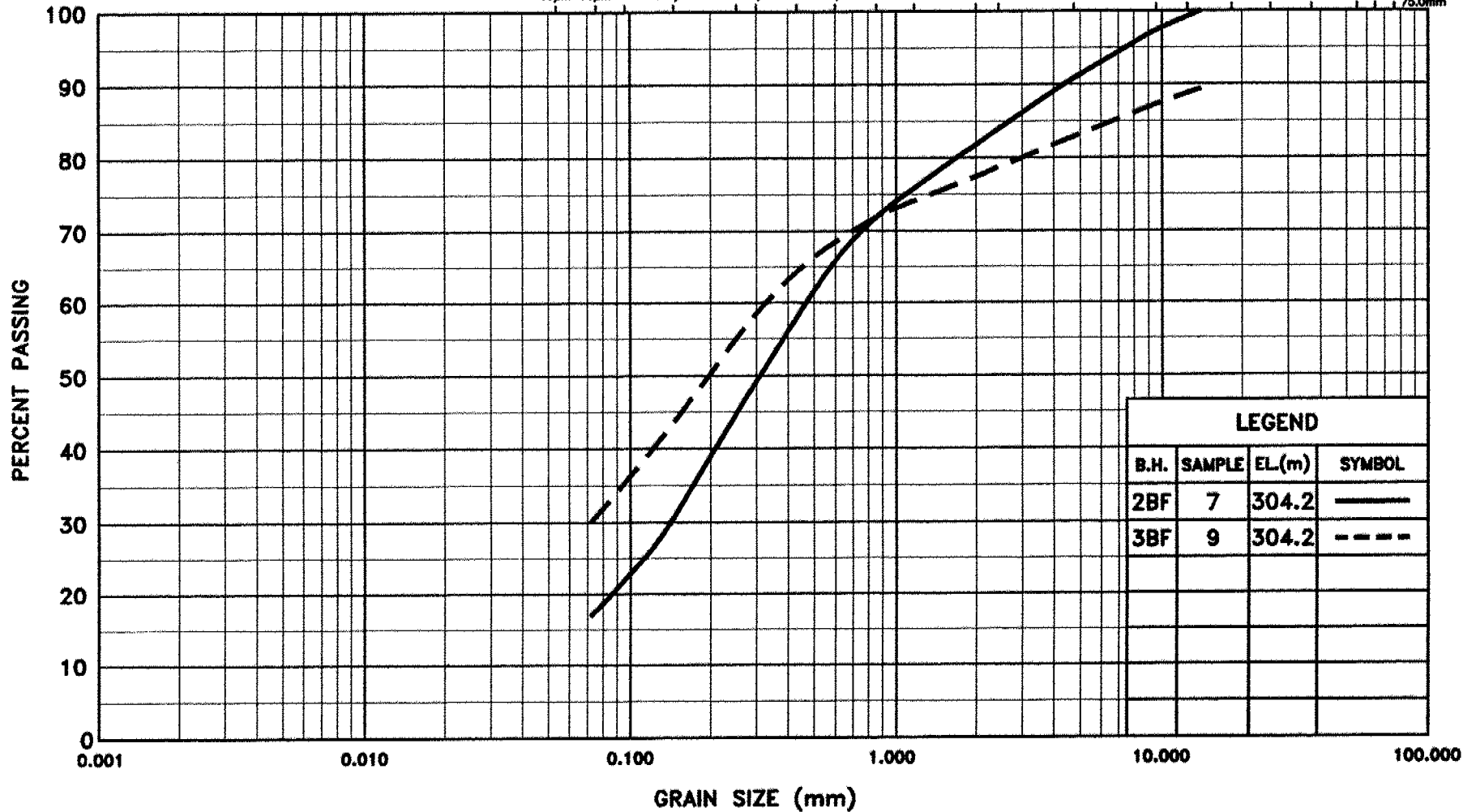


# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)

53µm 75µm 106µm 150µm 250µm 425µm 600µm 850µm 1.18mm 2.0mm 2.36mm 4.75mm 9.5mm 13.2mm 19.0mm 26.5mm 37.5mm 53.0mm 75.0mm



Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION  
SILTY SAND & GRAVEL TILL

FIGURE 6

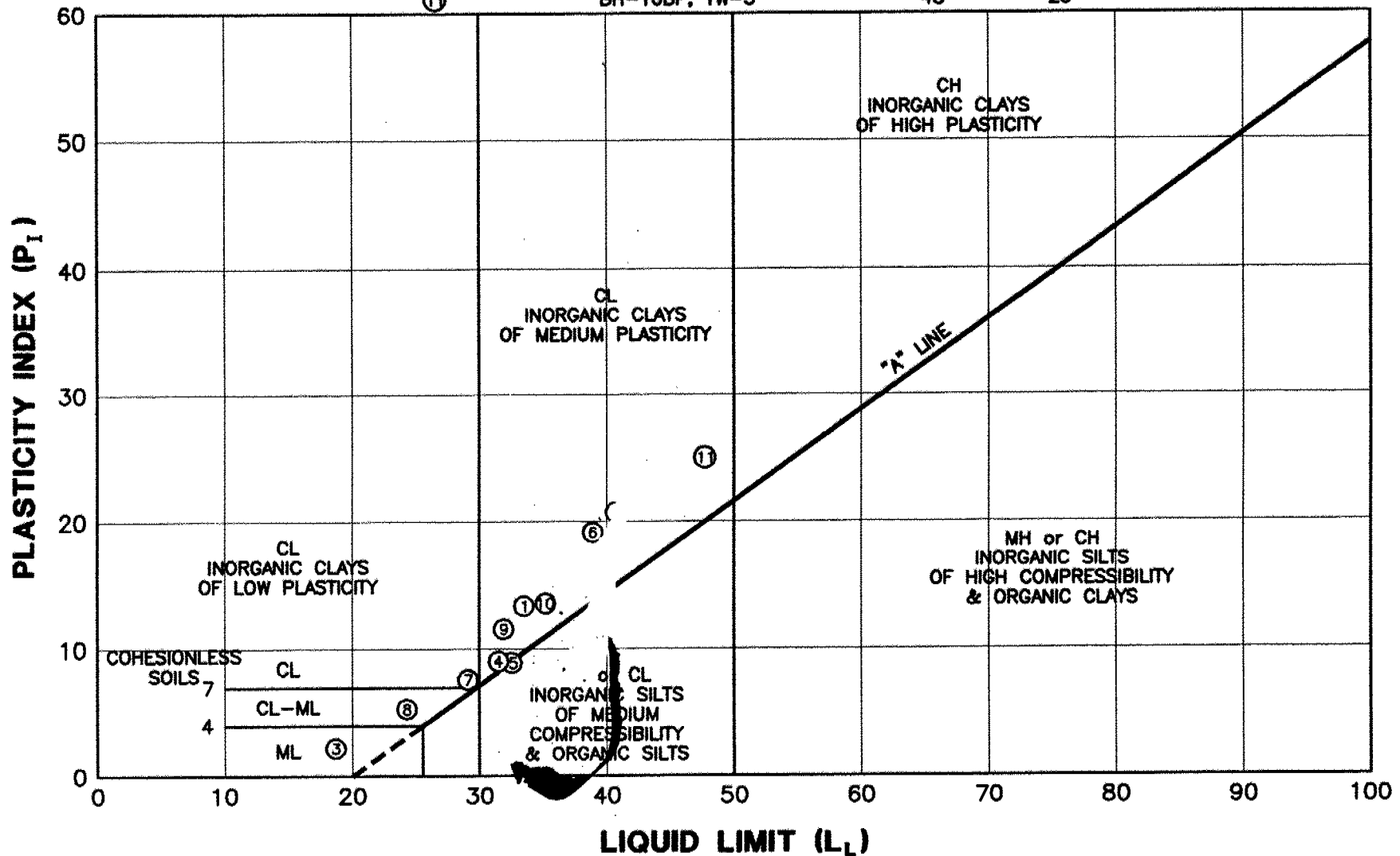
W.P 774-93-00

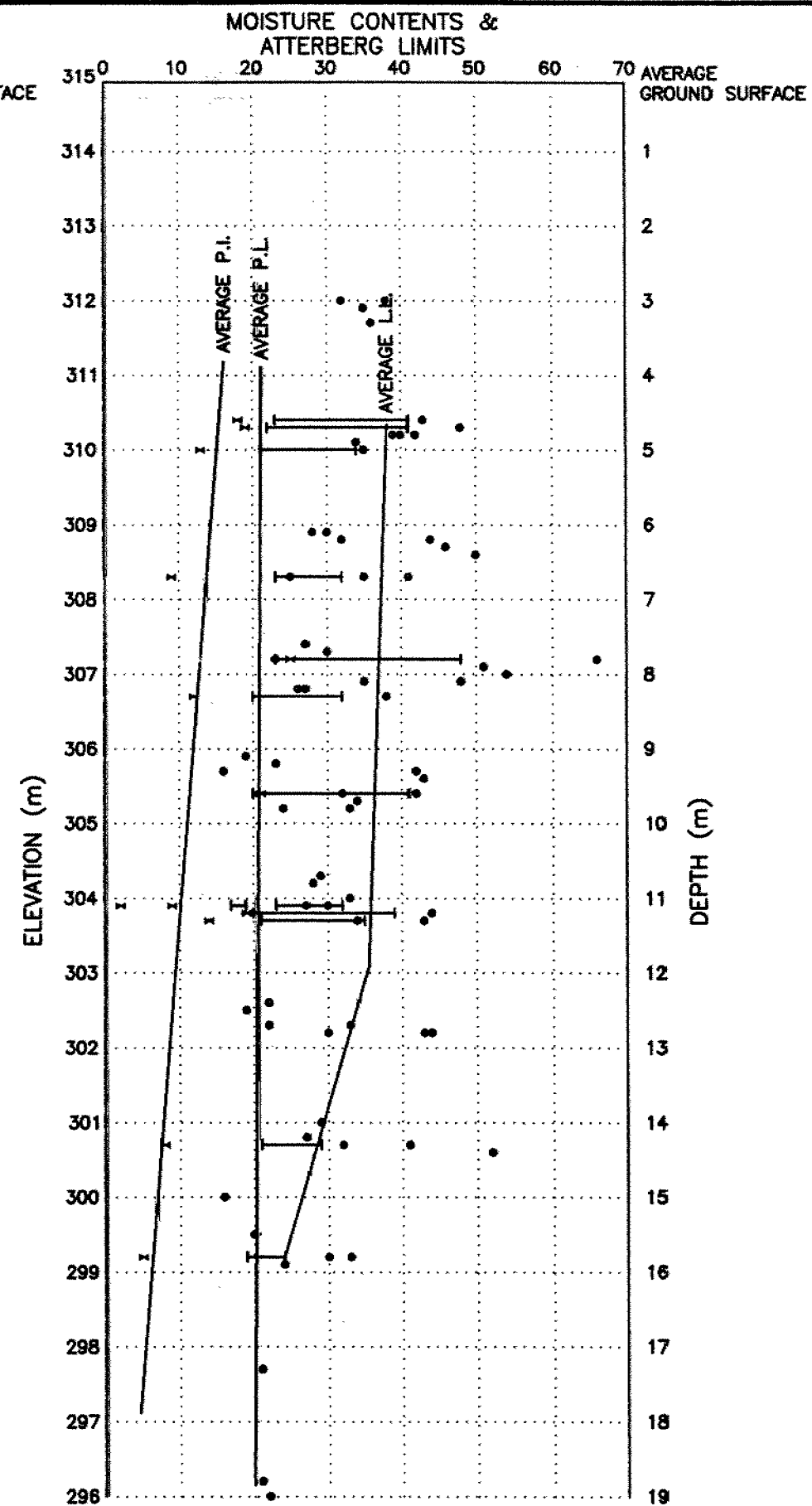
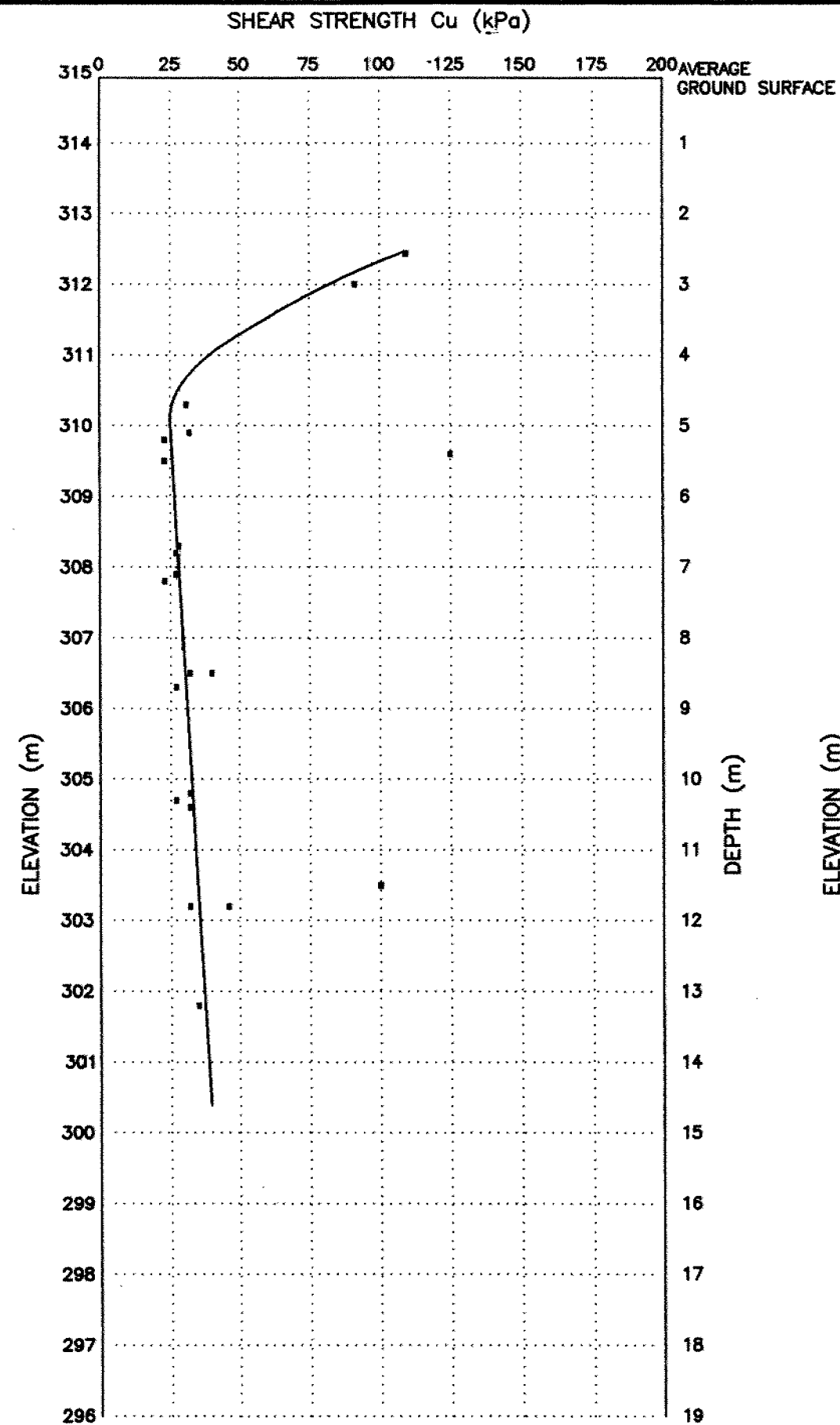


PROJ. No. S07524GB

# **ATTERBERG LIMITS - PLASTICITY CHART**

SYMBOL	DESCRIPTION	LL	PI
①	BH-2BP, SS-3	34	13
②	BH-2BP, SS-6	41	21
③	BH-2BP, SS-7	19	2
④	BH-3BP, TW-4	32	9
⑤	BH-3BP, SS-7	32	9
⑥	BH-4BP, SS-7	39	19
⑦	BH-4BP, SS-9	29	8
⑧	BH-4BP, SS-10	24	5
⑨	BH-9BP, SS-5	32	12
⑩	BH-9BP, SS-7	35	14
⑪	BH-10BP, TW-5	48	25





METRIC

CONT No

WP 774-93-00

SHEET

**LEGEND**

- MOISTURE CONTENT
- ATTERBERG LIMITS (PLASTIC vs. LIQUID)
- x PLASTIC INDEX
- SHEAR STRENGTH

**Trow**

GEOTECHNICAL EVALUATION

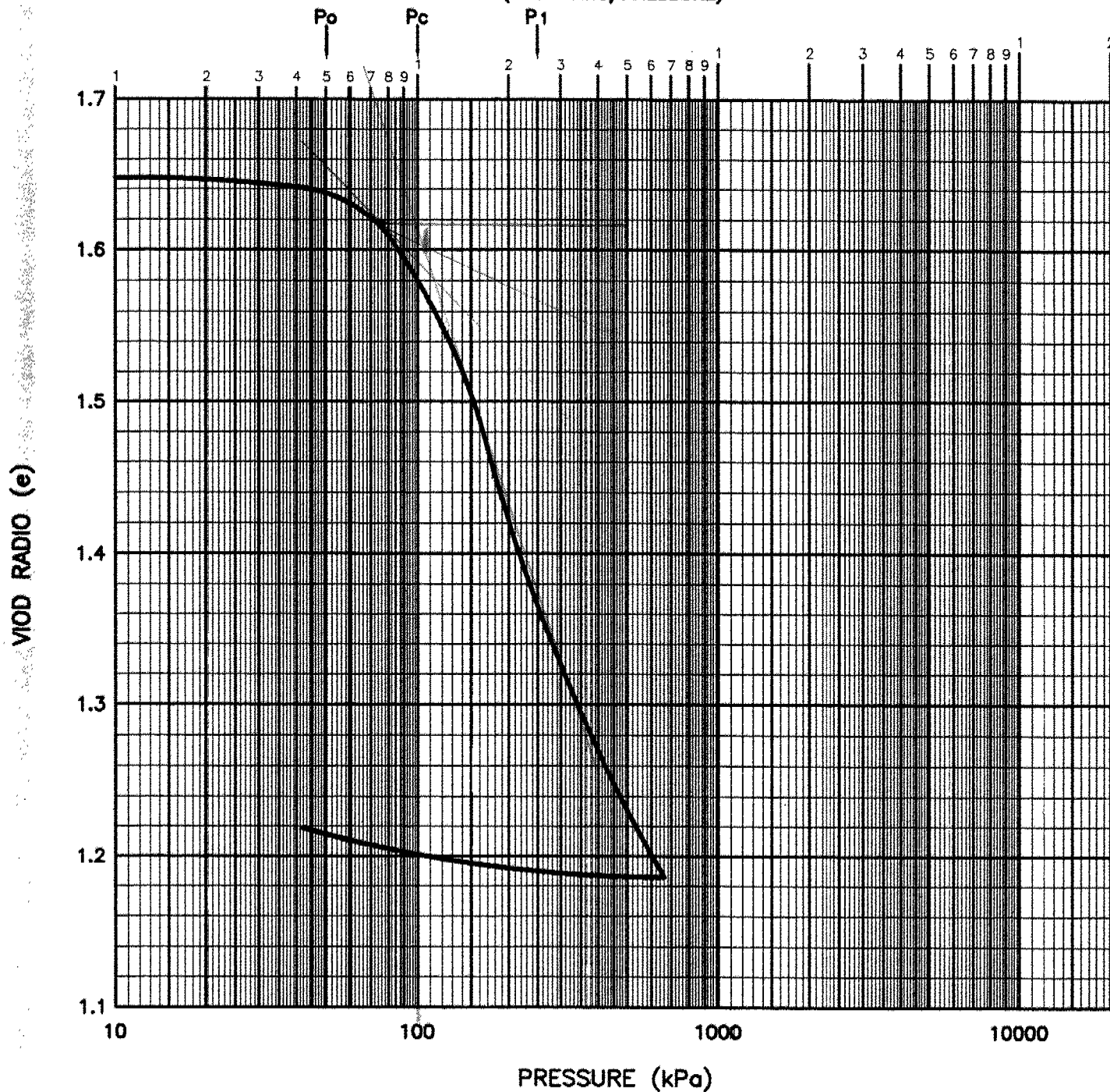
**TROUT CREEK NORTH INTERCHANGE  
SILTY CLAY - SILT STRATA  
SHEAR STRENGTH & ATTERBERG PROFILES**

PROJ. No. S075246/B

DATE: NOVEMBER 1998

FIG. No. 8

CONSOLIDATION TEST RESULTS  
(VOID RATIO/PRESSURE)



BOREHOLE No. 10, TW-5

DEPTH 7.7 m

MOISTURE CONTENT 61 %

LIQUID LIMIT 48 %

PLASTIC LIMIT 23 %

SAMPLE DESCRIPTION

SILTY CLAY, with SILT layers

$m_v = 0.52 \text{ MPa}^{-1}$

$C_v = 4.7 \text{ m}^2/\text{yr}$

$C_c = 0.57$

$C_u = 0.032$

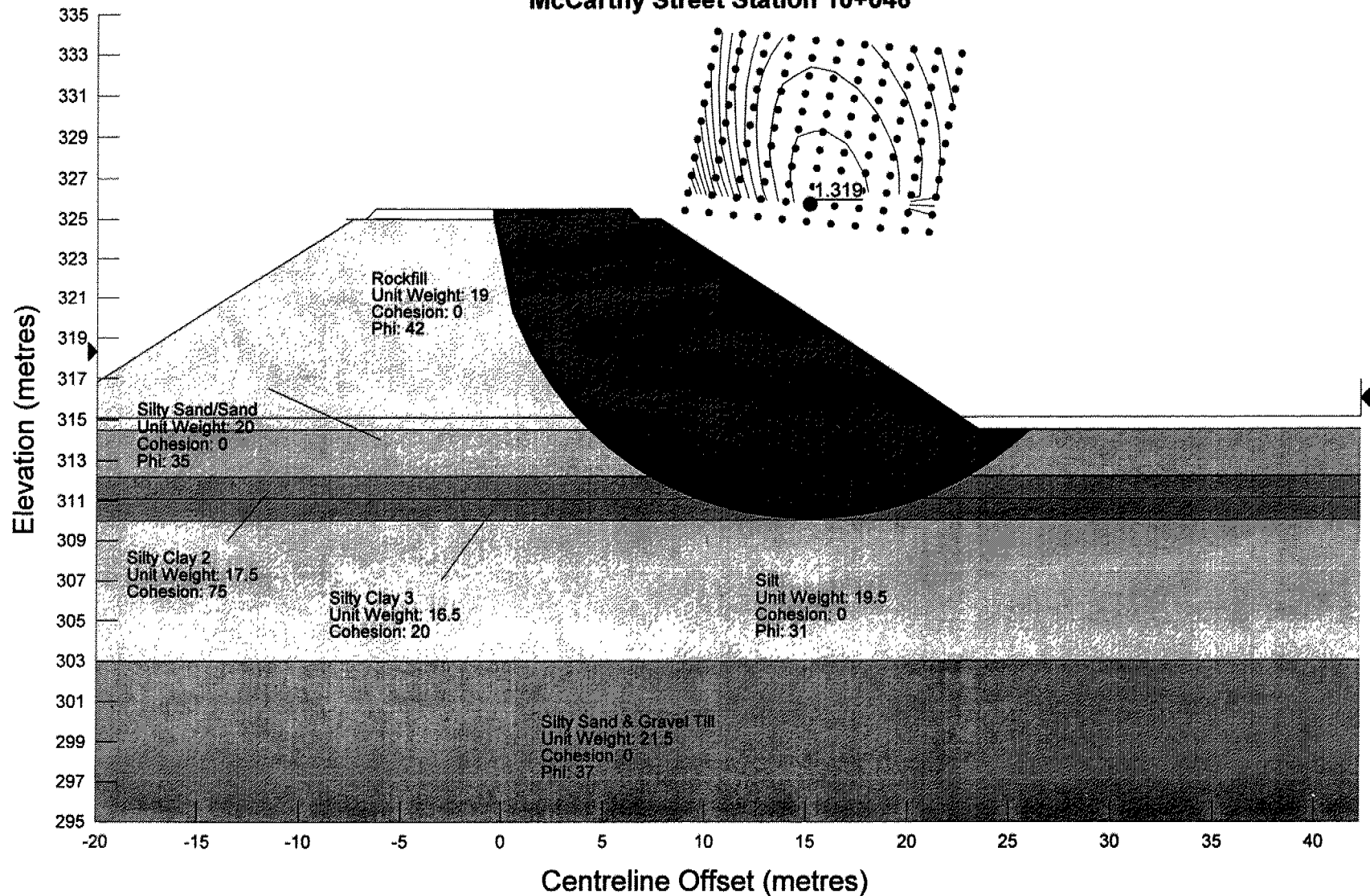
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**APPENDIX D**

**STABILITY ANALYSES**

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**W.P. #774 Highway #11 Trout Creek Bypass**  
**North Interchange Approaches**  
**11 Metre High Embankment**  
**McCarthy Street Station 10+046**

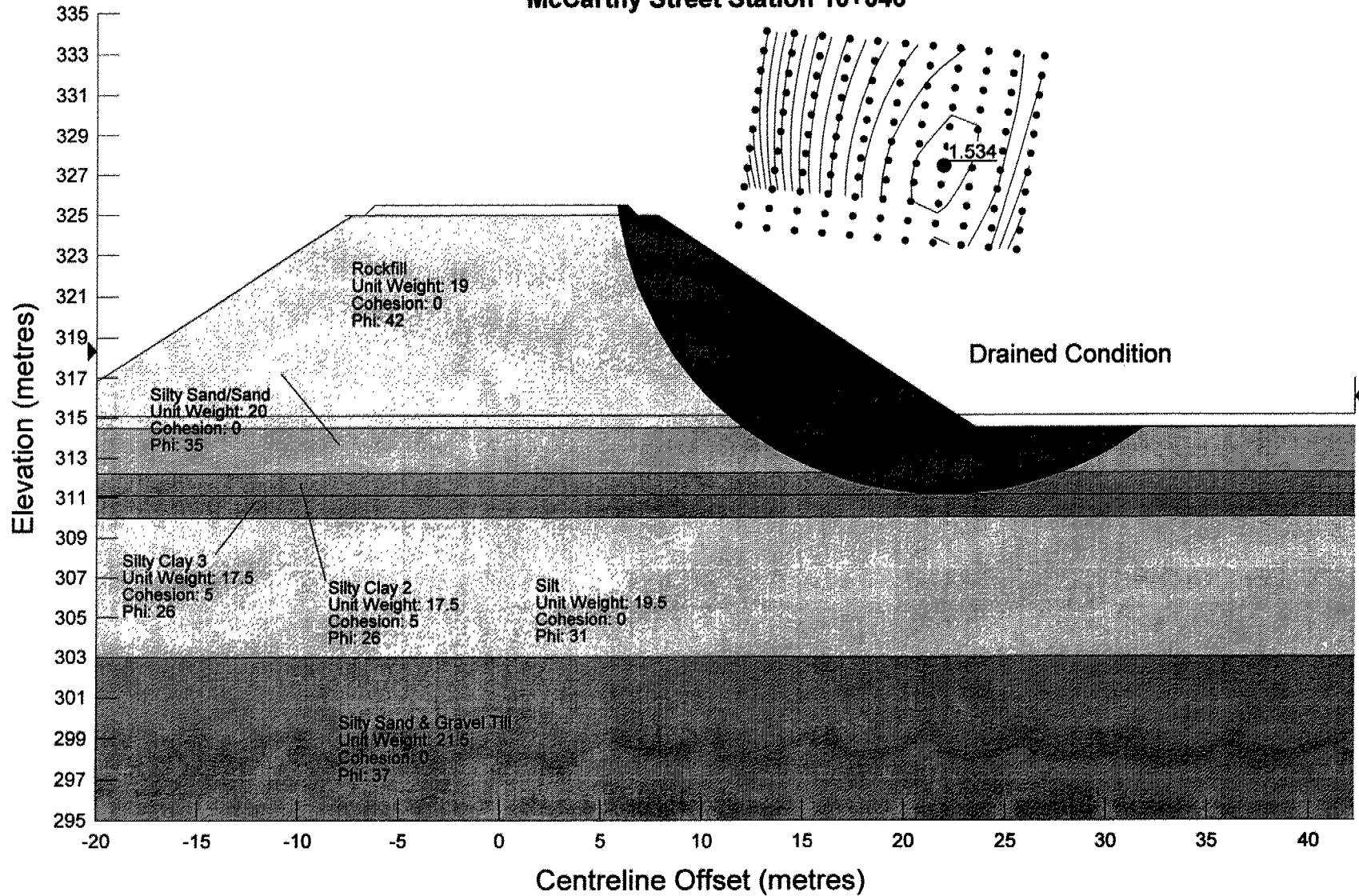


# W.P. #774 Highway #11 Trout Creek Bypass

## North Interchange Approaches

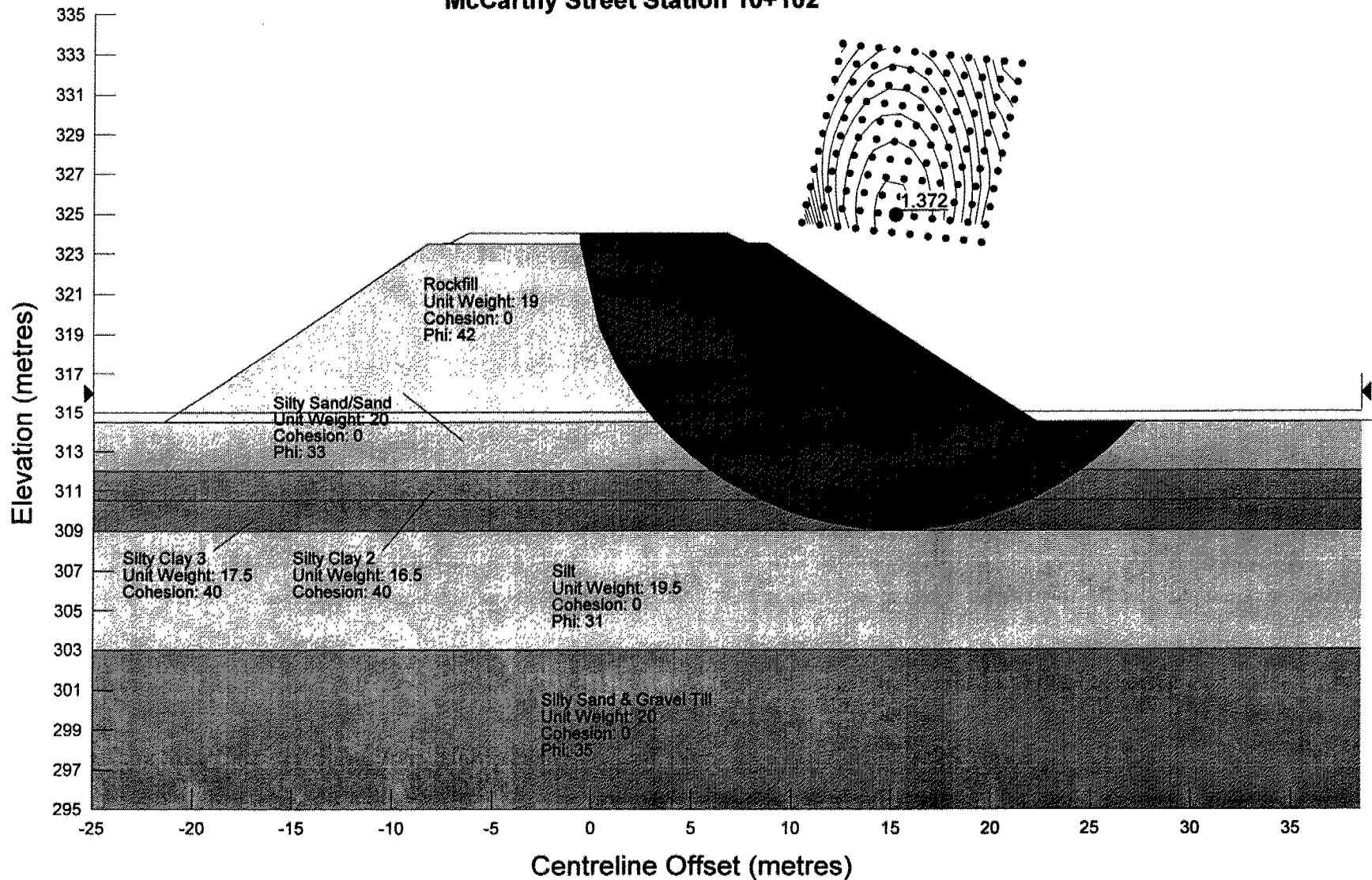
### 11 Metre High Embankment

#### McCarthy Street Station 10+046





**W.P. #774 Highway #11 Trout Creek Bypass**  
**North Interchange Approaches**  
**9.5 Metre High Embankment**  
**McCarthy Street Station 10+102**

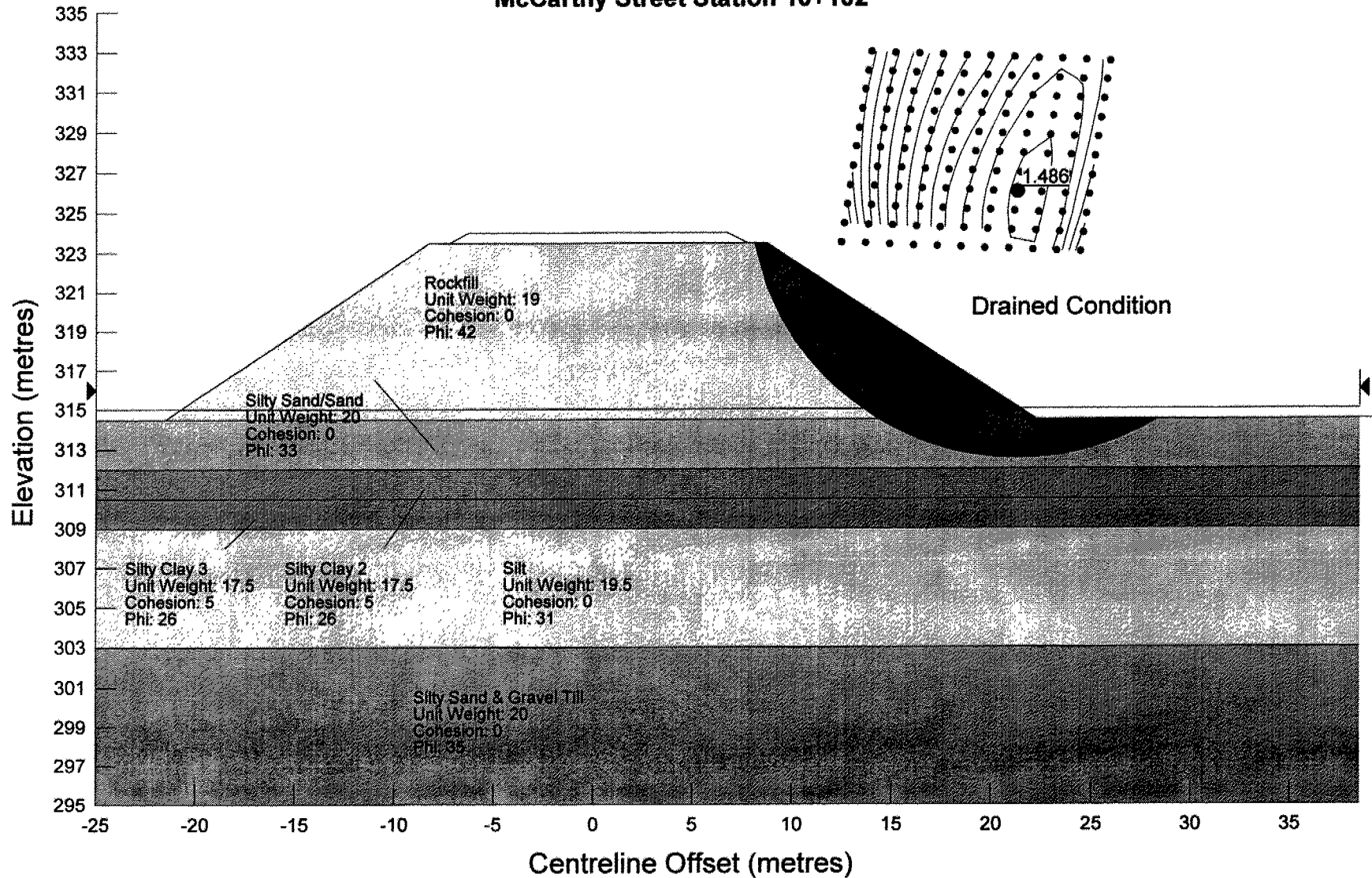


**W.P. #774 Highway #11 Trout Creek Bypass**

**North Interchange Approaches**

**9.5 Metre High Embankment**

**McCarthy Street Station 10+102**

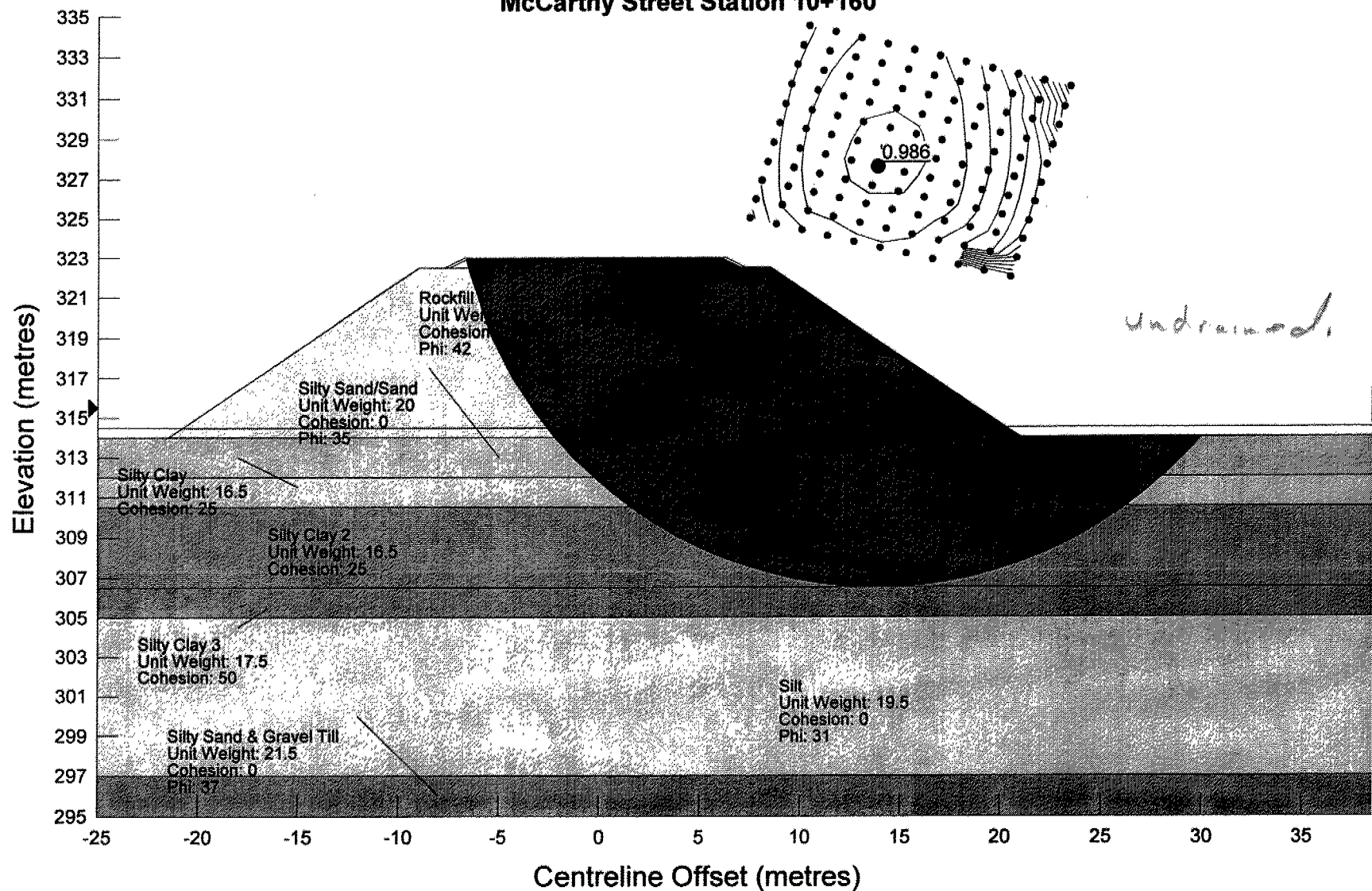


# W.P. #774 Highway #11 Trout Creek Bypass

## North Interchange Approaches

### 9 Metre High Embankment

#### McCarthy Street Station 10+160

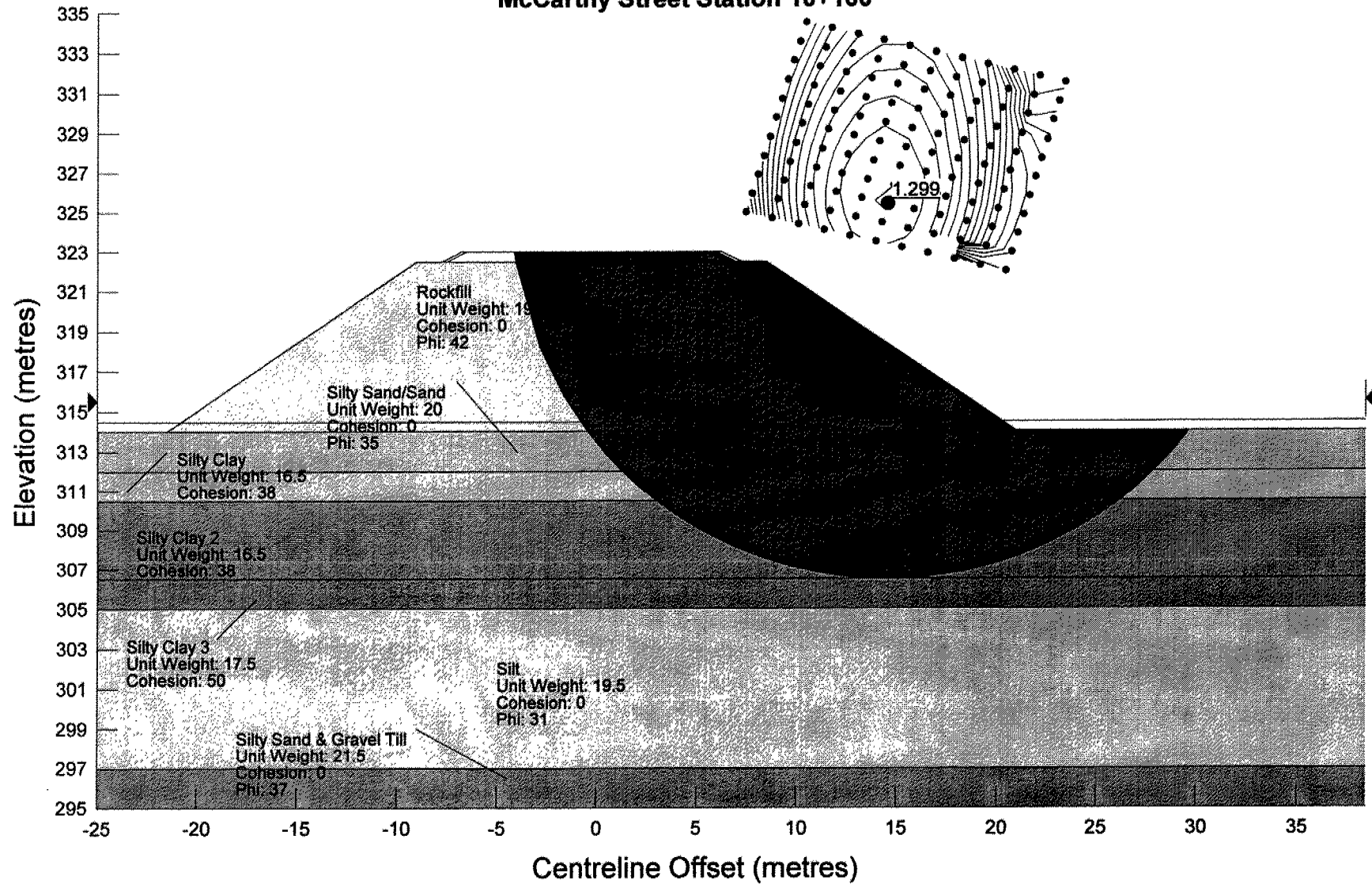


# W.P. #774 Highway #11 Trout Creek Bypass

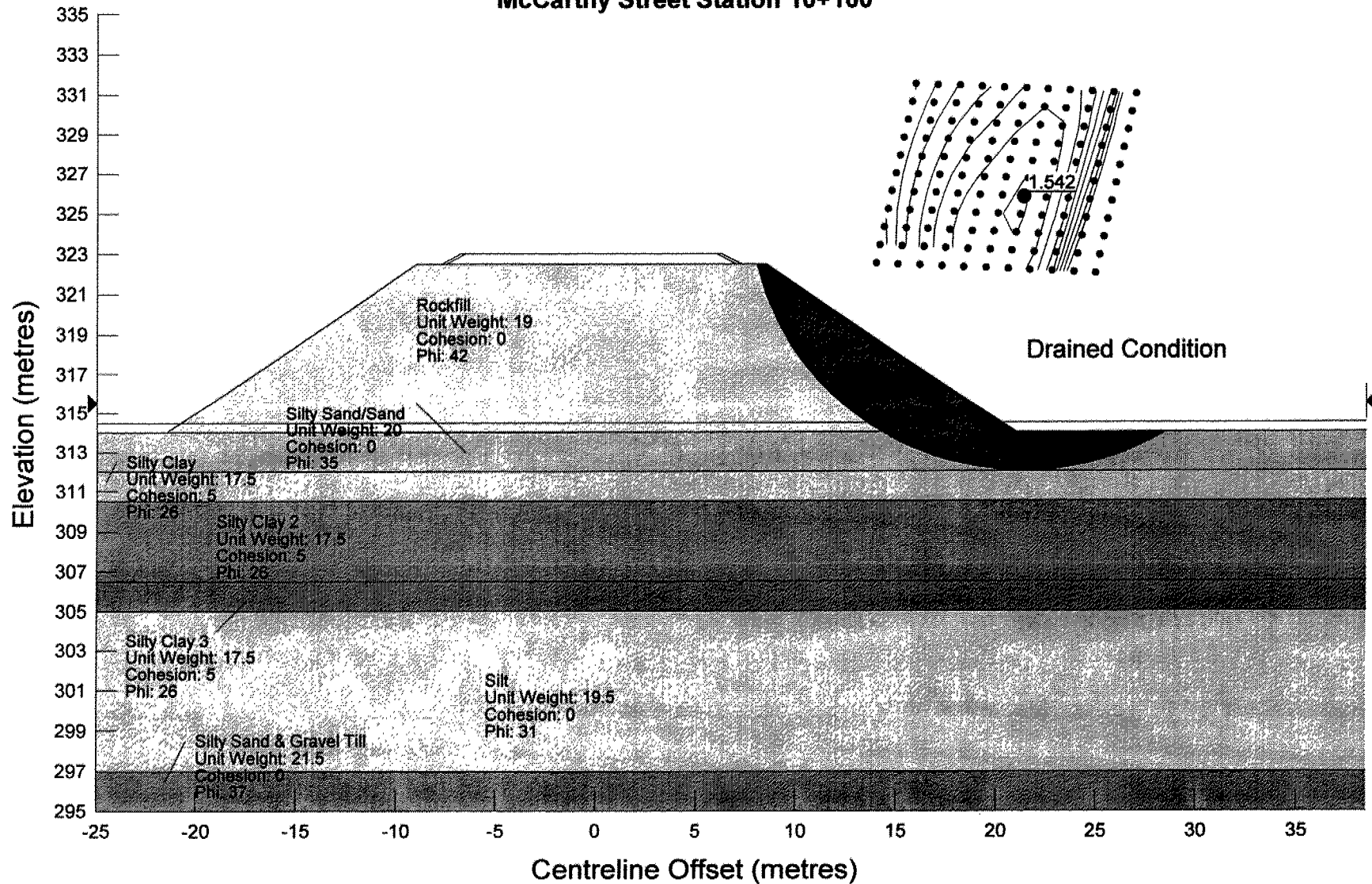
## North Interchange Approaches

### 9 Metre High Embankment

### McCarthy Street Station 10+160



**W.P. #774 Highway #11 Trout Creek Bypass**  
**North Interchange Approaches**  
**9 Metre High Embankment**  
**McCarthy Street Station 10+160**



Bern Design

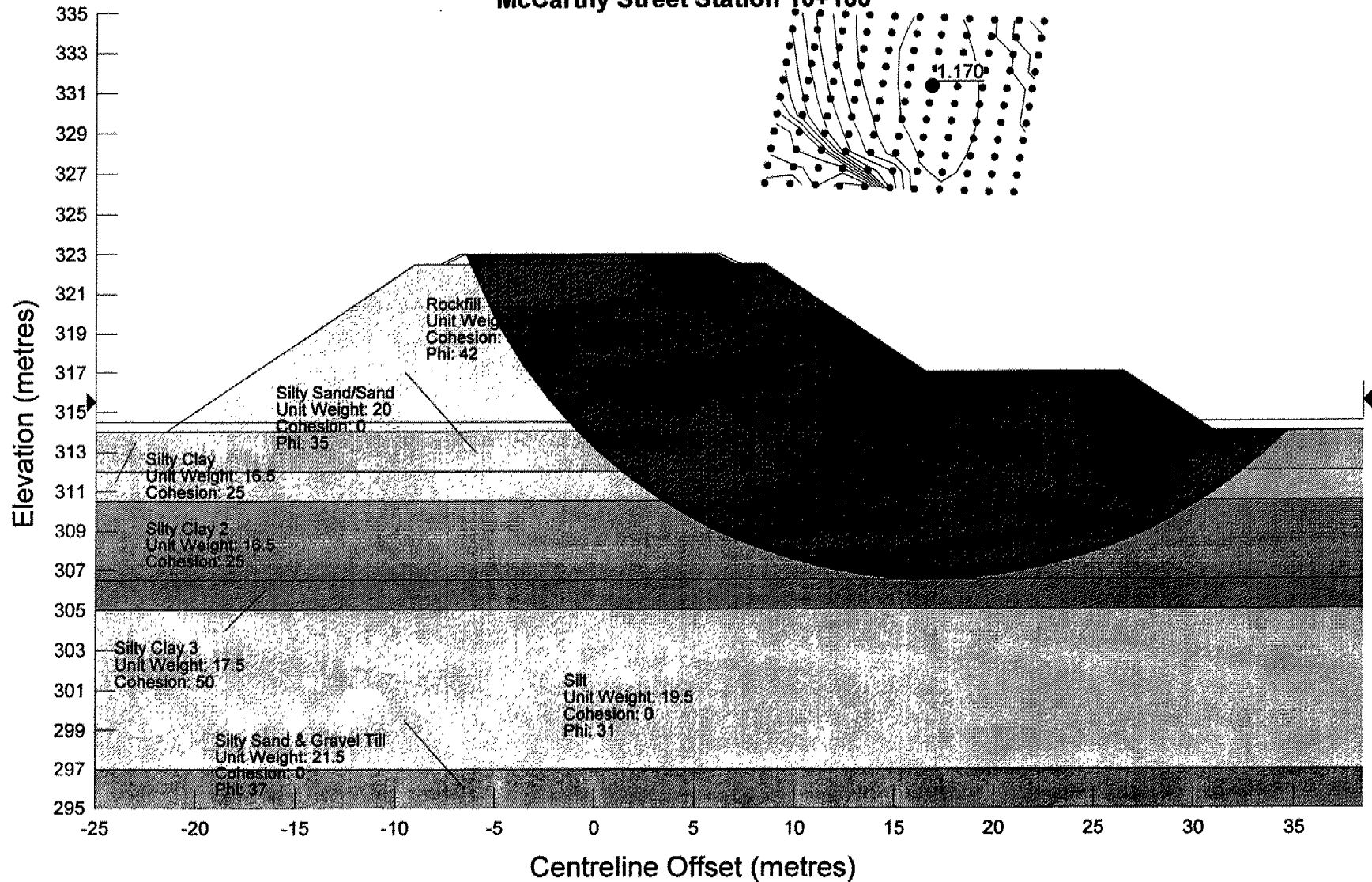


**W.P. #774 Highway #11 Trout Creek Bypass**

**North Interchange Approaches**

**9 Metre High Embankment**

**McCarthy Street Station 10+160**

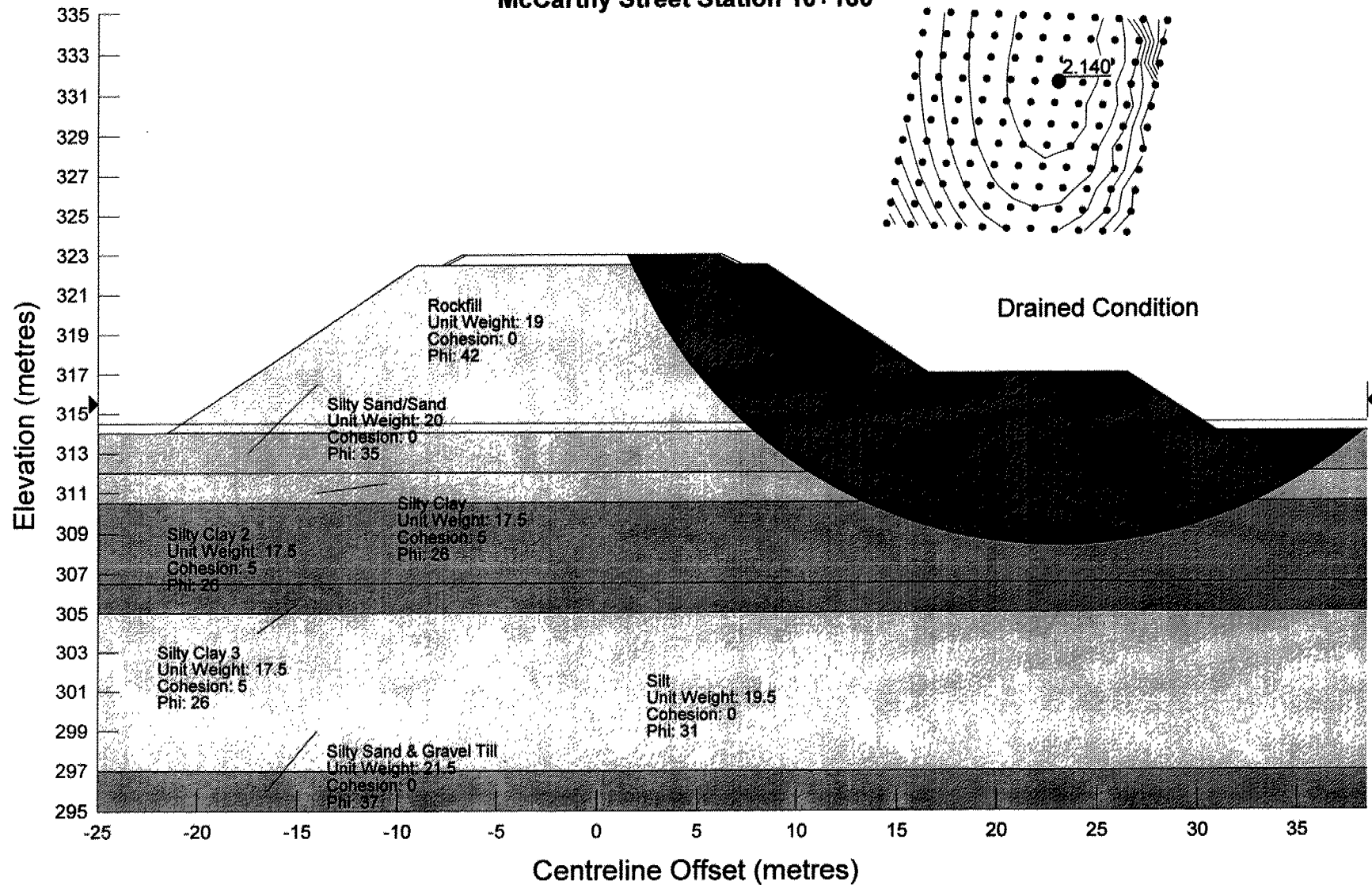


**W.P. #774 Highway #11 Trout Creek Bypass**

**North Interchange Approaches**

**9 Metre High Embankment**

**McCarthy Street Station 10+160**



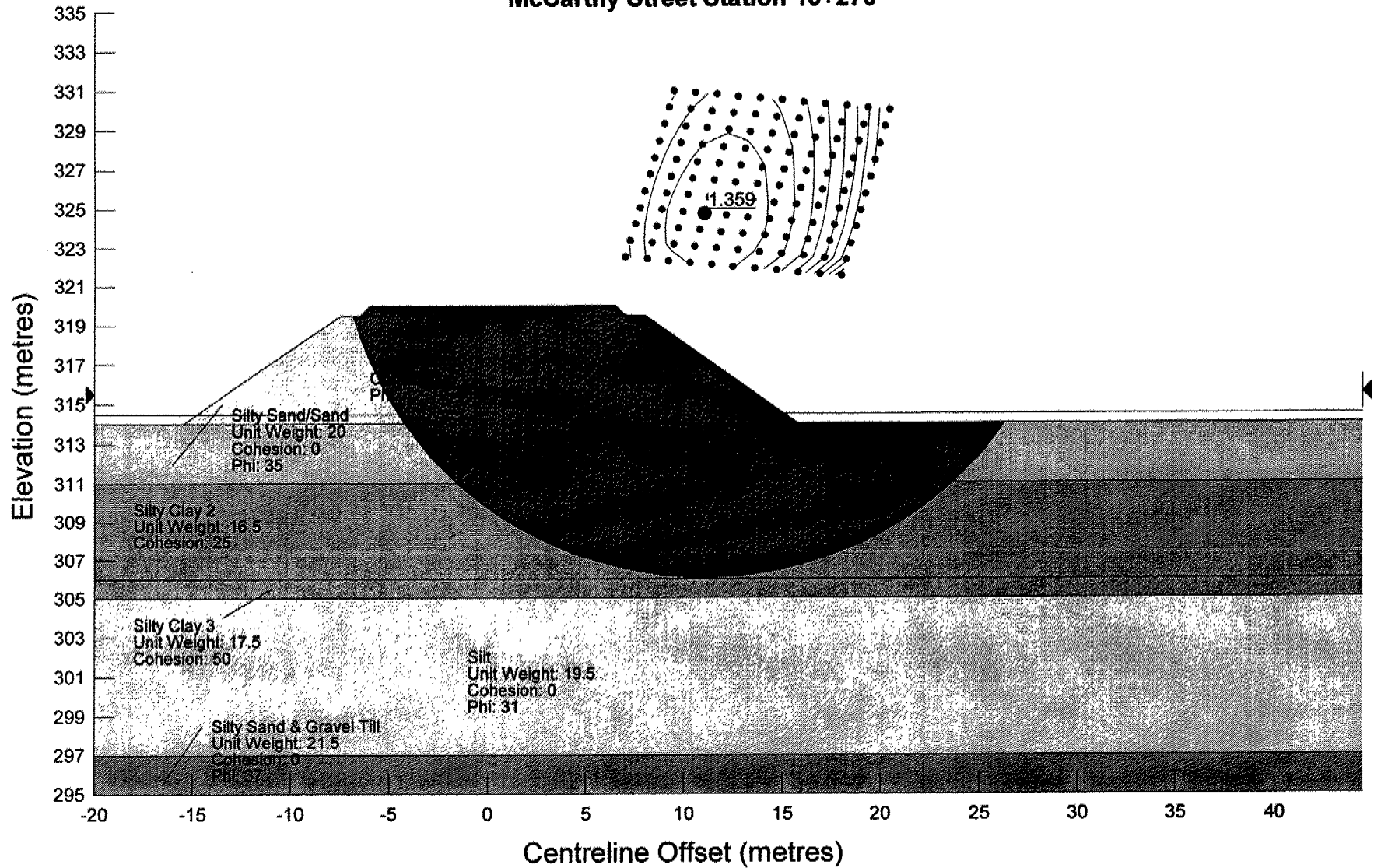


**W.P. #774 Highway #11 Trout Creek Bypass**

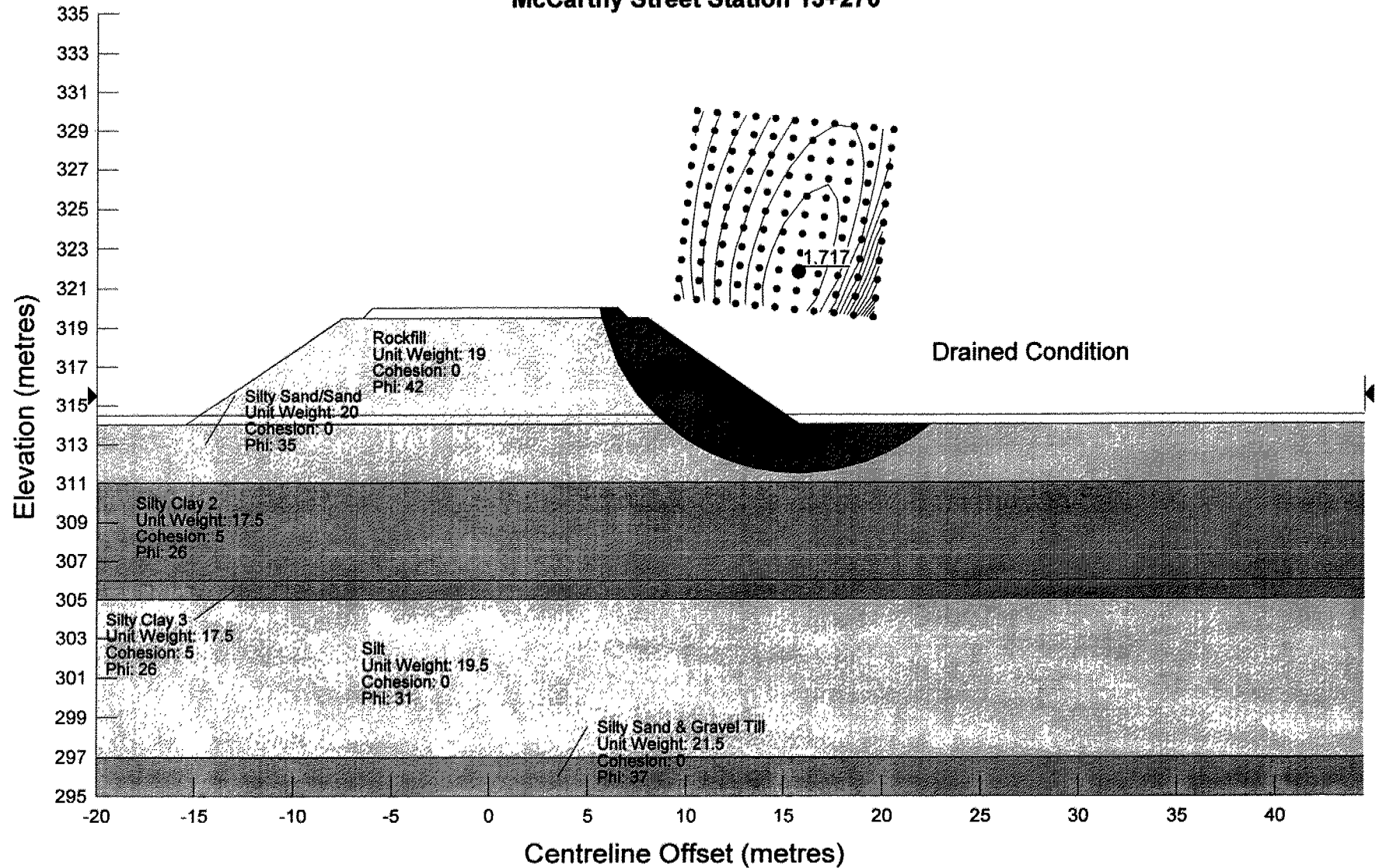
**North Interchange Approaches**

**6 Metre High Embankment**

**McCarthy Street Station 13+270**



**W.P. #774 Highway #11 Trout Creek Bypass**  
**North Interchange Approaches**  
**6 Metre High Embankment**  
**McCarthy Street Station 13+270**



Project Status Report - Trout Ck By-Pass Structures

WP	Status	Comments
North Interchange	Foundation reponse - February 5, 1999	No comments received from Consultant - coming from Trow week of June 7/99
South Interchange	Foundation reponse - January 17, 1999	No comments received from Consultant - response forwarded April 21, 1999
Hwy 522	Foundation reponse - September 10, 1998 -	No comments received from Consultant - response forwarded, 1998. <i>Mar 10</i>
Hwy 11(12+350 to 12+850)	Foundation reponse - March 25, 1999 - not received by Trow	No comments received from Consultant.
Glen Roberts Rd	Foundation reponse - March 25, 1999 - not received by Trow	No comments received from Consultant.
Trout Creek Bridges	Foundation response rec'd May 28/99 - Trow responding	



Sudbury Branch

Trow Consulting Engineers Ltd.  
1074 Webbwood Drive  
Sudbury, Ontario P3C 3B7  
Telephone: (705) 674-9681  
Facsimile: (705) 674-8271

S07524G/C

March 10, 1999

Marshall Macklin Monaghan  
80 Commerce Valley Drive East  
**THORNHILL, ON**  
L3T 7N4

**ATTENTION: Mr. Robert Kivi, P.Eng.**

Dear Sirs:

**RESPONSE TO MTO'S COMMENTS  
HIGHWAY 522 OVERPASS FOUNDATION REPORT  
TROUT CREEK BY-PASS  
HIGHWAY 11  
GWP 774-93-00**

Trow's response to the MTO's comments on the Foundation Investigation and Reports issued for the above noted project, is presented in the following letter. Please note that the structure of our letter is to address the items as listed within the memorandums (foundation and structural) received from the MTO.

**1.0 Comments from the MTO Foundation Section**

Comments were issued by the MTO foundation section on September 10, 1998 for the Foundation Report for the Highway 522 Structure, and the Foundation Report for the Highway 522 Approach Embankments. Comments on the factual component of both the reports were addressed together, with the following points:

1. The lab shear tests conducted on selected soil samples were pocket penetrometer tests. This will be identified within the final version of the report. ✓
2. The following sentences will be added to the report to address borehole backfilling procedures.

**RESPONSE TO MTO'S COMMENTS  
HIGHWAY 522 OVERPASS FOUNDATION REPORT****SO7524G/C**

- 1.0 All of the previous reports Trow has issued, while working as a subconsultant on MTO projects, have been prepared for our client, the Prime Consultant. We believe it is proper to issue the report addressed to our client, the Prime Consultant.
- 2.0 Overpass will be changed to the proper terminology of underpass on Page 8 of the report. Revisions will be made to the report to include the dept to bedrock from the existing ground level and the proposed grade at the abatement levels.
- 3.0 The reference 'subject to inspection by a qualified geotechnical engineer' is a standard phrase included in most geotechnical reports, since the reports are based upon a limited sampling programme, and it is important to ensure the assumptions based upon the samples are verified during construction. This work will not be performed by the TPM consultant under the current agreement. Comments regarding bearing capacity for the bedrock are given in section 1, item 2.2.
- 4.0 & 5.0 The reference 'subject o inspection by a qualified geotechnical engineer' will be added to the sections pertaining to the centre pier and west abatement.
- 6.0 Should the option to utilize lean concrete instead of granular material for footings placed on a pad be selected, more details will be given to the Prime Consultant as required.
- 7.0 Should dowels be utilized for this project, more information will be given as to the minimum requirements for dowels. As indicated in Section 1.0, Table 2-8 has been modified.
- 8.0 The typo referring to OPSA has been corrected to OPSD. Given the short length of piles feasible at this site, very limited lateral resistance will be developed with vertical piles, thus the recommendation to utilize inclined piles to support lateral loads.
- 9.0 The slopes given in Section 2.5 of the report are the maximum safe gradient of the various material types, and restrictions on the slope gradients are not given. Thus the usual forward and side slope of 1.5:1 could be utilized on this site, with appropriate consideration given the material type.

Mechanically stabilized earth retaining walls are not given as a requirement, but as a suggestion. This suggestion will be removed from the final version of the report.

- 10.0 See item 9.0, Section 1.0

# MEMORANDUM



To: Mike McCormick  
Structural Engineer  
Structural Section  
Northern Region

Date: September 3, 1997

From: Pavements and Foundations Section  
Room 223, Central Building  
1201 Wilson Avenue  
Downsview, Ontario

Tel: (416) 235-3506  
Fax: (416) 235-5240

Re: TPM Consultant Assignment  
Foundation Engineering Consultant  
WP 774-93-00, Trout Creek Bridges, Site 44-369/370/371/372, HWY. 11

(A)

The Foundation Engineering component of this project is considered to be of **foundation medium complexity**. The Foundation Engineering consulting firms indicated on the attached list are eligible for this project. In order to ensure consistency and accountability, the designated principal contact must be the liaison for this project.

**Tae C. Kim , P. Eng.**  
Sr. Foundation Engineer

attach

**Version Aug 25/97  
Eligibility List  
Foundation Engineering Consultants  
Medium Geotechnical Complexity Assignments**

**Consultant**

**Designated Principal**

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Jacques, Whitford Limited  
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McClymont and Rak Engineers, Inc.  
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Tel: (416) 675-0160  
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**Designated Principal**

Ivan Lieszkowszky

Fin Heffernan/Dennis Becker

Charles Kwok

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Shaheen Ahmed/Ken Peaker



**Consultant**

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Etobicoke, Ontario  
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Fax: (416) 503-3010

**Designated Principal**

Mike Tanos/ Paul Bowen

P.K. Chatterji

**From:** Ken Ahmad  
**To:** Kim  
**Date:** 9/3/97 10:54am  
**Subject:** WP 774-93-00 Hwy 11 Dist 54 -Forwarded -Reply

September 3, 1997

Tae:

As requested, I have reviewed the Geocres 31E37, 31E109, 31L57 and 31L58 to find out the soil condition near the proposed structures. The soil condition in the area varies from mainly very loose to compact sandy silt to silt with layers of soft to firm silty clay. The hard bedrock in the region is expected to be at approximate depths ranging from 10m to 36m. Most of the structures in the area are founded on deep foundations.

The proposed three underpasses are two span concrete structures approximately 80-90m long. The structure on Trout Creek will be three span 110m long steel girders. Based on the guidelines of project complexity, the proposed structures appear to be of *medium* complexity.

Ken

>>>> Tae C. Kim 09/03/97 09:16am >>>>

Dear Ken !

Would you review this projects and advise me the complexity of these projects ASAP.

Thanks

Tae

**From:** Tae C. Kim  
**To:** Ahmad  
**Date:** 9/3/97 9:16am  
**Subject:** WP 774-93-00 Hwy 11 Dist 54 -Forwarded

Dear Ken !

Would you review this projects and advise me the complexity of these projects ASAP.

Thanks

Tae

**From:** Mike McCormick  
**To:** MTOHO1.TORHO2.Kim  
**Date:** 9/2/97 11:24am  
**Subject:** WP 774-93-00 Hwy 11 Dist 54

The above noted project is a new four lane alignment of highway 11 from 4 km south of Hwy 522 northerly 7.9 km and is a bypass of the town of Trout Creek west of existing Highway 11. The following new structures will be part of the project:

Trout Ck South Interchange U'pass ( Laurier Twp Con 14/Himsworth South Twp Con 1), Site 44-372

Trout Creek NBL & SBL Water Crossings, Site 44-371

Hwy 522 U'pass, Site 44-370

Trout Ck North Interchange U'pass (McCarthy St), Site 44-369

The three underpasses will probably be two span concrete structures approximately 80-90 m long with integral abutments where possible. The twin structures at Trout Creek will probably be three span 110m long steel girder structures with integral abutments, high piers (20 m) and extensive retaining walls.

We are in the process of evaluating the EOI submissions for the TPM RFP for detailed design.

\*Please review the project and provide a list of acceptable foundation firms capable of performing foundation investigation work for the project . Thank you.

**Foundation Investigation and Design Report  
Bridge Structure, Approaches and Embankment Fills  
South Interchange (Boundary Road) - Site 44-372  
Trout Creek By-Pass, King's Highway 11  
District 54, Sudbury, Ontario  
GWP No. 774-93-00**

Prepared For:

Marshall Macklin Monaghan  
80 Commerce Valley Drive East  
Thornhill, Ontario  
L3T 7N4

**TROW CONSULTING ENGINEERS LTD.**

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## Preface

Work Project GWP 774-93-00 is one of a series of projects for the four lane expansion of Highway 11. This project involves the four lane design of Highway 11, from 4.0 km south of Highway 522, northerly for 7.9 km. It will result in the construction of a by-pass of the existing Highway 11 to the west of the Town of Trout Creek.

This highway section is located in the Townships of Laurier and South Himsworth, within the geographic District of Parry Sound, as shown on the Key Plan, Figure A1 in Appendix A. The project requires geotechnical input for the following major components:

- New pavement design for the entire length of the four lane by-pass, including associated service roads.
- New structure, Trout Creek South Interchange (underpass), Site 44-372.
- New structure, Trout Creek Northbound Lanes, Site 44-371.
- New structure, Trout Creek Southbound Lanes, Site 44-371.
- New structure, Highway 522 (underpass), Site 44-370.
- New structure, Trout Creek North Interchange (underpass), Site 44-369.

The following report deals with the new bridge structure at the proposed Trout Creek South Interchange (Boundary Road), Site 44-372, as well as the embankments and approach fills. Separate reports will be submitted for the additional components.

Part 1 of this report, Foundation Investigation, describes the geotechnical investigation methodology and the results derived, as pertaining to the design parameters required for the bridge foundations and related earthworks.

Part 2 of this report, Engineering Discussion and Recommendations, addresses the geotechnical engineering design considerations pertaining to the bridge foundations and the approaches and embankment fills.

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## **Appendix B**

Borehole Logs and Rock Descriptions

## **Appendix C**

Results of Laboratory Testing (grain size, Atterberg Limits, consolidation tests, triaxial tests)

## **Appendix D**

Stability Analysis Printouts

## **Part 1 Foundation Investigation**

### **1.1 Introduction**

This report presents the results of a geotechnical foundation investigation by Trow Consulting Engineers Ltd. (Trow) for the proposed bridge structure, approaches and embankment fills at the South Interchange to the Town of Trout Creek (the proposed King's Highway 11 Trout Creek By-Pass and Boundary Road) at Site 44-372.

It is Trow's understanding that a two span structure will be constructed to carry Boundary Road (presently McFadden Line) traffic over the realigned four lane Highway 11, with the central pier located in the median of the proposed King's Highway 11. An approximate 4 m grade increase of the new four lanes is proposed at the bridge location, requiring up to approximately 12 m grade raises for the realigned section of Boundary Road at the bridge abutments, in accordance with the grading plan proposed to meet the design objectives.

### **1.2 Site Description and Geological Setting**

The site is located in the Townships of South Himsforth and Laurier, in the vicinity of the intersection of McFadden Line and Highway 11, about 2 km south of the Town of Trout Creek, as shown on Figures A1 and A2, in Appendix A.

The terrain at the site is generally flat and poorly drained (swampy), as evidenced by surficial ponding water. The relief generally varies within only a few metres.

There are mature trees, with heavy underbrush, across the site. The easement for TransCanada PipeLines runs in a north-south direction, located approximately 400 m west of the proposed bridge location.

According to OGS Maps 2544 and 2556, as well as Ontario Geological Survey Map P. 3160 (Quaternary geology, South River area), the site is located in what is known as the Central Gneiss Belt, i.e. mainly felsic igneous rocks of the Mesoproterozoic Group.

The overburden soils have been mapped as comprising organic deposits (peat) over much of the site of the South Interchange with underlying glacio-lacustrine deposits. These deposits consist of laminated, rhythmically bedded (also referred to as varved) to massive silts and clays. The western portion of the site, in the vicinity of the proposed N-EW and EW-S ramps, is mapped as a bedrock drift complex with relatively thin, primarily cohesionless soil cover (till).

## 1.3 Investigative Procedures

### 1.3.1 General

Part 1 of this report describes the investigative procedures adopted for the geotechnical assessment of the south interchange (Boundary Road) at Trout Creek By-Pass, King's Highway 11. Properties of the overburden soils were obtained by *in situ* and laboratory testing and the procedures employed during the investigation are described below.

### 1.3.2 Field Investigation

The initial field work for the investigation related to the proposed bridge structure and highway embankments was carried out between May 28 and August 13, 1998. The locations of boreholes, cones and probes are shown on Figures A2a, A2b, A3 and A4, in Appendix A. These locations, as well as the surface elevations, were established by a survey crew from Marshall Macklin Monaghan, and are referenced to geodetic datum.

Investigations in connection with the bridge foundations consisted of three boreholes (BH-1FF, 2FF, 3FF), three dynamic cone penetration tests (C-1FF, 2FF, 3FF) and three additional auger probes (AP-12FF, 13FF, 14FF). The dynamic cones and boreholes were advanced to refusal at depths ranging from 14.6 m to 22.1 m.

Investigation in connection with the embankments included 21 fully sampled boreholes (BH-1FP to BH-11FP, and BH-15FP to BH-24FP) and 5 dynamic cones (C-1FP to C-5FP). Following a September, 1998 progress meeting with the MTO and with their approval, additional boreholes were drilled and sampled to more fully outline the aerial and vertical extents of the soils in areas where the lower grade raises are proposed. These include the areas along the proposed Highway 11 south of the bridge structure, the south portion of the S-EW Ramp, and the north-northwest end of Boundary Road. Between September 17, 1998 and September 23, 1998, eight additional holes were

drilled (BH-25FP to BH-32FP) and on November 26, 1998, BH-33FP was completed. Thus, 30 boreholes were drilled and sampled in connection with the embankments.

The boreholes, cones and probes were advanced through the overburden soils using a Bombardier mounted CME-55 drill and a Dietric 50 drill, equipped with solid and hollow stem augers, and supplied by a soils drilling contractor, Master Soil Investigations Limited. Soil samples were obtained by using a 51 mm OD split-spoon sampler in conjunction with standard penetration tests at approximately 0.75 m and 1.5 m intervals. The standard penetration (N) values, together with the blows from the dynamic cone penetration tests, were recorded and used to provide an assessment of the compactness or consistency of the overburden soils. Sampling and testing procedures were in general accordance with ASTM D1586.

Selected, undisturbed, 50 mm diameter, "Shelby" tube samples (50.88 mm outside diameter) were also obtained in cohesive deposits. Field vane testing was also completed in the boreholes and the auger probes throughout the cohesive soils to measure the *in situ* undrained shear strength of the cohesive soils. The field vane used had dimensions of 152 mm long by 70 mm diameter, not including the pointed 45° point. Torque measurement was by using two calibrated scales on a calibrated lever arm threaded to the drill rods. The testing was in general accordance with ASTM D2573.

All of the recovered soil samples were taken to Trow's Sudbury laboratories for additional examination, identification and laboratory testing.

At the three borehole locations, at each of the three bridge foundation elements, conventional rock coring techniques were used to advance the boreholes approximately 3 m into the underlying bedrock. A BQ size core barrel and casing was used and core samples of the bedrock were retrieved for rock quality determination and classification.

Details of the soil and bedrock conditions encountered in the boreholes are included on the logs in the attached Appendix B.

### 1.3.3 Laboratory

The laboratory testing program for selected soil samples consisted of the following:

- Natural moisture content
- Grain size distribution

- Laboratory shear tests
- Uniaxial compression tests
- Atterberg limits
- 1-d consolidation tests

The laboratory test results are summarized on the attached borehole logs in Appendix B and are presented in Appendix C, also.

## 1.4 Subsurface Conditions

The borehole locations are shown on the site plans, Figures A2a, A2b, A3 and A4, in Appendix A. Included in Appendix B are the borehole, probe and dynamic cone penetration logs. Soil sections, longitudinal, as well as at each of the three foundation elements, are plotted on Figures A3 and A4. Stratigraphic sections along Highway 11, Boundary Road and the various ramps are presented as Figures A5 to A9, inclusive. In general, the following different soil layers were encountered, with increasing depth:

- organics
- silt to sandy silt
- silty clay
- silt
- sand/sand and gravel
- bedrock

A summary of the above soil strata encountered in the boreholes, and inferred from the probes and dynamic cone penetration tests, is presented below.

### 1.4.1 Organics

The majority of the site is overlain by a surficial layer of black to dark brown organics on the flat, poorly drained terrain. Based on the soil exploration data, it appears that the organics consist of very soft, fibrous peat and ranges in thickness to about 1.7 m.

Peat was not encountered in Boreholes BH-18FP to 21FP, inclusive. These holes are located in the area mapped as bedrock drift complex and a thin (up to about 600 mm) layer of sandy topsoil was encountered on the surface.

### 1.4.2 Silt to Sandy Silt

A thin deposit of silt to sandy silt was encountered in all boreholes, except BH-18FP, 19FP and 20FP, immediately beneath the peat. These three holes encountered sand or sand and gravel, as described below. The silt contains some root fibres and is grey and wet, and locally contained very thin clay laminations. The standard penetration resistance "N" values ranged from 3 to 30 blows/300 mm, indicating a very loose to compact condition. The thickness ranged from about 1.0 m to 4.5 m, and the moisture content from about 15% to 25%.

### 1.4.3 Silty Clay

Beneath the upper silt, a stratum of silty clay was encountered as the principal soil in all boreholes except BH-18FP, 19FP, 20FP and 21FP, at the western end of the site, where sand and gravel was encountered.

The silty clay persists to depths of up to about 18 m, with a maximum thickness of about 14 m. Generally, the silty clay is thinly laminated with silt and clayey silt (varved). The individual layers, or laminations, varied in thickness from a few millimetres to a few centimetres, but in general were less than about one centimetre. The proportions of clay to silt and clayey silt varied also, but in general the clay portion dominated.

*In situ* field vanes, laboratory unconfined and laboratory shear vane testing indicate that the silty clay has a relatively stiff crust with undrained shear strengths exceeding 100 kPa. The silty clay consistency, however, reduces with depth to soft to firm, with undrained shear strengths of 20 kPa to 30 kPa, at depths of about 5 m to 6 m, and gains strength thereafter to a maximum value of about 40 kPa at a depth of about 15 m. Sensitivity, the measure of peak shear strength to remoulded shear strength, ranged from 2 to 8, with an average of about 4, indicating the silty clay is moderately sensitive. The undrained shear strength profile which includes shear strength data from all boreholes is shown on Figure A10.

It is evident from the data and interpretation presented on Figure A10, that the silty clay is heavily to moderately overconsolidated in the upper 4 m to 5 m, becoming lightly overconsolidated with increasing depth. Preconsolidation pressures have been estimated to range as high as about 400 kPa

near the top of the silty clay. Overconsolidation ratios have been estimated to range from over 30 to less than 2.

The natural moisture content of the silty clay varies from about 30% to almost 60% (depending on the silt content). Atterberg limit determinations provided the following approximate ranges: Plastic Limit, 15 to 24; Liquid Limit, 25 to 60; Plasticity Index, 7 to 25. These data indicate that, in general, the clay can be described as a medium plasticity silty clay, CI (in accordance with the MTO Soil Classification System). The data are shown on the borehole logs, on Figure A10, and in Appendix C.

Four conventional one-dimensional consolidation tests were performed on samples of the silty clay extruded from thin walled "Shelby" tubes, obtained from four boreholes. In addition, horizontal specimens were subsampled from two of the "Shelby" tubes and similarly tested, in order to measure the horizontal coefficient of consolidation. The results are presented graphically on strain vs. pressure and coefficient of consolidation vs. pressure graphs on Figures C8 to C11, inclusive, in Appendix C. The data are also summarized below in Table 1-1.

<b>Table 1-1. Measured and Calculated Consolidation Test Parameters on Silty Clay</b>				
<b>Parameter</b>	<b>Borehole and Sample Depth</b>			
	<b>BH-7FP 6.3m</b>	<b>BH-8FP 5.0m</b>	<b>BH-26FP 9.4m</b>	<b>BH-30FP 7.9m</b>
Initial Void Ratio, $e_0$	0.99	1.31	1.07	1.31
Compression ratio, $C_c' (= C_c/(1+e_0))$	0.08	0.20	0.07	0.21
Recompression ratio, $C_r' (= C_r/(1+e_0))$	-	0.015	0.002	0.035
Coefficient of consolidation (recompression), $C_{vr}$	11	35	22-47	14-17
Coefficient of consolidation (virgin), $C_v$	12-15	7-14	12-15	2-6
Horizontal coefficient of consolidation, $C_h$	-	-	35-45	8-18
$C_v/C_v$	-	-	2.3 - 3.6	1.4 - 8.6
Coefficient of secondary compression, $C_{\alpha\epsilon}^*$	0.002-0.003	0.005-0.006	0.002-0.003	0.003-0.008
Notes: Coefficients of consolidation in units of $m^2/year$				
* $C_{\alpha\epsilon} = \Delta H/H_0$ (= secondary compression ratio, in some literature)				

#### 1.4.4 Silt

A second zone of essentially non-plastic silt was generally encountered beneath the silty clay, where the silty clay is present. Locally, very thin silty clay and/or sandy silt layers were encountered. The silt varies in thickness from approximately 1.5 m to 5.5 m thick. The standard penetration resistance "N" values ranged from about 2 to 14 blows/300 mm, indicating a very loose to compact condition. Moisture contents are generally in the range of 20% to 30%.

#### 1.4.5 Sand/Sand and Gravel

A basal deposit of sand and/or sand with gravel was encountered as the principal soil in BH-18FP, 19FP, 20FP and 21FP, to depths of auger refusal ranging from about 2.0 to 6.7 m. These holes are located on the western side of the project site. The basal sand and gravel was also encountered directly above the bedrock in the three cored holes at the bridge site (BH-1FF, 2FF, 3FF) and beneath the lower silt in the remaining boreholes, where the silt was fully penetrated.

The proven thickness of this stratum ranges from about 2 m to 9 m, where encountered. Standard penetration resistance "N" values ranged from 2 to over 50 blows/300 mm, indicating a very loose to very dense condition. The higher N values are likely due to a larger proportion of gravel and, perhaps, cobbles. Moisture contents range from about 8% to 20%.

#### 1.4.6 Bedrock

The presence of bedrock was established by retrieving "BQ" size cores in the sampled boreholes (BH-1FF, 2FF and 3FF), i.e. at one borehole beneath each of the three foundation elements. Based on the borehole, probe and dynamic cone penetration tests, the bedrock was established at the following depths:

- East Abutment (BH-3FF and C-3FF)  
18.5 m (~El. 294.6 m)
- Centre Pier (BH-2FF and C-2FF)  
19 m (~El. 294.4 m)
- West Abutment (BH-1FF and C-1FF)  
15 m (~El. 298.3 m)



Detailed descriptions of the rock are presented in Table 1-1 in Appendix B, following the logs. Generally, the bedrock can be described as a pinkish, light grey, biotite-hornblende gneiss. The rock is strong and unweathered for the most part.

Rock core recovery was 100% for all runs and the Rock Quality Designation (RQD) values ranged from 69% to 96%. The bedrock quality, based on the RQDs, is described as fair to excellent.

## 1.5 Groundwater Conditions

Information regarding the groundwater levels at the site was obtained by measuring the water levels in the open boreholes after completion of drilling. The measured groundwater levels are shown on the borehole logs and the stratigraphic sections. In general, the groundwater table at the time of the field work was established close to the ground surface (within the upper metre). In several areas, however, ponding water, up to 400 mm deep, was evident above the ground surface in random, local depressions over the poorly drained, flat terrain.

## **Part 2     Engineering Discussion and Recommendations**

### **2.1     Introduction**

The following subsections address geotechnical design considerations pertaining to the proposed two-span bridge for the Boundary Road, Trout Creek By-Pass (King's Highway 11) interchange, as well as the approaches and embankment fills.

A two-span bridge is proposed to carry Boundary Road traffic over the four lanes of the new By-Pass. Each span will be about 43 m in length. The central pier will be located in the By-Pass median, with the abutments located on the west and east sides of the north and south bound lanes of Highway 11.

Fills of up to about 12 m height above existing grade will be required to achieve the design objectives for the realigned Highway 11, Boundary Road (new), and the various ramps. Over 1,600 lineal metres of Highway 11 and Boundary Road construction will require fill, while over 2,000 lineal metres of the various ramps will require fill construction.

It is understood that the project is scheduled for completion within a two year time frame. Accordingly, some of the discussions and design recommendations presented herein are based on this time frame.

### **2.2     Foundations**

In view of the presence of substantial thicknesses of weak and compressible clay at the locations of the three proposed foundation elements, it is not considered feasible to support the proposed bridge foundations on spread footings. Accordingly, it is recommended that the structure be supported on end-bearing steel H-piles driven to refusal on the underlying bedrock. Bedrock was encountered at depths of approximately 15 m (west abutment), 19 m (central pier) and 18.5 m (east abutment).

Current design practice requires that consideration be given to downdrag loads that may be generated as a result of soil consolidation due to fills, groundwater table fluctuations and the soil stresses induced by pile installation. We consider that, even after primary consolidation of the clayey soils due to the fill placement is complete, the potential for downdrag loads due to events such as significant groundwater lowering and the long term process of secondary compression exists.

Secondary compression of the clay foundation soils is discussed further in a subsequent section of this report. Accordingly, we have reduced the pile capacities given in the following subsection, where applicable.

## 2.2.1 Pile Capacity and Length

Piles driven to bedrock can be designed based on the following ultimate limit states (ULS) design values, and should not exceed the factored axial resistance values, as shown in Table 2-1, below.

<b>Table 2-1. Design Pile Capacities (kN)</b>									
	<b>HP 310x79</b>			<b>HP 310x110</b>			<b>HP 310x132</b>		
Factored Structural Capacity	2325			3285			3890		
Factored Axial Resistance (MTO*)	1430			2000			2300 (estimate)		
Pile Location ----->	W	P	E	W	P	E	W	P	E
Factored downdrag load	1450	350	1095	1490	355	1120	1505	360	1135
Factored Axial Capacity at ULS (OHBDC)	875	1975	1230	1795	2930	2165	2385	3530	2755
<b>Factored Axial Capacity at ULS (Design)**</b>	<b>875</b>	<b>1430</b>	<b>1230</b>	<b>1795</b>	<b>2000</b>	<b>2000</b>	<b>2300</b>	<b>2300</b>	<b>2300</b>
Notes: MTO* = Structural Office Policy Memo 98-01, April 15, 1998 W = west abutment, P = pier, E = east abutment ** Factored axial capacity at ULS (Design) is the lesser of, a) factored structural capacity minus factored downdrag load, or b) factored axial resistance. SLS capacity not applicable to piles driven to bedrock.									

The axial capacity at SLS is not applicable to steel piles driven to bedrock because very high loads, in excess of the ULS capacity, are required to produce unacceptable deformations. The pile structural capacity will govern.

The capacities given are axial capacities, for both vertical and inclined piles. No reduction is required for inclined piles. Notwithstanding the above, the horizontal and vertical loads acting on all piles must be resolved axially, such that the vector sum of these loads does not exceed the ULS capacity provided above.

For design purposes, the modulus of horizontal subgrade reaction for steel H-piles can be taken as 8,000 kN/m<sup>3</sup> for the cohesive soils (clays) and 40,000 kN/m<sup>3</sup> for the cohesionless soils.

Based on the attached borehole logs in Appendix B, the following Table 2-2 shows a summary of the approximate bedrock elevation at the test locations at which piles would be expected to be founded. Figures A3 and A4, in Appendix A, show interpreted soil and rock subsurface profiles at the two abutments and pier.

<b>Table 2-2. Estimated Pile Tip Refusal Depths and Elevations</b>			
<b>Location</b>	<b>Boreholes &amp; Probe Holes</b>	<b>Approximate Existing Overburden Thickness (m)</b>	<b>Approximate Bedrock Elevation (m)</b>
West Abutment	Borehole 1FF Cone C-1FF	15.0	298.3
Centre Pier	Borehole 2FF Cone C-2FF	19.0	294.4
East Abutment	Borehole 3FF Cone C-3FF	18.5	294.6

It should be noted that the elevations given in Table 2-2 are based on the findings in the boreholes and dynamic cone holes drilled near the abutments and pier locations. Interpolation between boreholes and probe holes is approximate; accordingly, actual founding elevations will depend on the conditions encountered at the time of construction. Although not experienced in the borings advanced at this site, the bedrock elevation in northern Ontario can be variable and may change significantly over a short distance.

Pile caps should be provided with a minimum of 2.0 m of earth cover for frost protection.

### **2.2.2 Pile Installation**

As discussed in following subsections of this report, substantial settlements of the approach fills will occur as will some associated lateral strains. Accordingly, piles should not be installed until after the majority of the fill settlement has occurred, based on monitoring during construction. This is particularly important for inclined piles, that would be subjected to bending stresses.

All piles should be driven to bedrock. Since the boreholes indicate that the bedrock elevations are relatively uniform, the potential for irregular steeply sloping bedrock at the foundation locations is

considered to be low to moderate. The bedrock surface in this part of Northern Ontario, however, is known to be variable. As such, some minor problems may arise during pile seating. At some locations, the piles may have a tendency to skip over the bedrock surface resulting in alignment problems and deeper penetration. In the event that this problem occurs, somewhat longer piles may be required and, in some cases, piles may have to be added or replaced.

To minimize seating difficulties, rock injector points should be used to facilitate proper seating. All piles must be fitted with reinforcing plates welded to the flanges as per OPSD 3301 to minimize pile damage. It is recommended that, during pile driving and upon initial contact with the bedrock, the pile driving energy should be reduced and subsequently increased incrementally until the piles have been sufficiently seated.

## 2.3 Backfill

Backfill for abutments or retaining walls should consist of free draining granular materials such as Granular A, Granular B, or rock fill. Computation of earth pressures shall be in accordance with Section 6.7.4 of the Ontario Highway Bridge Design Code. Unfactored properties for backfill materials are provided in the following Table 2-3.

Table 2-3. Fill Types and Unfactored Geotechnical Properties					
Material	Friction Angle, $\phi'$	$\gamma$ (kN/m <sup>3</sup> )	$K_A$	$K_P$	$K_0$
Granular A	35 degrees	22.0	0.27	3.7	0.43
Granular B	30 degrees	21.0	0.33	3.0	0.50
Rock Fill	42 degrees	20.0	0.20	5.0	0.33
Note: Values given for $K_A$ and $K_P$ are for horizontal backfill, and will vary for other configurations. $K_A$ is the earth pressure coefficient corresponding to the active state. $K_P$ is the earth pressure coefficient corresponding to the passive state. $K_0$ is the earth pressure coefficient at rest.					

The mobilization of full active or passive resistance requires a measurable and perhaps significant wall movement or rotation, in general accordance with that indicated in Figure C6-7.1 of the OHBDC. Therefore, unless the structural element can tolerate these deflections, the at rest earth

pressure should be used in design. Alternatively, partially mobilized active and passive earth pressure coefficients may be used, in accordance with the deflections given in the figure.

The effects of compaction surcharge should be taken into account in the calculation of active and at rest earth pressures. In accordance with Section 6-7.4.3 of the OHBDC, the lateral pressure due to compaction should be taken as at least 16 kPa at the surface, and its magnitude should be assumed to diminish linearly with depth to zero at the depth where the active (or at rest) pressure is equal to 16 kPa. This pressure distribution should be added to the calculated active (or at rest) pressure. Notwithstanding, lighter compaction equipment and smaller lifts should be used adjacent to walls to prevent overstressing.

If rock fill is used as backfill behind abutments, the particle size should be limited to no greater than 300 mm and the backfill must be placed carefully in a manner that does not cause damage to the abutments or other structural components of the bridge.

Weep holes should be provided in the abutments and a filter wrapped, 150 mm diameter perforated PVC drainage pipe should be installed behind the abutments, at or slightly above the base.

## 2.4 Excavations

Minimal excavations for the pile caps will be required, since the grades will be raised. The existing organic zone (~600 mm to 1500 mm thick) will have to be removed from the entire construction area. In accordance with the Occupational Health and Safety Regulations for Construction Projects, excavation procedures for Type 3 soils will be adequate, provided the groundwater in the overburden soil is removed. If appropriate dewatering is not done, the soil would have to be classified as a Type 4 soil and any excavation greater than 1.2 m should then be sloped to 3 horizontal to 1 vertical, starting from the base of the excavation or appropriate shoring provided.

Dewatering of the excavations, to remove surface and groundwater infiltration, can likely be accomplished by pumping from sumps located within the excavations. The collected water should be discharged to suitable locations with appropriate sediment control measures.

## 2.5 Bridge Approach Fills and Embankments

The construction of the south interchange, including the bridge, will require embankment fills of up to about 12 m height, over areas with varying thicknesses of soft to firm, compressible clay soils. This creates two principal design and construction considerations: embankment stability and consolidation settlement. These two issues are discussed in the following; however, it is noted that the implications of the two issues are interrelated.

In all of the following discussions, it is assumed that all organic material is removed beneath the embankments and they are constructed on the native mineral soils, and fill heights should be measured from the top of the native mineral soil.

### 2.5.1 Embankment Stability

Highway embankments can be constructed with rock or earth fill. However, it is understood that in this area they are typically constructed using rockfill as the principal component. A typical MTO section will consist of a minimum (steepest) slope of 1.25H to 1V to a maximum height of 6 m. For embankment sections greater than 6 m height, a 2 m wide horizontal bench is required before the next 6 m, 1.25H to 1V lift is constructed. This geometry as well as a 16 m crest width has been assumed for the analyses.

Slope stability analyses have been performed using SLOPE/W (ver. 3.x), based on Bishop's Method using both total stress and effective stress parameters. The undrained shear strength profile shown on Figure A10 was used to provide the shear strength parameters for the clay soils. Table 2-4 lists the parameters used. Effective stress parameters were established from multistage consolidated undrained triaxial strength tests performed on representative samples from the project<sup>[1]</sup> as well as from various geotechnical literature and our experience.

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<sup>[1]</sup>Sample from Trout Creek Bridge Site, see Figure C12.

<b>Table 2-4. Layers and Geotechnical Parameters for Slope Stability Analyses</b>				
<b>Material (thickness - m)</b>	<b><math>\gamma_{total}</math> (kN/m<sup>3</sup>)</b>	<b><math>c_u</math> (kPa)*</b>	<b><math>\phi'</math></b>	<b><math>c'</math> (kPa)</b>
Rockfill	20.0	0	42°	0
Silty Clay 1 (2 m)	20.5	120 - 50	28°	8
Silty Clay 2 (1 m)	18.3	50 - 30	27°	7
Silty Clay 3 (1 m)	18.0	30 - 25	26°	5
Silty Clay 4 (2 m)	17.3	25	26°	4
Silty Clay 5 (2 m)	17.8	25 - 27.5	26°	4
Silty Clay 6 (2 m)	17.8	27.5 - 32.5	26°	4
Silty Clay 7 (2 m)	18.0	32.5 - 37.5	26°	4
Lower Silt (2 m)	19.5	0	31°	0
Sand and Gravel	21.5	0	35°	0
Notes: * See Figure A10 for the undrained shear strength profile Embankment crest width 16 m, side slopes = 1.25H:1V				

Many of the SLOPE/W graphical printouts, for the various analyses performed and discussed below, are included in Appendix D. Note that for the total stress analyses,  $C_u$  is shown as  $C_v$  on the printouts.

### 2.5.1.1 Total Stress Analyses

The results of the slope stability analyses, in terms of total stresses using undrained shear strengths for the clay, as would apply to rapid construction (short term stability), are shown graphically in the top panel of Figure A11 (Figure A11-A), and in Table 2-5, below. Interpolation of the curves is possible. All embankments up to about 7.5 m height can be constructed based on this analysis, with a resulting factor of safety against foundation failure greater than the recommended value of 1.3. Depending on the clay thickness underlying the embankments, however, some of the higher embankments can not be built rapidly, based on this analysis.

In general, the factor of safety decreases not only with increased embankment height, but with increased clay thickness as well. These require special consideration for design and construction,



as discussed in the following subsections. The embankments with height/clay thickness combinations resulting in factors of safety greater than 1.3 should be safe, based on the total stress analyses conducted.

<b>Table 2-5. Factors of Safety for Total Stress (Short Term) Stability Analyses</b>					
<b>Embankment Height</b>	<b>Silty Clay Thickness</b>				
	<b>4 m</b>	<b>5 m</b>	<b>6 m</b>	<b>8 m</b>	<b>&gt;10 m</b>
6 m	2.15	1.89	1.74	1.64	1.59
8 m	1.80	1.56	1.41	1.31	1.24
10 m	1.54	1.31	1.19	1.09	1.02
12 m	1.40	1.15	1.03	<1	<1
Notes: Embankment crest width 16 m, side slopes = 1.25H:1V, with mid height bench above 6 m Shaded data indicates factor of safety < 1.3					

### 2.5.1.2 Total Stress Analyses with Stabilizing Berms

We have considered the use of stabilizing berms to allow the safe, rapid construction of the embankments, while maintaining a calculated factor of safety 1.3 against foundation failure. Typical berm heights would be in the order of one-third to one-half the embankment height. The various calculated configurations are shown in Table 2-6, below, and graphically on Figure A11-B, in Appendix A.

While the use of properly designed berms would allow for rapid construction while maintaining adequate factors of safety against foundation failure, the settlement issues, discussed in Section 2.5.2, below, would persist.

**Table 2-6. Required Berm Configurations for Short Term FS = 1.3**

Embankment Height	Clay Thickness	Berm Height	Berm Length
8 m	8 m	2.5 m	1.0 m
	10 m	2.5 m	4.5 m
	12 m	2.5 m	5.5 m
10 m	6 m	3.5 m	5.0 m
	8 m	3.5 m	9.5 m
	10 m	3.5 m	14.0 m
	12 m	3.5 m	17.0 m
12 m	5 m	4.0 m	5.0
	6 m	4.0 m	8.5
	8 m	4.0 m	15.0
	10 m	5.0 m	19.5
	12 m	6.0 m	23.0

Notes: Embankment crest width 16 m, side slopes = 1.25H:1V

### 2.5.1.3 Effective Stress Analyses

The results of our effective stress stability analyses are presented in Table 2-7, below and are shown graphically on Figure A12-A. The analyses are based on the assumption that the embankments will be constructed to full height over a period of 3 months (steady ramp rate), thereby allowing for some dissipation of excess porewater pressures that will develop in the clay. The pore pressure parameter,  $A$ , that measures the pore pressure response to increases in total stress was assumed to be equal to 0.7, based on the results of our triaxial tests and geotechnical literature. Based on this analysis, and considering only vertical consolidation drainage (no sand drains or wicks), as governed by the  $C_v$  values discussed in Section 2.5.2, the figure shows that steady slow (3 month loading) should result in higher factors of safety at the end of construction. Most embankments higher than about 8 m will require special consideration, as discussed in the following.

A longer, steady ramping of the embankment loads, say 6 months, would increase all factors of safety significantly.

**Table 2-7. Factors of Safety for Effective Stress Stability Analyses at End of Construction (3 months)**

Embankment Height	Clay Thickness			
	6 m	8 m	10 m	12 m
6 m	1.43	1.40	1.39	1.38
8 m	1.35	1.28	1.27	1.26
10 m	1.28	1.18	1.16	1.14
12 m	1.21	1.10	1.07	~ 1.0

Notes: Embankment crest width 16 m, side slopes = 1.25H:1V, with mid height bench above 6 m  
Shaded data indicates factor of safety < 1.3

#### 2.5.1.4 Effective Stress Analyses using Wick Drains

Table 2-8, below, and Figure A12-B (in Appendix A) shows the factors of safety, again based on effective stresses, with a 3 month construction period and the use of vertical sand or wick drains. The calculations are based on drain spacings of 3 m, 4 m, and 5 m, installed in a triangular pattern, as discussed in Section 2.5.2. The use of such vertical drainage will allow the construction of all of the proposed embankments while maintaining an adequate factor of safety against foundation failure.

**Table 2-8. Factors of Safety for Effective Stress Stability Analyses at End of Construction Using Wick Drains (3 months)**

Embankment Height	Wick Drain Spacing		
	3 m	4 m	5 m
8 m	1.64	1.53	1.43
10 m	1.51	1.41	1.31
12 m	1.44	1.35	1.24

Notes: Embankment crest width 16 m, side slopes = 1.25H:1V, with mid height bench above 6 m  
Shaded data indicates factor of safety < 1.3

## 2.5.2 Consolidation Settlement of Embankments

### 2.5.2.1 Magnitudes of Consolidation Settlement

Consolidation calculations have been performed using the effective stress profiles shown on Figure A10 and an average compression ratio ( $C_c' = C_c / [1 + e_0]$ ) equal to 0.18. This average value was established from the consolidation test data as well as from geotechnical literature. A recompression index ( $C_r' = C_r / [1 + e_0]$ ) equal to 0.015 was used. The stress increases in the foundation soils due to the embankment loadings have been calculated based on the Boussinesq elastic solutions. Recompression and virgin consolidation has been calculated based on the assumed existing overburden pressure and preconsolidation stress profile shown on Figure A10.

The results of the calculations for embankment centerline settlement using rockfill to construct the embankments are provided in Table 2-9, below and are shown graphically on Figure A13.

<b>Table 2-9. Estimated Embankment Centerline Consolidation Settlement (mm)</b>						
<b>Embankment Height</b>	<b>Clay Thickness</b>					
	<b>2 m</b>	<b>4 m</b>	<b>6 m</b>	<b>8 m</b>	<b>10 m</b>	<b>12 m</b>
2 m	15	25	30	40	45	50
4 m	20	55	115	170	210	240
6 m	25	105	205	295	370	430
8 m	35	155	285	405	505	590
10 m	55	200	360	505	630	735
12 m	70	240	420	585	730	860
Notes: Embankment crest width 16 m, average side slopes = 1.5H:1V Values rounded to nearest 5 mm						

As an example, the western approach for the proposed bridge requires a grade raise of about 12 m and the clay thickness at this location is about 8 m. Accordingly, about 585 mm of consolidation settlement of the clay (and overlying soil and fill) can be expected.

Figures A14, A15 and A16 show the calculated settlements (for embankment heights ranging from 2 m up to 12 m) that may be expected under the various portions of the embankments, measured out from the centerline to the toe. It is apparent that in addition to total consolidation settlements, differential settlement between the embankment centerline and crest will occur, and the design should take these into account by specifying overbuilding of the embankments.

All of these charts will allow superposition of overlapping embankments and the total consolidation settlement can be taken as the sum of the individual cases.

If consideration is to be given to the use of light weight fill, for preliminary design purposes, the total consolidation settlements can be reduced **approximately** by proportion of rockfill replaced by lightweight polystyrene (eg. 30% replacement of fill height with polystyrene will result in about 70% of the indicated settlement).

#### 2.5.2.2 Time Rate of Consolidation Settlement

The time rate of consolidation settlement has been calculated for the various cases of embankment height and clay thickness for vertical drainage only, and for sand (or wick) drains spaced in a triangular pattern at distances of 3 m, 4 m and 5 m. It is also assumed for the purposes of calculation, all embankments are raised steadily over a 3 month construction period. A coefficient of consolidation,  $C_v$ , of 10 m<sup>2</sup>/year has been used in the analyses, based on the results of the consolidation tests and the geotechnical literature. For radial (horizontal) drainage, a conservative ratio of horizontal to vertical coefficient of consolidation of 3 has been assumed, based on the consolidation test data and the geotechnical literature. It is noted that the thinly laminated nature of the predominantly clay foundation soils make consideration of the use of vertical drains attractive.

Figure A17 shows the fraction of excess porewater pressure remaining (ie. the fraction of settlement remaining) for the various cases considered for vertical and for radial drainage as a function of time following start of construction. These curves are based on a three month, steady loading construction period and can be applied to the settlement magnitudes given in the previous figures. It can be seen that the use of vertical drains greatly accelerates the rate of settlement, compared to drainage without vertical drains.

Figures A18 to A23, inclusive, show the calculated magnitudes of embankment centerline consolidation settlement remaining for the various heights of rockfill embankments.

### **2.5.3 Wick Drain Design and Construction Considerations**

Detailed vertical drain design is beyond the scope of this phase of the project. The charts and tables provided herein and discussed above (embankment stability, settlement magnitude, time rate of settlement) will allow the detailed design of vertical drain installation patterns, depending on project scheduling constraints and tolerable post construction settlements. Relative to time rate, three vertical drain spacings have been presented in addition to time rates for no drains. It is expected that larger spacings (and perhaps no drains) can be used in some of the non-critical areas, such as near to the toes of the embankments, as well as where total settlements can confidently be completed in a relatively short period of time. We expect that by optimizing the drain installation pattern once the final embankment configuration for the project is established, along with scheduling constraints, a significant savings in the drain installation costs will result, compared to specifying a single spacing for the entire project.

It is expected that if vertical drainage is to be implemented on this project, drainage wicks will be selected over sand drains, primarily due to cost. It is noted that either should perform similarly, provided that in the case of drainage wicks (or prefabricated vertical drains), their discharge capacity is not impaired by kinking that will reduce their efficiency. Vertical strains of up to about 8% have been calculated due to consolidation settlement. Systems such as those with a filter fabric covered ribbed core construction should be used, since these appear to better maintain their hydraulic properties than the corrugated or grooved types, for example, when kinking or folding occurs, or due to the high lateral pressures at depth within the consolidating soil.

Additionally, these drains should fully penetrate the clay and underlying silt, and be terminated a minimum of 1 m into the underlying sand, to ensure adequate drainage.

### **2.5.4 Secondary Compression of Clays**

Calculations have been performed to estimate the magnitudes of secondary compression of the foundation clays, following the completion of primary consolidation. For purposes of analysis and discussion, the primary consolidation is assumed to be essentially complete within 1.5 years from the start of construction of the embankments. For this assumption to be valid, vertical sand drains or drainage wicks will be required for much of the project. The spacings required to achieve this can be determined from the previously described charts and figures, using the appropriate embankment height and clay thickness.

The stress increases in the clay foundation soils due to embankment loading will, in most cases, exceed the preconsolidation pressures when the embankment height exceeds about 2 m. The graphs in Figure A24 show the calculated embankment centerline settlement due to secondary compression. Table 2-10, below, summarizes the calculated secondary compression at 15 years. The calculations are based on use of a coefficient of secondary compression,  $C_{\alpha\epsilon}$ , of 0.004, based on the results of the consolidation tests and the geotechnical literature.

**Table 2-10. Calculated Secondary Compression at 15 Years**

Embankment Height	Clay Thickness					
	2 m	4 m	6 m	8 m	10 m	12 m
2 m	< 5 mm	< 5 mm	< 5 mm	10 mm	10 mm	10 mm
3 m	< 5 mm	< 5 mm	10 mm	20 mm	30 mm	35 mm
4 m	< 5 mm	10 mm	15 mm	25 mm	30 mm	40 mm
6 m	< 5 mm	10mm	15 mm	25 mm	30 mm	40 mm
8 m	< 5 mm	10mm	15 mm	25 mm	30 mm	40 mm
10 m	< 5 mm	10 mm	15 mm	25 mm	30 mm	40 mm
12 m	< 5 mm	10 mm	15 mm	25 mm	30 mm	40 mm

Notes: Values are rounded to nearest 5 mm. Refer to graphs, Figure A24.

While the magnitudes are essentially independent of the stress levels (as long as the preconsolidation pressure is exceeded), the graphs have been produced using the effective stress profiles shown in Figure A10, which takes into account the previously calculated loading and clay thickness cases where the preconsolidation pressure is exceeded.

As an example, at the western abutment where the fill height is about 12 m and the clay thickness is about 8 m, the lower panel of Figure A24 shows that the secondary clay compression after 5, 10 and 20 years may be about 15 mm, 20 mm and 30 mm, respectively.

It is noted that while the magnitude of the secondary compression can be controlled by limiting final stresses to less than the preconsolidation pressures, its rate cannot be accelerated.

### 2.5.5 Rockfill and Rockfill Settlement

Rockfill used in the embankments and approaches should consist of hard, durable, angular quarried rock. Rockfill should be tested for acid drainage potential and checked against the appropriate regulatory requirements for acceptability.

For the bridge abutments that are to be supported by driven steel H-piles, as discussed in a previous section, these will have to be installed after all the embankment fill has been placed. The driving of steel piles through rockfill will be difficult, if not impossible, unless care is taken to ensure that the rockfill meets a tight size range, generally between 50 mm and 150 mm. Larger rockfill sizes may require predrilling for pile installation. Consideration should be given to a 75 mm maximum size. For the majority of the embankments and approach fills, however, larger rockfill can be used.

The use of large rockfill, loosely placed or end dumped and compacted by conventional construction traffic only may result in long term settlements of up to about 2% of the fill height. Accordingly, a 10 m high rockfill may settle up to 200 mm following completion of construction. This is primarily due to the rockfill "adjusting" under highway service conditions.

In order to minimize the long term settlements associated with the rockfill itself, consideration can be given to specifying the use of a finer graded rockfill, with a maximum size of 400 mm. This finer graded rockfill should be placed in lifts limited to about 800 mm and compacted with heavy vibratory rollers. While the production of the finer graded rockfill and its placement, as described above, carries with it a cost premium, the long term performance of the finished highway pavement will be improved.

### 2.5.6 Surcharging of Embankment Fill

In most cases where the embankment fill height will exceed about 2 m, the final stresses in the silty clay foundation soils will exceed the preconsolidation pressure. This will result in a new "normally consolidated" condition. Accordingly, additional loadings will result in potentially large settlements, as shown on Figure A13 and in Table 2-9. It is therefore recommended that surcharging be implemented on these embankments. This should consist of a minimum 1 m additional fill (20 kPa), to be removed on completion of the required consolidation settlement, prior to final grading.



## 2.6 Culverts

While we have not specifically been requested to address culverts, it is expected that these will be required as part of the site drainage plan. The number of culverts and their locations are not presently known. Because of the settlement issues discussed previously, any centerline culverts installed prior to the embankment construction will settle with the embankment. Differential settlements between the embankment centerline and the toe may be as high as several hundred millimetres, as shown on the figures discussed previously. This must be considered in the design and several options are presented for your consideration.

Preferably, culverts should be located in areas where smaller settlements are expected, such as under the lower height embankments and thinner clay areas.

If culverts must be located in areas of greater potential settlement, they can be designed with a reverse camber to accommodate some of the differential settlement. In this case, they should be oversized and standing water at the inlets and/or outlets should be tolerable, since it will be impossible to predict (ensure) a positive final grade along the length of the culvert.

Alternatively, culverts can be supported on piled foundations. It is expected, however, that this technique will result in differential settlements between the pile supported culvert (essentially zero settlement) and the highway on either side, resulting in a pronounced "bump".

Finally, culverts in higher settlement areas may be installed following completion of the majority of the associated embankment settlement. In some cases, this may require deep excavations into the embankment fills.

## 2.7 Instrumentation and Construction Monitoring

Construction of embankments, based on effective stress analyses and design, are recommended to be monitored during construction. The effective stress analyses on which the design recommendations summarized herein are based, rely on excess pore water pressure dissipation (and resulting settlement). This should be monitored during construction using a properly designed and effective system of instrumentation, consisting primarily of piezometers installed in the clay, as well as settlement points. Lateral slope movements should also be monitored both during and following construction of the embankments.

Monitoring and continual interpretation of the data is essential in order to control the rate of excess porewater pressure generation and dissipation by controlling the rate of embankment fill placement. Excess porewater pressures must be kept below calculated threshold values that may indicate impending failure, that may require a slowing or stoppage of construction until pore pressures further dissipate sufficiently to maintain stability and allow construction to resume. The monitoring may, on the other hand, indicate that construction can proceed faster than originally planned, resulting in potential cost savings.

Notwithstanding the above, if stabilizing berms are designed for use to allow rapid construction of portions of the embankments, a monitoring program is still recommended. The use of berms is based on total stress analyses and design and does not require the construction monitoring of excess porewater pressures. However, some porewater pressure monitoring as well as settlement monitoring is recommended to allow for modifications to the final grading design, if necessary. This is primarily related to the amount of overbuilding of the final grades, in anticipation of the calculated settlements.

Detailed design of a construction monitoring program is beyond the scope of this phase of the project, but can be provided when the final design is nearing completion.

## 2.8 Closing Comments

The recommendations made in this report are in accordance with our present understanding of the project and are provided solely for the use of Marshall Macklin Monaghan and its design team for the design of the bridge foundations, approach fills and embankments at the South Interchange of the Highway 11 Trout Creek By-Pass Project. We request that we be retained to review the design and our recommendations as the design proceeds, to ensure that the final design is in general agreement with our recommendations and that our recommendations have been interpreted as intended.

A subsurface investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those reported at the test locations, we require that we be notified immediately in order to allow reassessment of our recommendations. It may then be necessary to carry out additional field work and analyses.

Contractors bidding on or undertaking the works should, relative to the subsurface conditions, decide on their own investigations, if deemed necessary, as well as their own interpretations of the

factual results provided herein, so they may draw their own conclusions as to how the subsurface conditions may affect them.

The information presented in this report is based on a limited investigation designed to provide information to support an overall assessment of the current geotechnical conditions at the site of the proposed Boundary Road and King's Highway 11 (Trout Creek By-Pass South Interchange). The conclusions presented in this report reflect site conditions existing at the time of the investigation. It is noted that the soil boundaries indicated on the logs are inferred from discontinuous sampling and observations during drilling. These boundaries are intended to reflect transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change.

This report has been prepared by Mr. D.N. Georgiou, P.Eng. and was reviewed by Messrs. S.E. Gonsalves, P.Eng., and E. Gonneau, P.Eng. The field investigation was performed by Messrs. I. Dumpis, C.E.T., R. Moore, C.E.T., and S. McAuliffe, and was supervised by Mr. E.A. Gonneau, P.Eng.

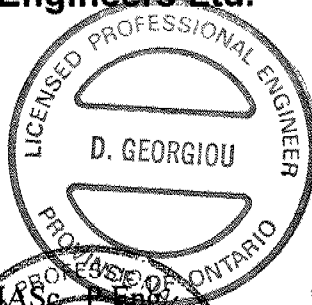
We trust this report is satisfactory for your purposes. Should you have any questions, please do not hesitate to contact us.

All the foregoing and attachments respectfully submitted,

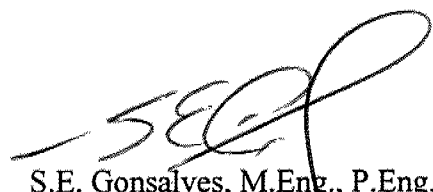
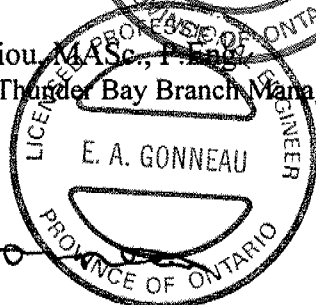
**Trow Consulting Engineers Ltd.**



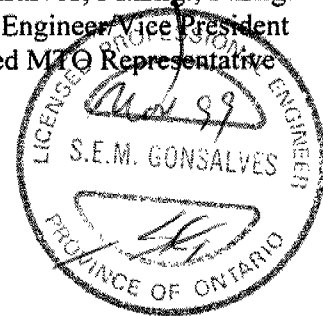
Demetri N. Georgiou, M.A.Sc., P.Eng.  
Principal Engineer/Thunder Bay Branch Manager



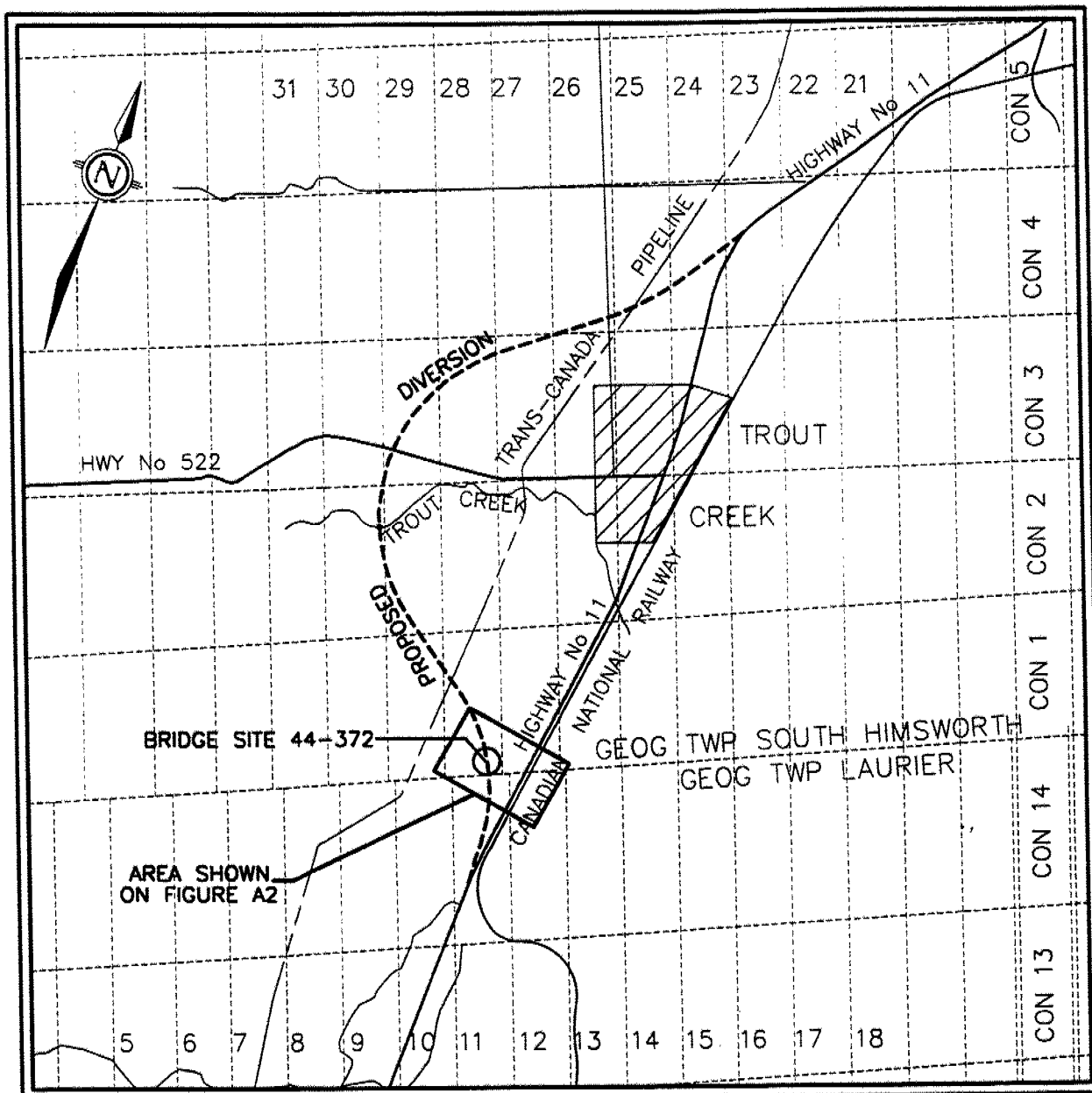
Eric A. Gonneau, P.Eng.  
Project Manager



S.E. Gonsalves, M.Eng., P.Eng.  
Principal Engineer/Vice President  
Designated MTO Representative



A



1 km 0 1 km



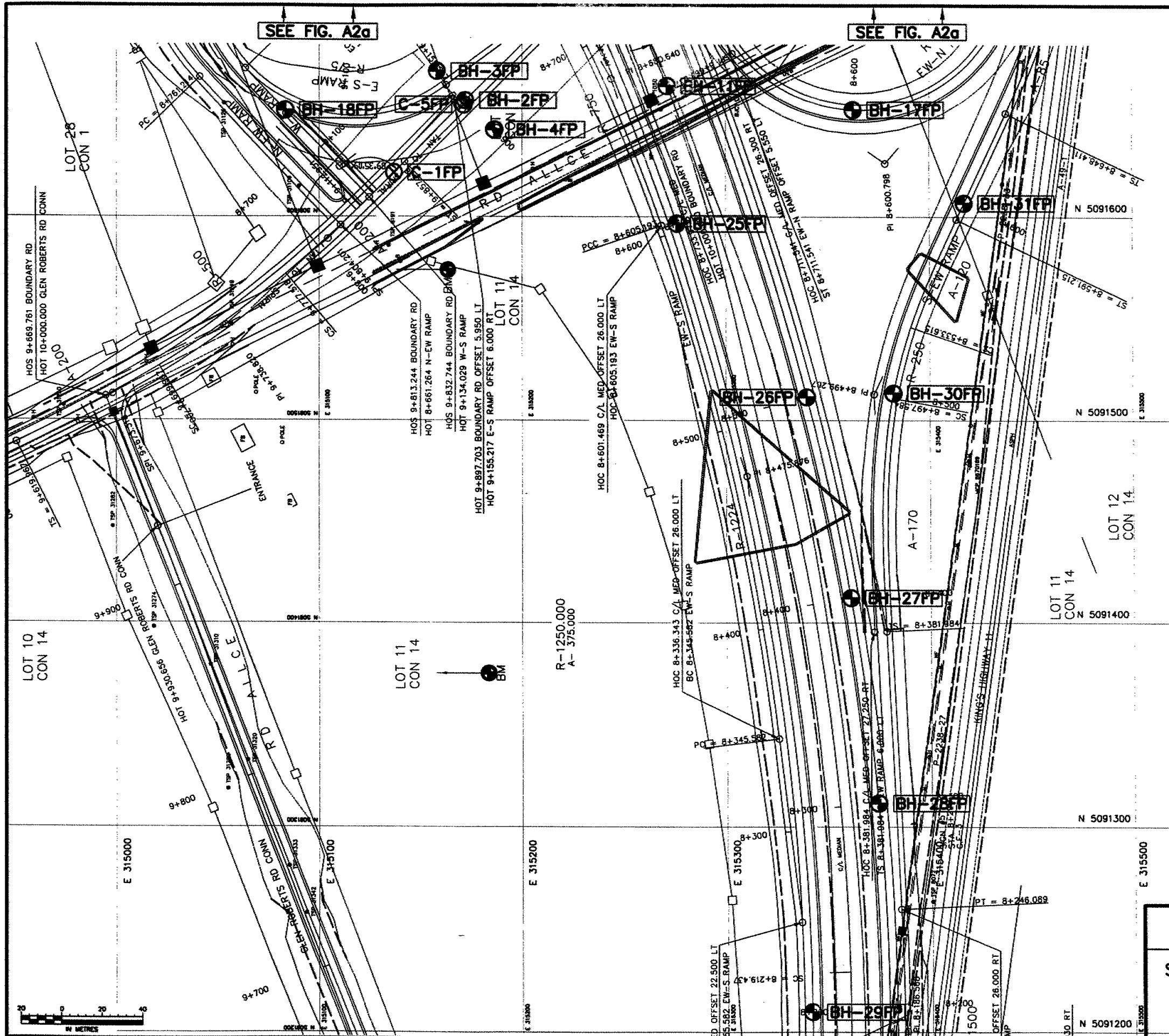
Trow Consulting Engineers Ltd.  
Thunder Bay, Ontario

FIGURE  
A1

KEY PLAN  
Trout Creek By Pass  
South Interchange

PROJECT NO.:	F-98179-A/G
SCALE:	AS SHOWN
DRAWN BY:	DT
CHECKED BY:	DG
DATE:	NOV. 24, 1999





HOC 8+74  
BC 8+74

# LEGEND:

- BH-10FP BOREHOLE
- C-3FP CONE PENETRATION TEST
- AP-13FF AUGER PROBE



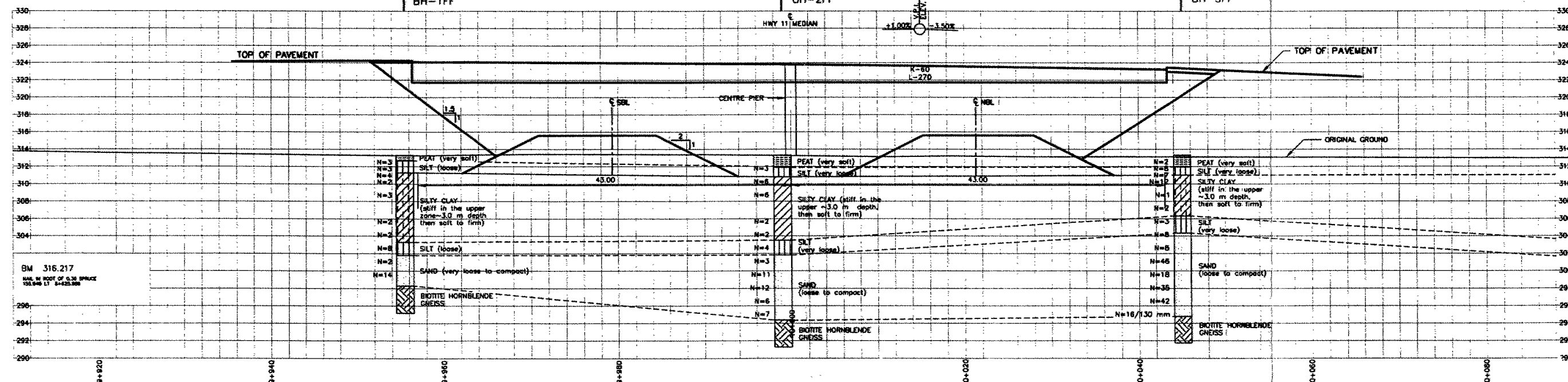
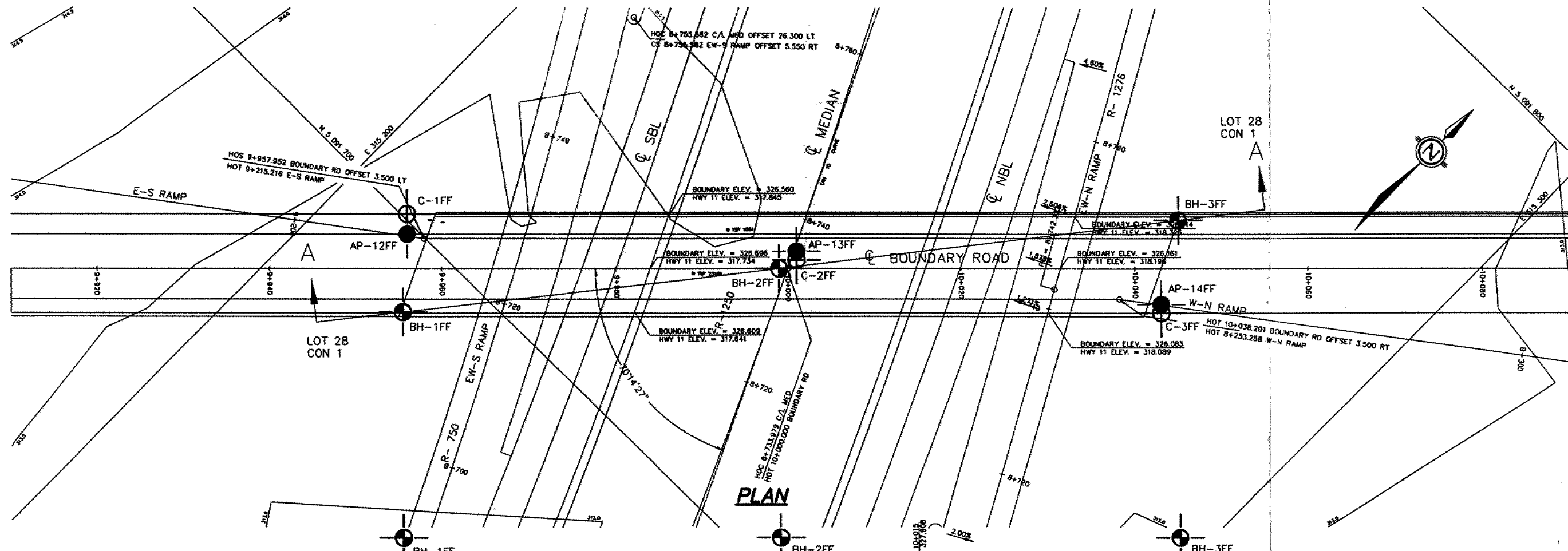
Trow Consulting Engineers Ltd.  
Thunder Bay, Ontario

FIGURE  
A2b

**SITE & BOREHOLE LOCATION  
PLAN**  
Trout Creek By Pass  
South Interchange

PROJECT NO.:	F-98179-A/G
SCALE:	1:2000
DRAWN BY:	DT
CHECKED BY:	DG
DATE:	NOV. 24, 1999





**SECTION A-A  
PROFILE OF BOUNDARY RD**

**NOTE:**

The stratigraphic boundaries and soil types have been established at Test Hole locations. Between Test Holes they are assumed and may be subject to error.



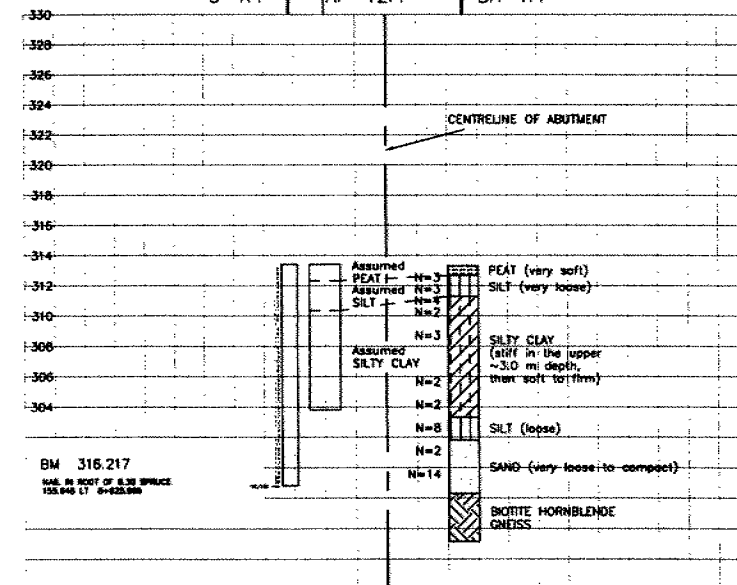
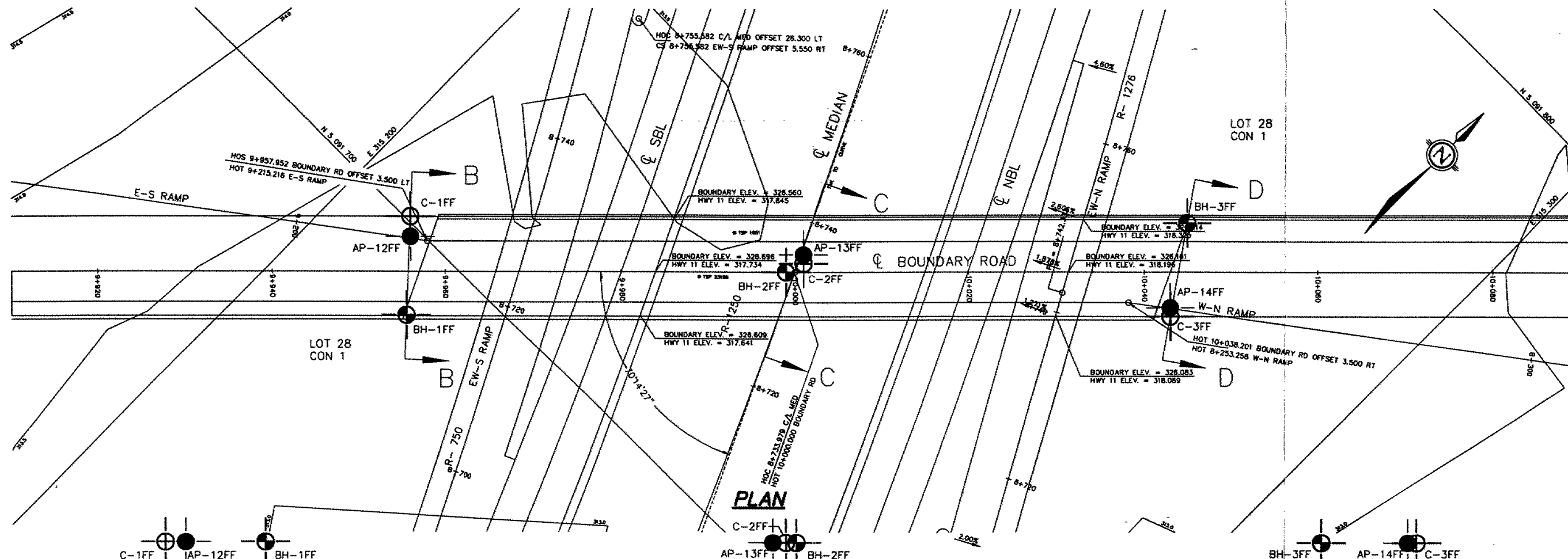
Trow Consulting Engineers Ltd.  
Thunder Bay, Ontario

**FIGURE  
A3**

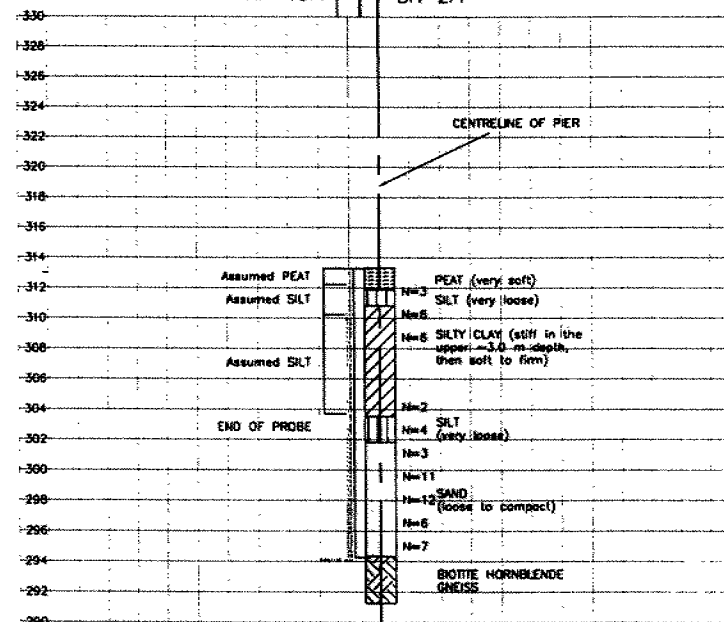
**PLAN & LONGITUDINAL SECTION  
AT PROPOSED BRIDGE**  
Trout Creek By Pass  
South Interchange

PROJECT NO.: F-98179-A/G  
SCALE: 1:500  
DRAWN BY: DT  
CHECKED BY: DG  
DATE: NOV. 24, 1999

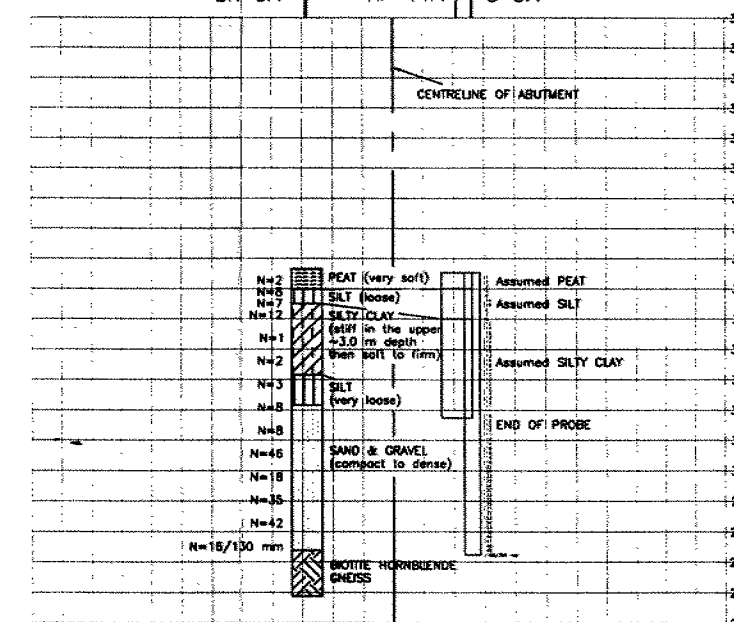




**SECTION B-B  
PROFILE OF BOUNDARY RD**



**SECTION C-C  
PROFILE OF BOUNDARY RD**



**SECTION C-C / PROFILE OF BOUNDARY RD**



**NOTE:**

The stratigraphic boundaries and soil types have been established at Test Hole locations. Between Test Holes they are assumed and may be subject to error.

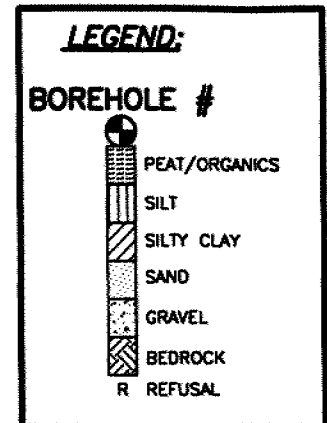
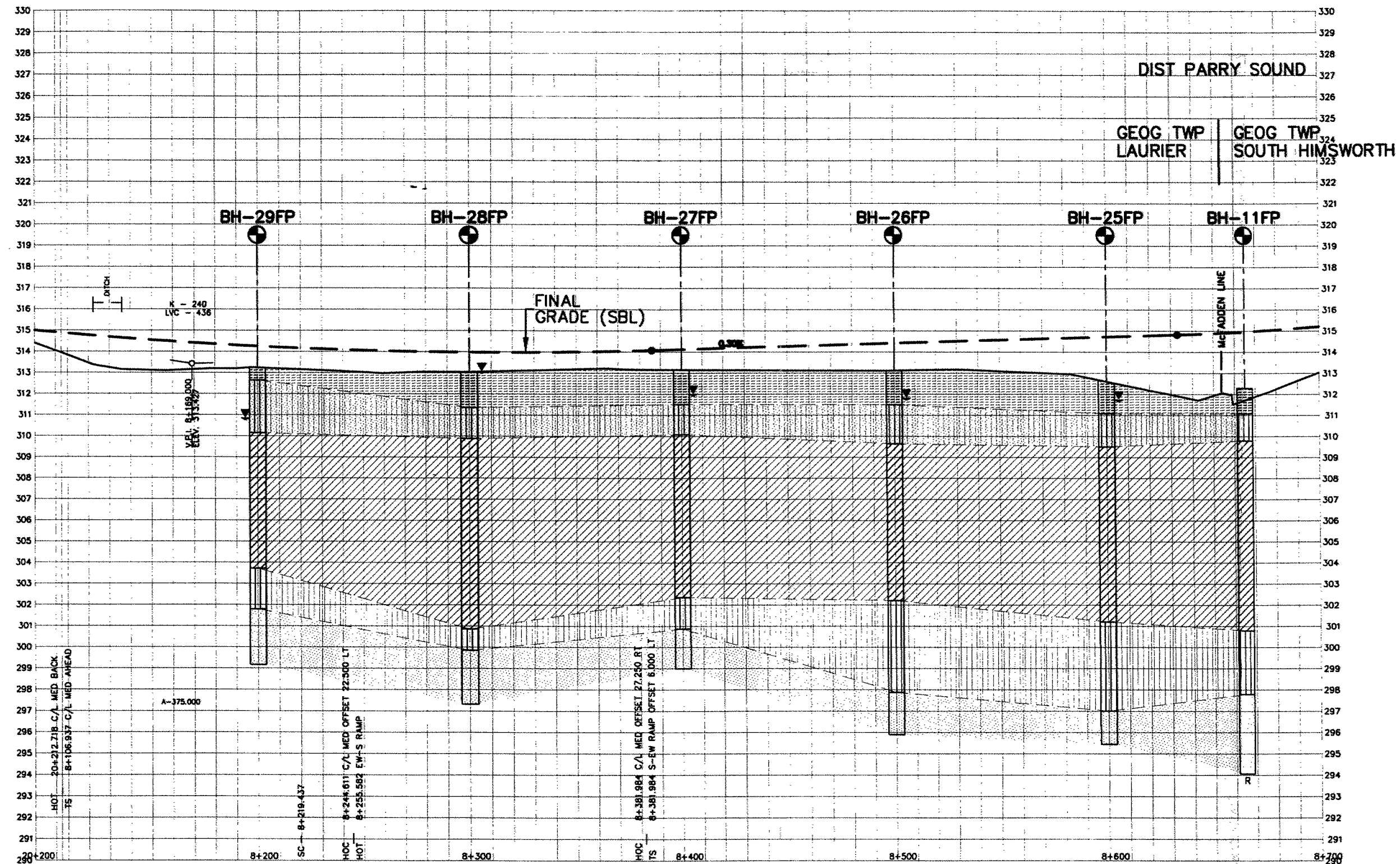


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FIGURE  
A4

PLAN & CROSS SECTIONS  
AT PROPOSED BRIDGE  
FOUNDATION ELEMENTS  
Trout Creek By Pass  
South Interchange

PROJECT NO.: F-98179-A/G  
SCALE: 1:500  
DRAWN BY: DT  
CHECKED BY: DG  
DATE: NOV. 24, 1999



**NOTE:**

The stratigraphic boundaries and soil types have been established at Test Hole locations. Between Test Holes they are assumed and may be subject to error.



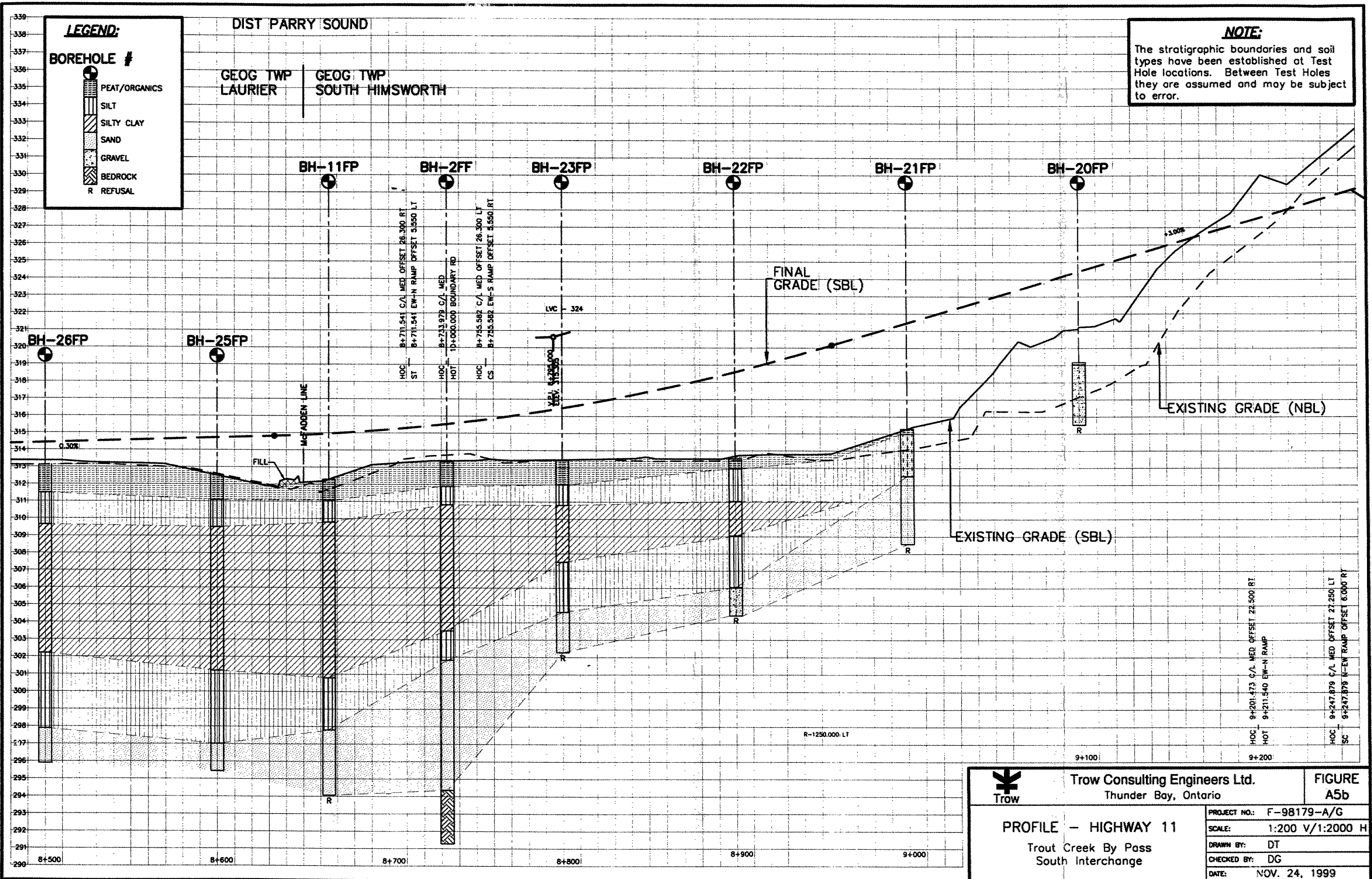
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Thunder Bay, Ontario

FIGURE  
A5a

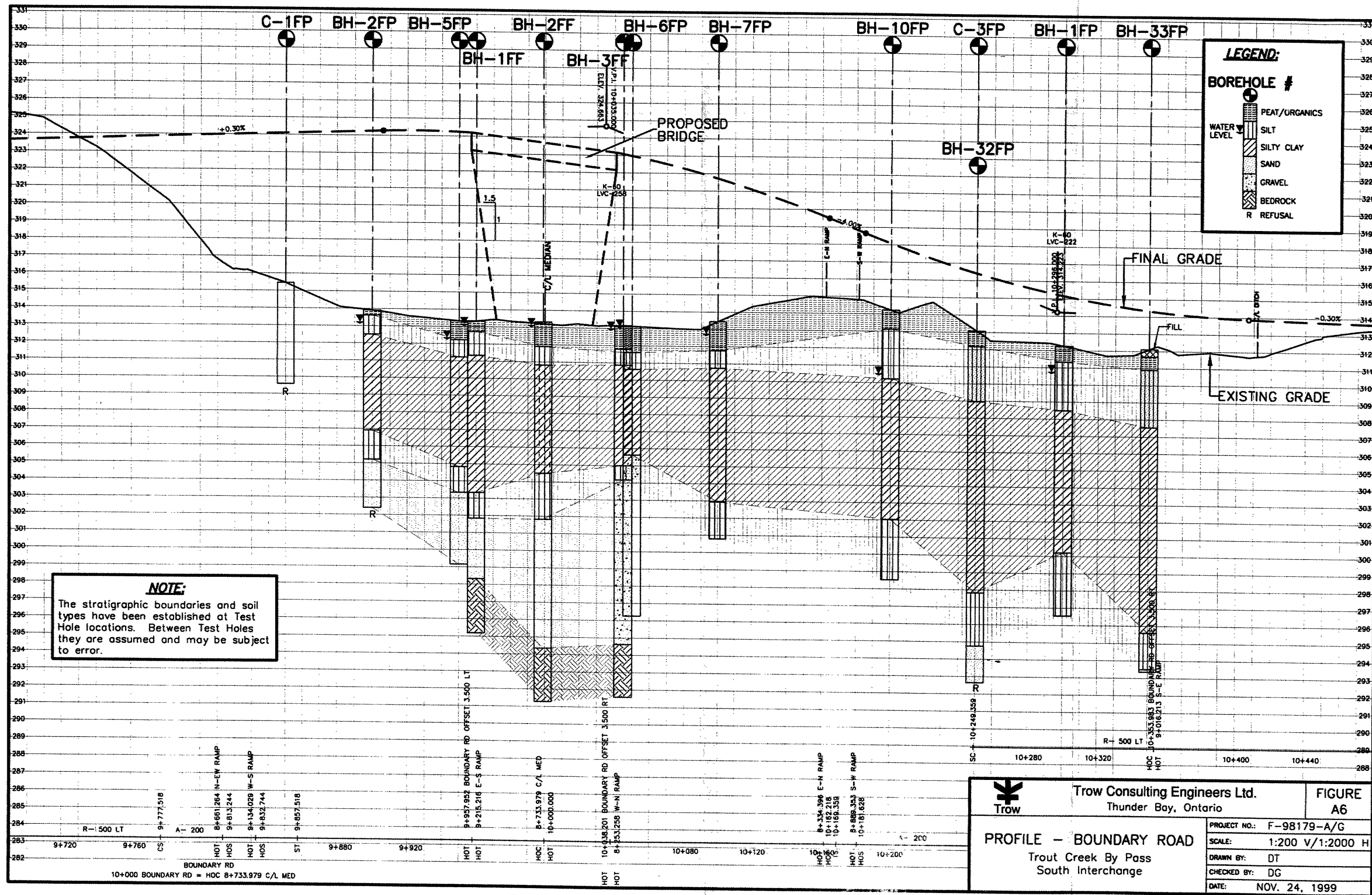
PROFILE - HIGHWAY 11

Trout Creek By Pass  
South Interchange

PROJECT NO.: F-98179-A/G  
SCALE: 1:200 V/1:2000 H  
DRAWN BY: DT  
CHECKED BY: DG  
DATE: NOV. 24, 1999







**LEGEND:**

**BOREHOLE #**

WATER LEVEL

PEAT/ORGANICS

SILT

SILTY CLAY

SAND

GRAVEL

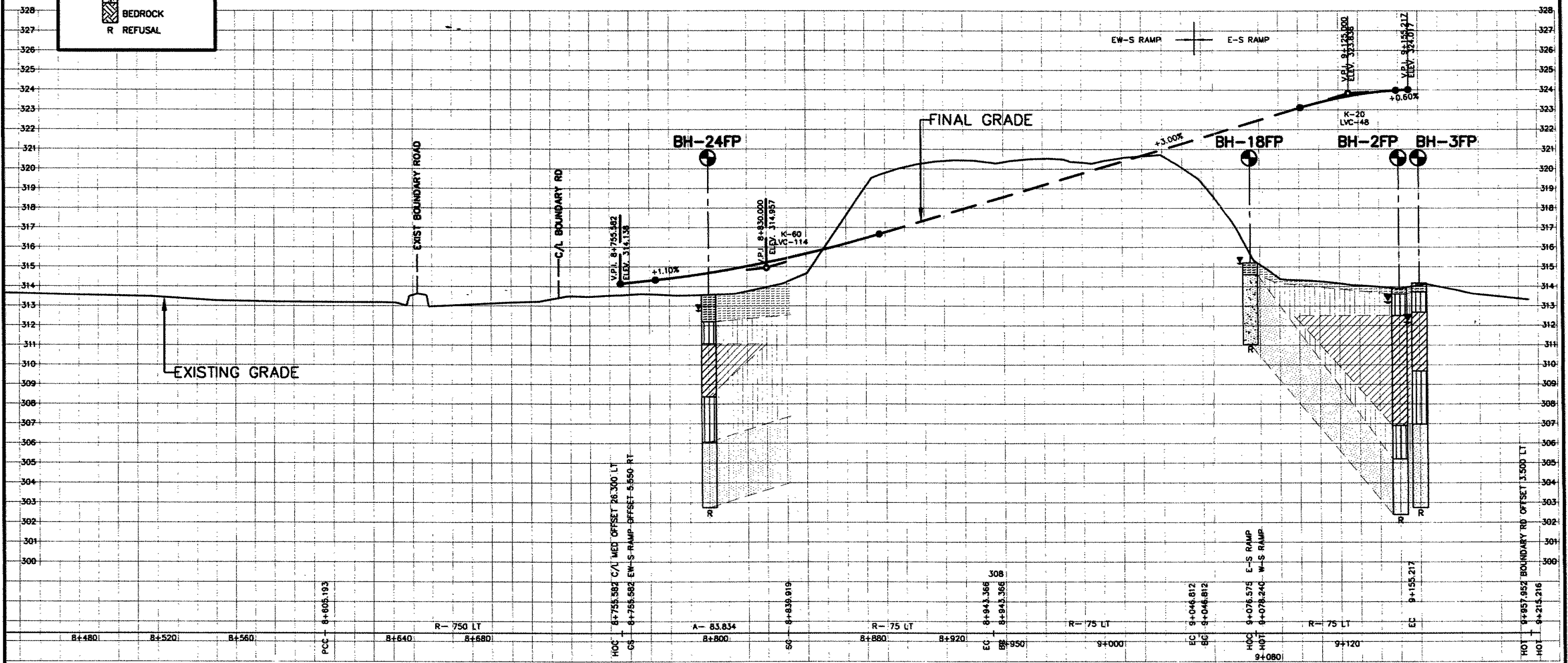
BEDROCK

R REFUSAL

**NOTE:**

The stratigraphic boundaries and soil types have been established at Test Hole locations. Between Test Holes they are assumed and may be subject to error.

EW-S + E-S



EW-S AND E-S RAMP  
8+255.582 = HOC 8+246.611 C/L MED OFFSET 22.500 LT

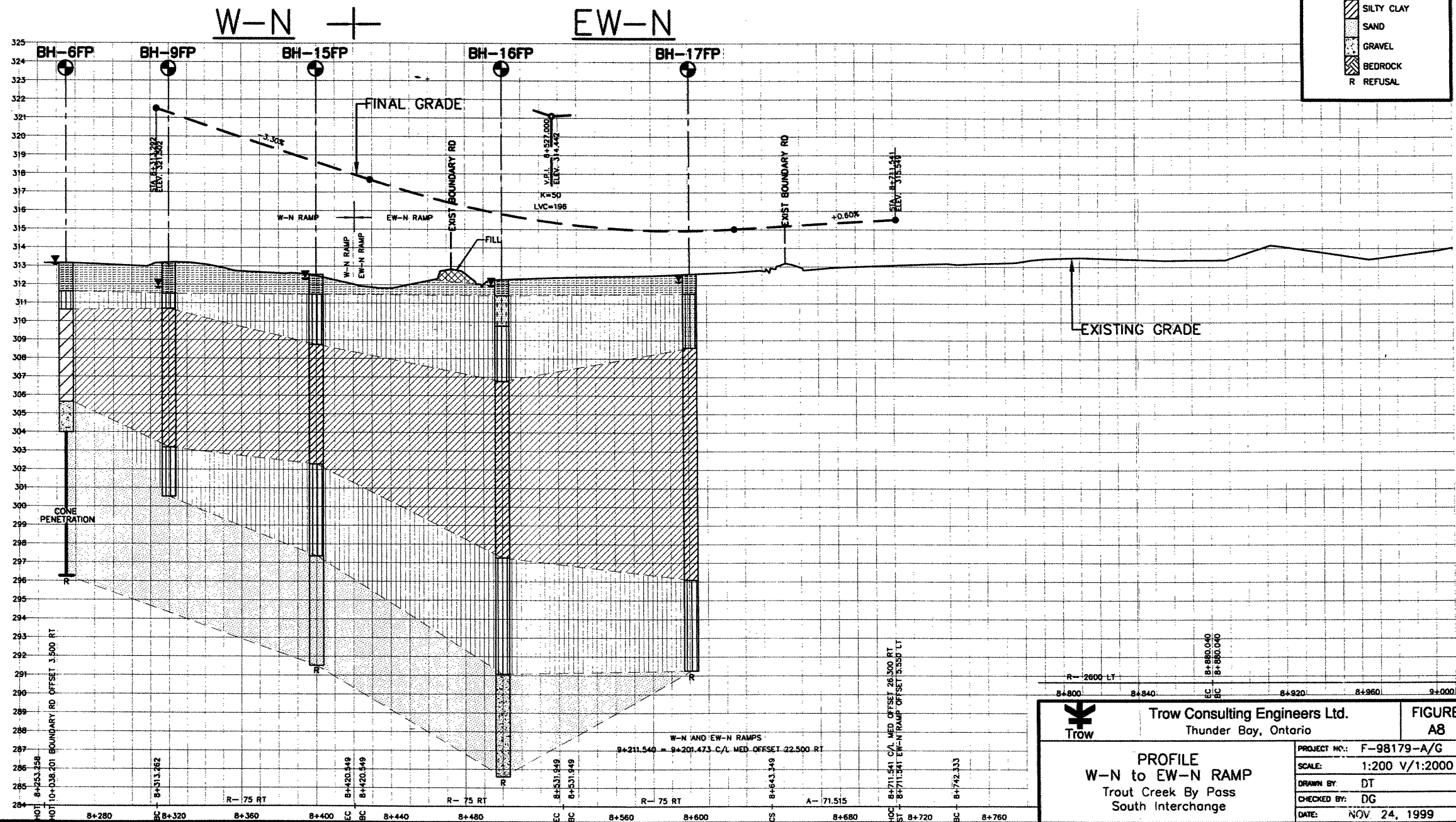
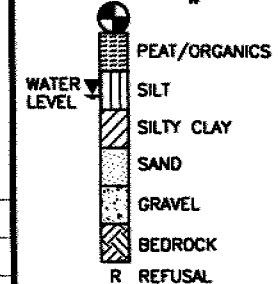
 <b>Trow</b> Trow Consulting Engineers Ltd. Thunder Bay, Ontario	<b>FIGURE A7</b>	
	<b>PROFILE</b>	
	<b>EW-S to E-S RAMP</b>	
	Trout Creek By Pass South Interchange	
	PROJECT NO.: F-98179-A/G	SCALE: 1:200 V/1:2000 H
DRAWN BY: DT	CHECKED BY: DG	
DATE: NOV. 24, 1999		

**NOTE:**

The stratigraphic boundaries and soil types have been established at Test Hole locations. Between Test Holes they are assumed and may be subject to error.

**LEGEND:**

**BOREHOLE #**



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Thunder Bay, Ontario

**FIGURE A8**

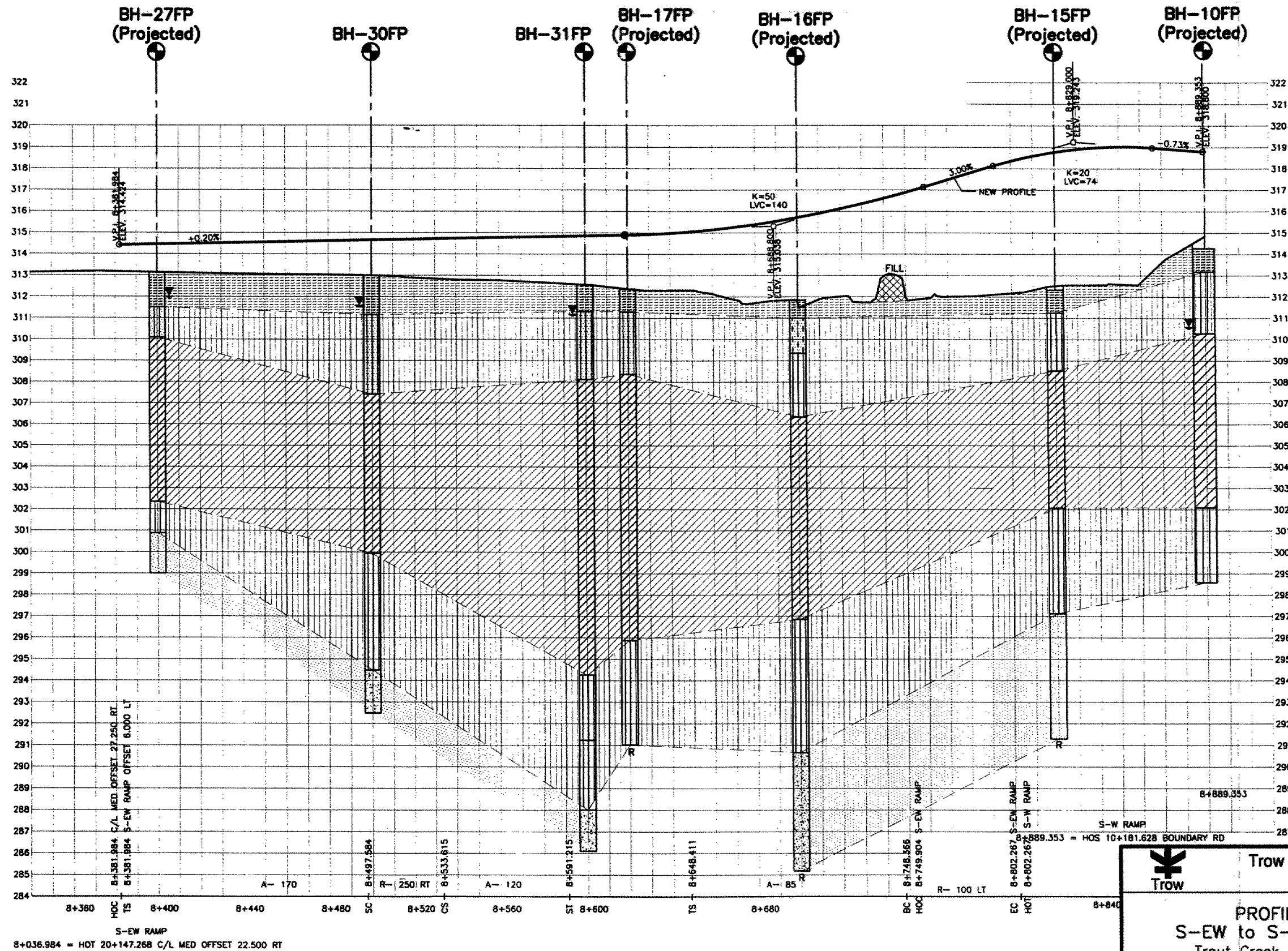
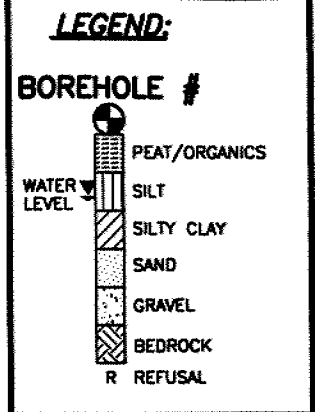
**PROFILE**  
**W-N to EW-N RAMP**  
Trout Creek By Pass  
South Interchange

PROJECT NO.:	F-98179-A/G
SCALE:	1:200 V/1:2000 H
DRAWN BY:	DT
CHECKED BY:	DG
DATE:	NOV. 24, 1999



S-EW

S-W



**NOTE:**

The stratigraphic boundaries and soil types have been established at Test Hole locations. Between Test Holes they are assumed and may be subject to error.



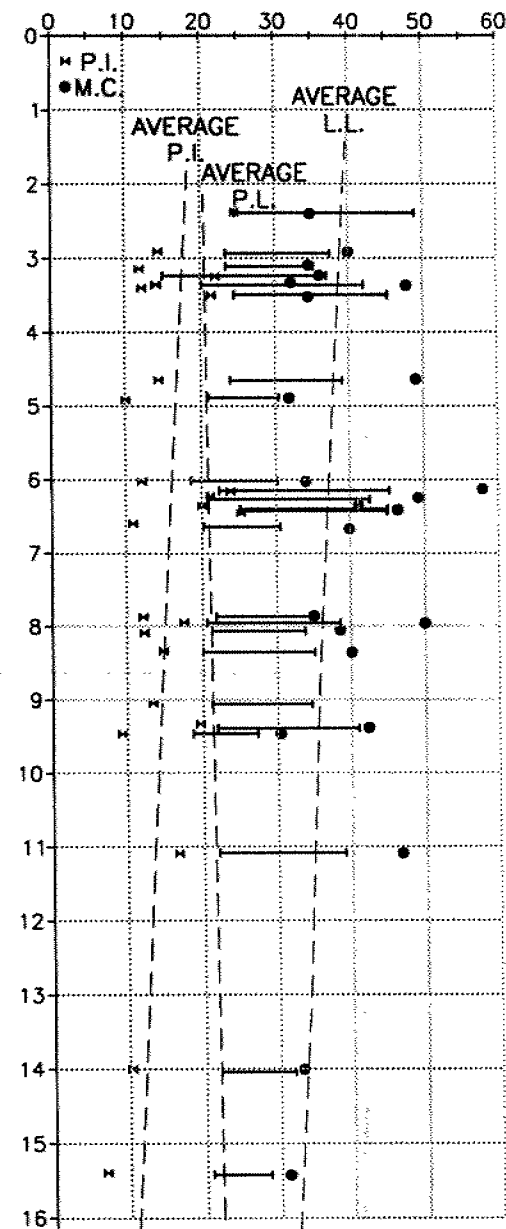
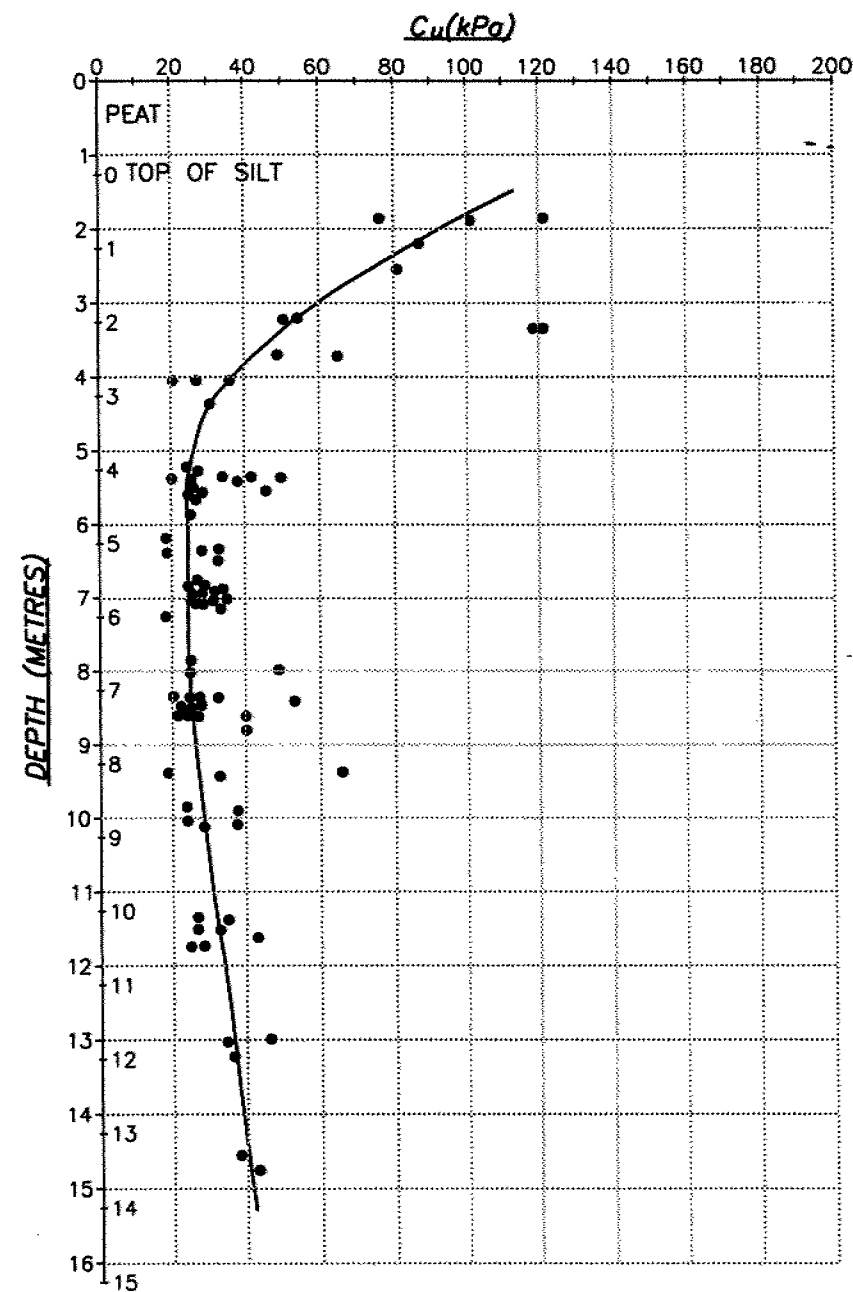
Trow Consulting Engineers Ltd.  
Thunder Bay, Ontario

FIGURE  
A9

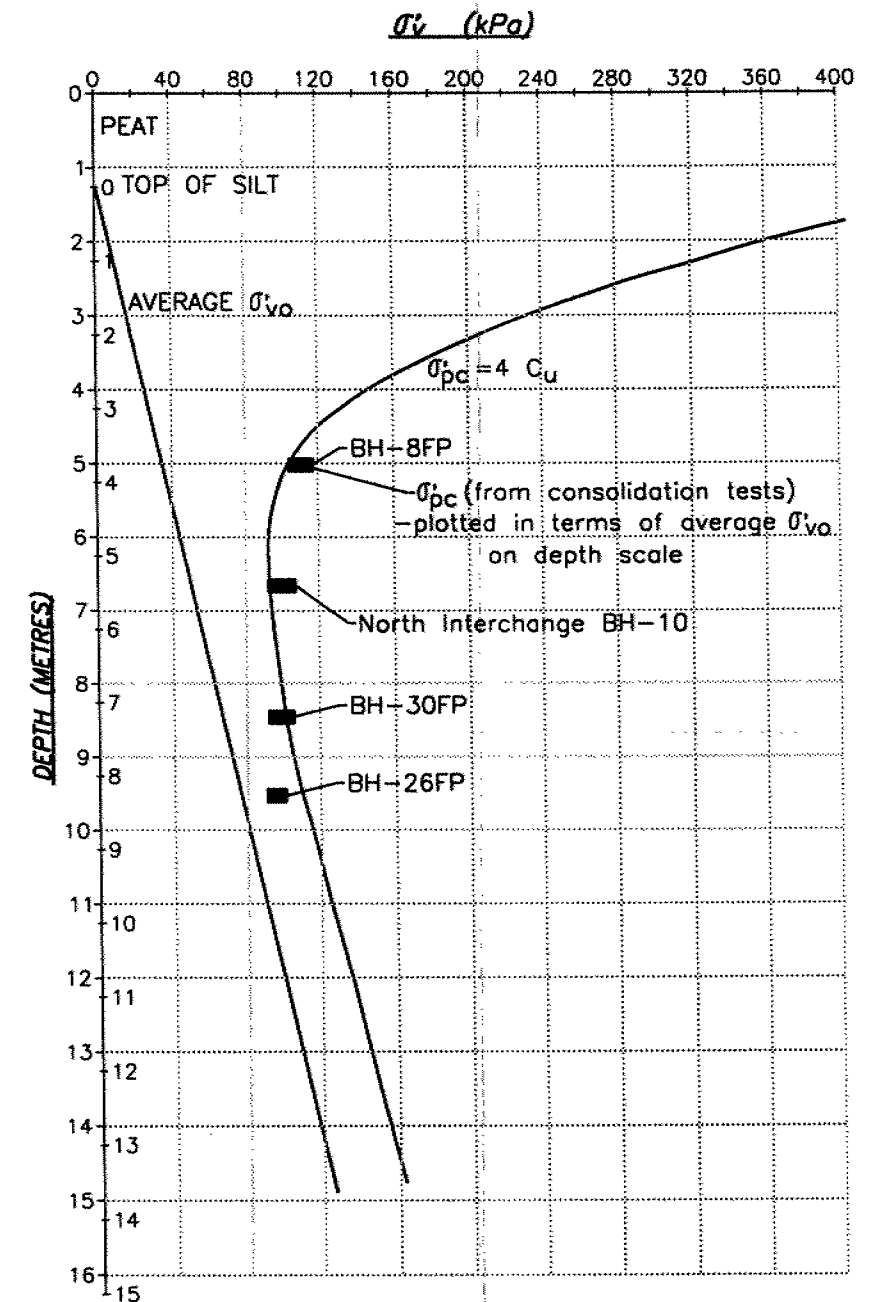
PROFILE  
S-EW to S-W RAMP  
Trout Creek By Pass  
South Interchange


PROJECT NO.:	F-98179-A/G
SCALE:	1:200 V/1:2000 H
DRAWN BY:	DT
CHECKED BY:	DG
DATE:	NOV. 24, 1999

# MOISTURE CONTENT & ATTERBERG LIMITS



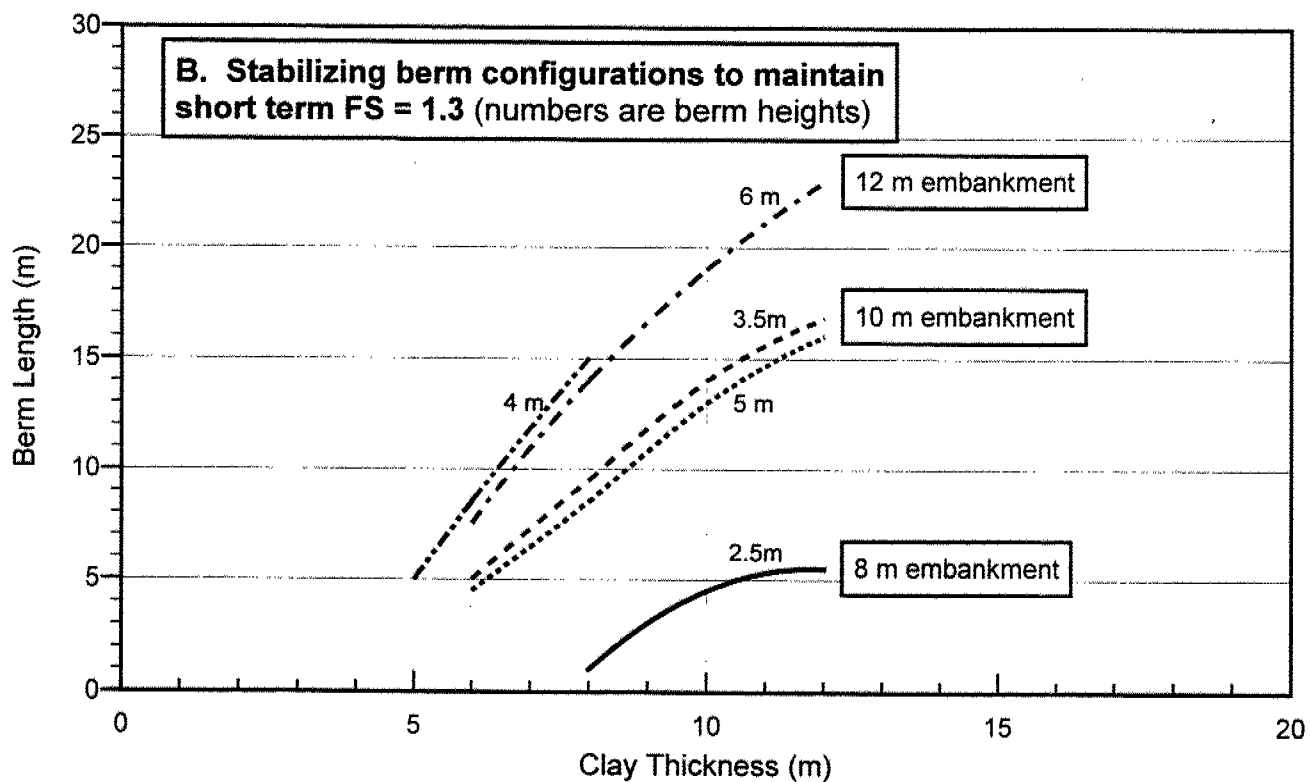
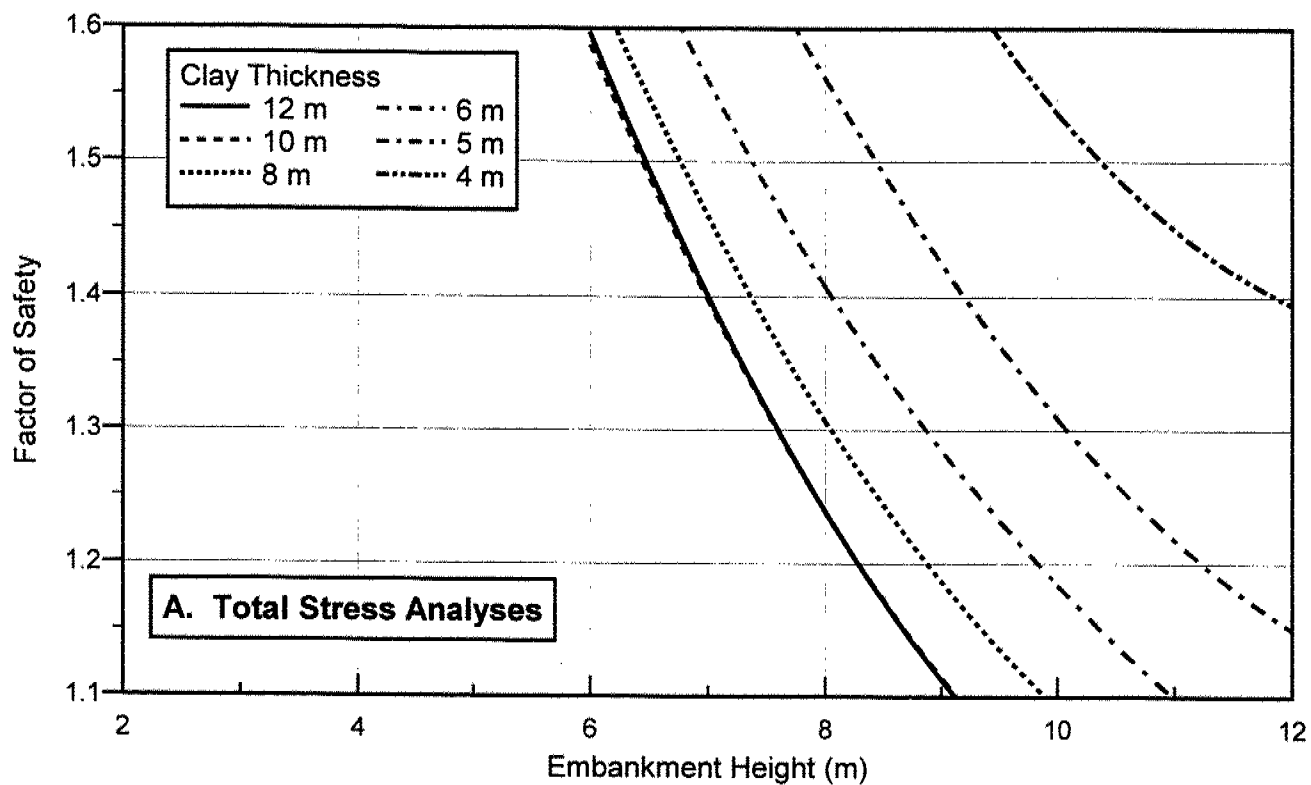
Only moisture contents associated with Atterberg Limits are shown.



 <b>Trow Consulting Engineers Ltd.</b> Thunder Bay, Ontario	<b>FIGURE A10</b>	
	PROJECT NO.: F-98179-A/G	
	SCALE: NOT TO SCALE	
	DRAWN BY: DT	
	CHECKED BY: DG	
DATE: NOV. 24, 1999		

UNDRAINED SHEAR STRENGTH,  
ATTERBERG LIMITS &  
EFFECTIVE STRESS PROFILES  
Trout Creek By Pass  
South Interchange





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### Embankment Stability Analyses Total Stresses

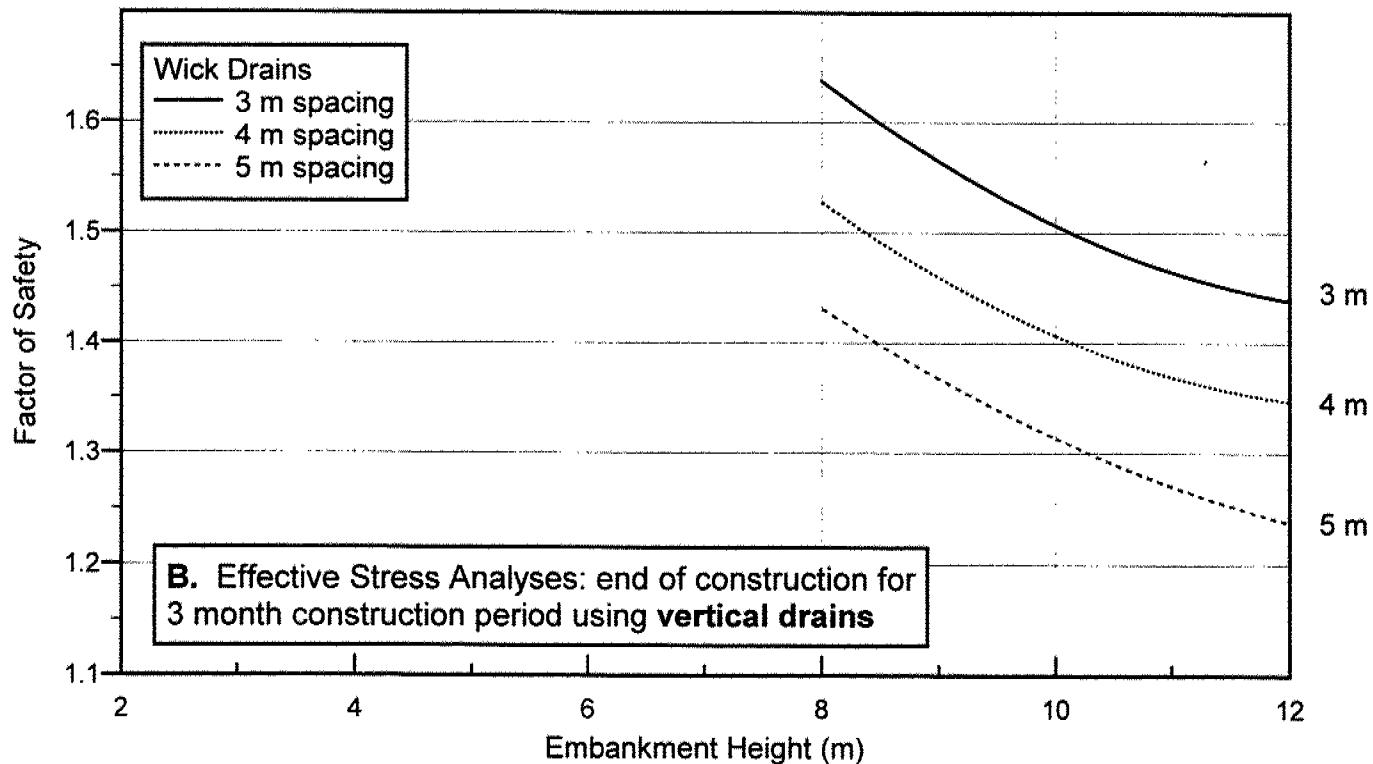
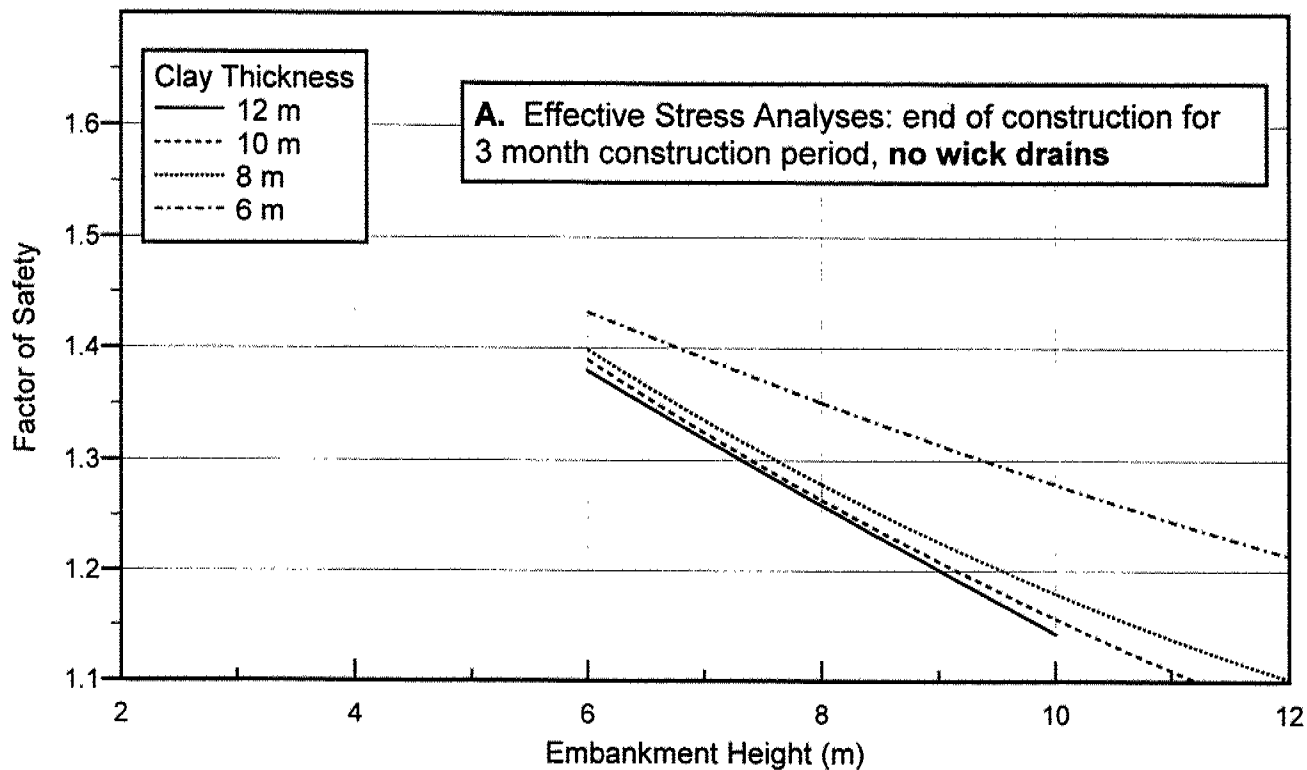
F98179-A/G

Mar 30/99

Marshall Macklin Monaghan

Trout Creek By Pass - South Interchange

Figure A11



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Thunder Bay, Ontario

### Embankment Stability Analyses Effective Stresses

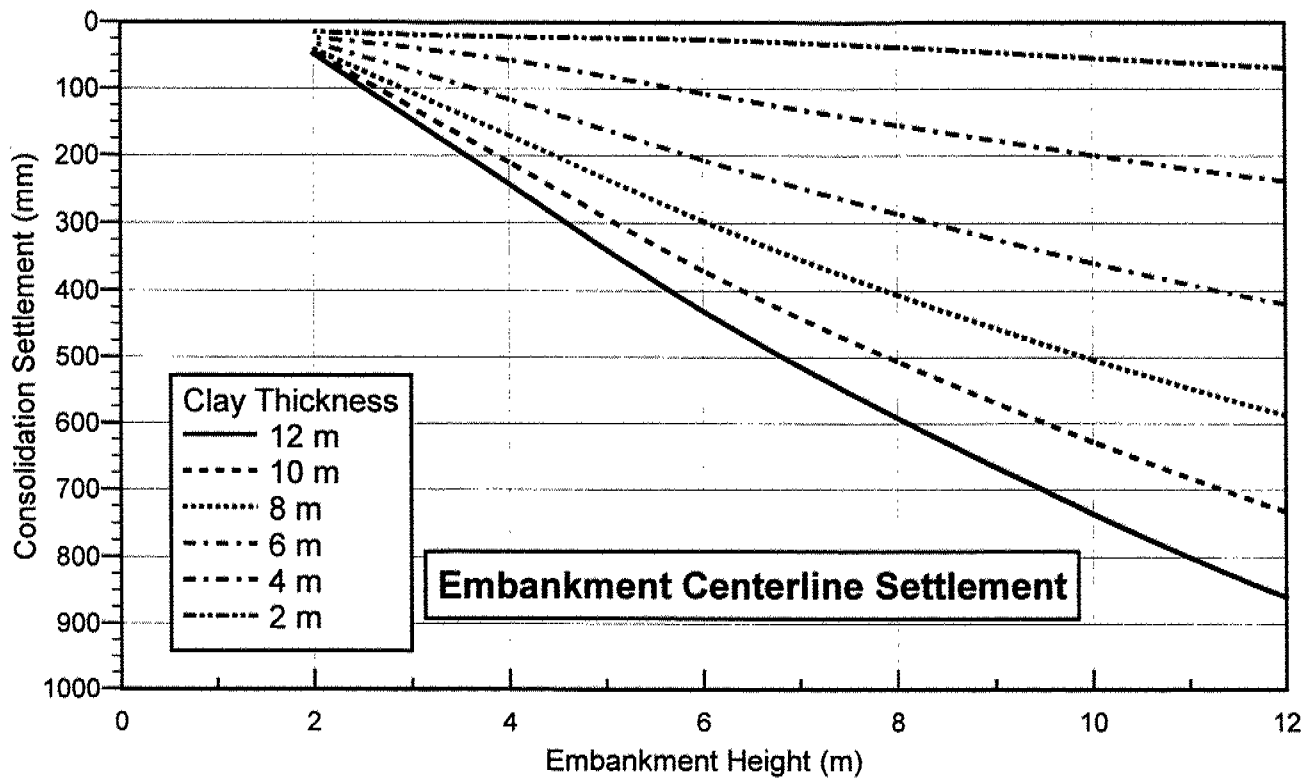
Marshall Macklin Monaghan

Trout Creek By Pass - South Interchange

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Mar 30/99

Figure A12



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**Estimated Embankment Consolidation  
Settlement - Centerline**

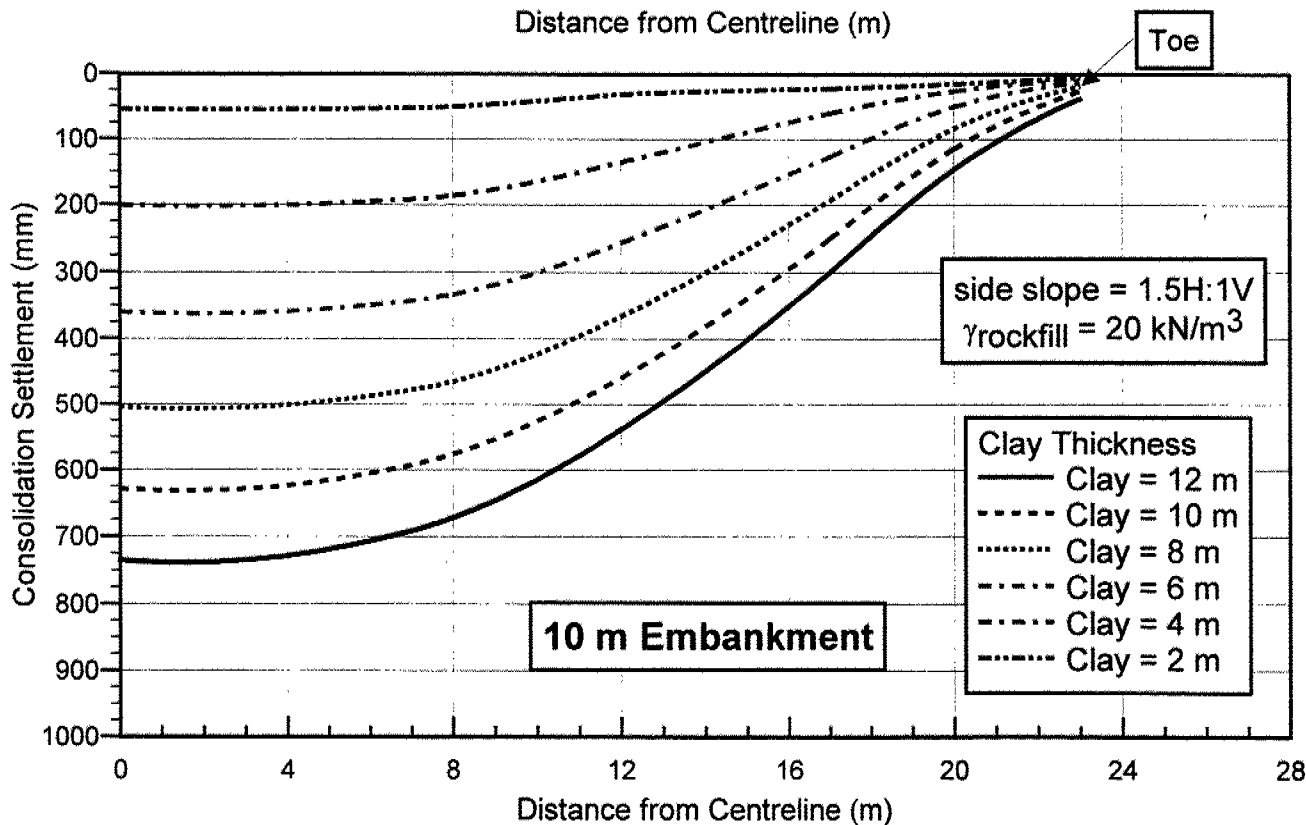
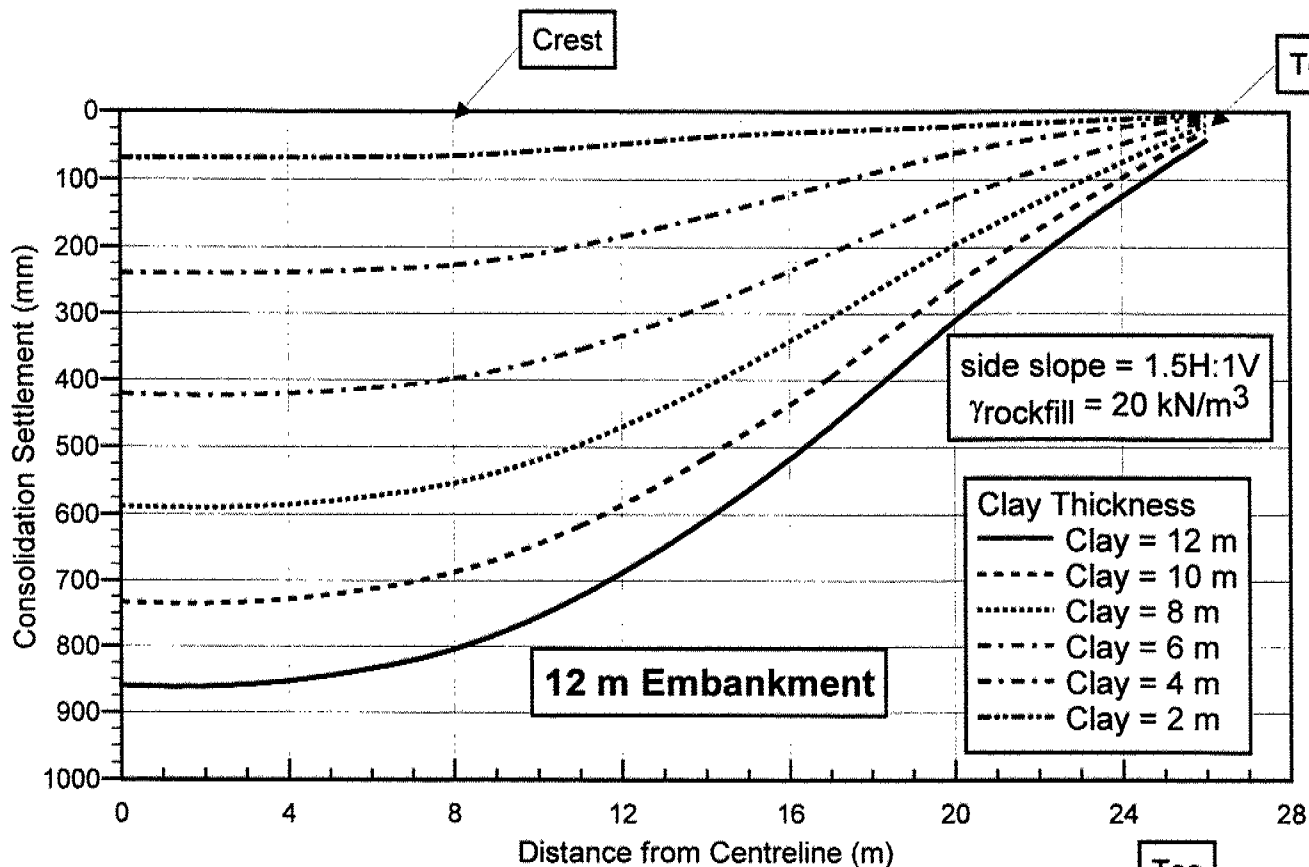
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Sep 23/98

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**Trout Creek By Pass - South Interchange**

Figure A13



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 Thunder Bay, Ontario

**Estimated Embankment Consolidation  
 Settlement Profile: 12 m and 10 m Height**

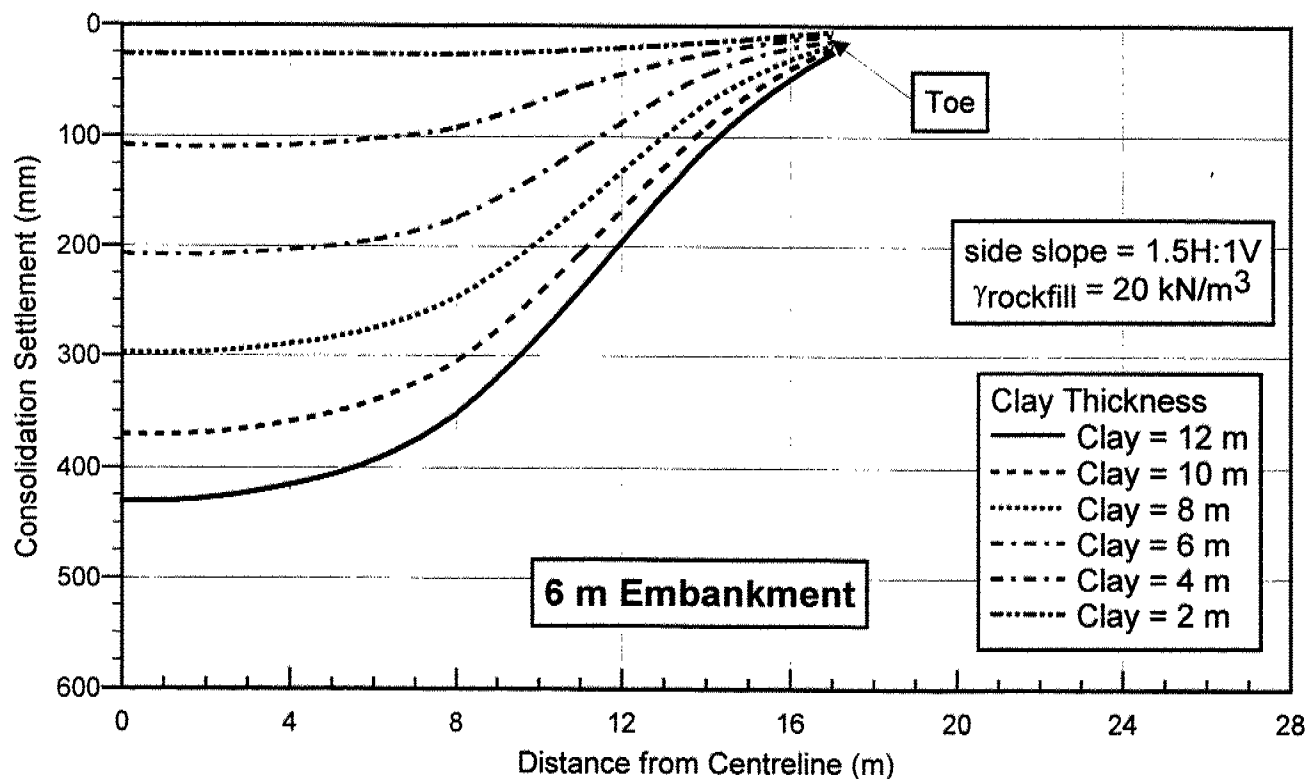
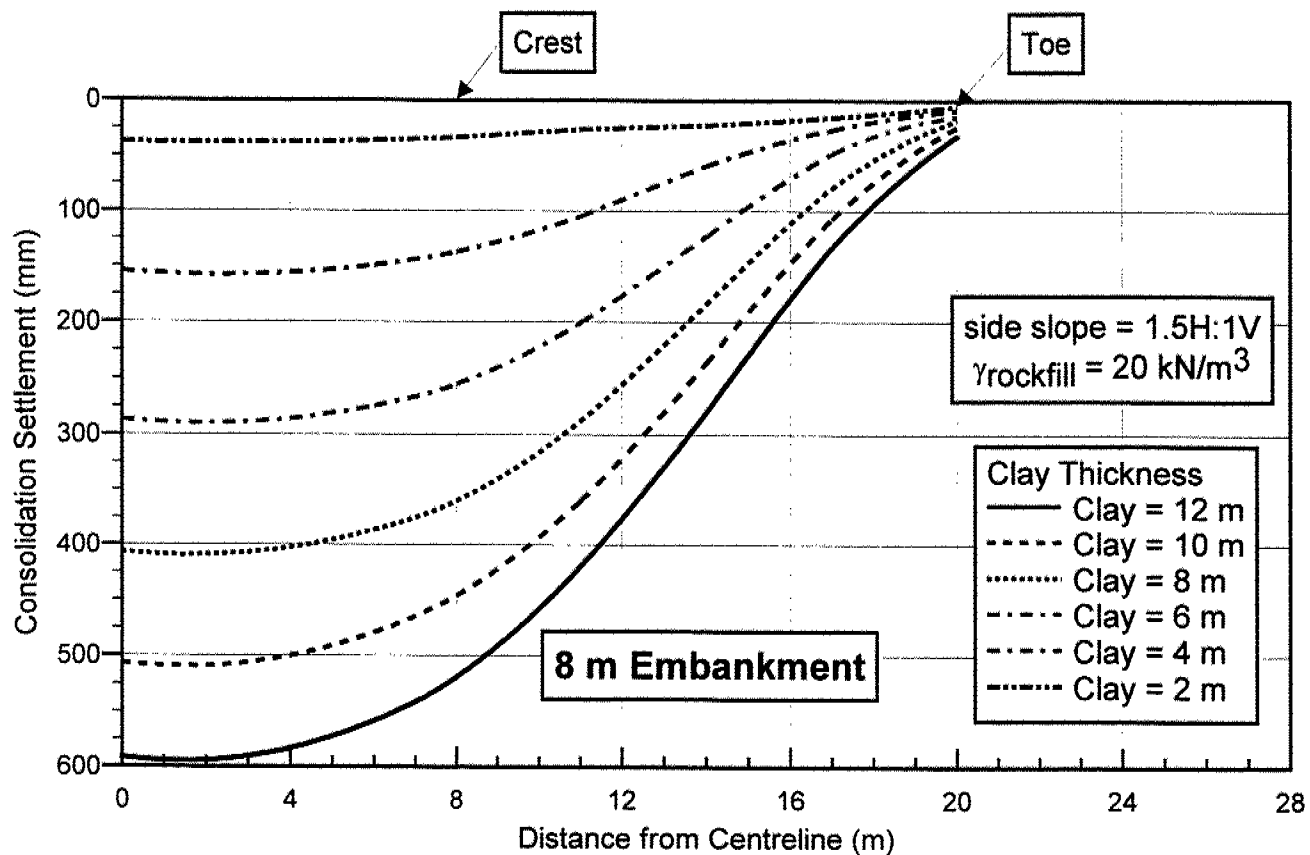
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Sep 23/98

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**Trout Creek By Pass - South Interchange**

Figure A14



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**Estimated Embankment Consolidation  
 Settlement Profile: 8 m and 6 m Height**

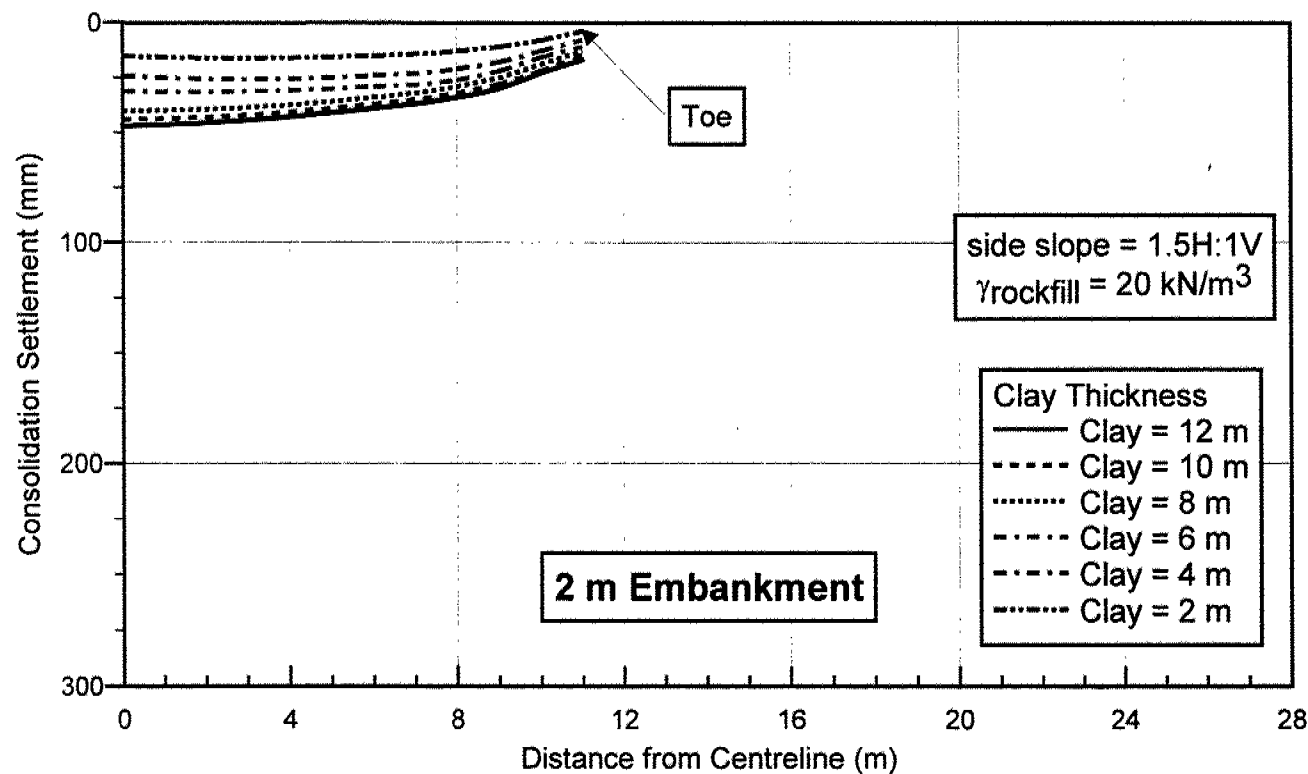
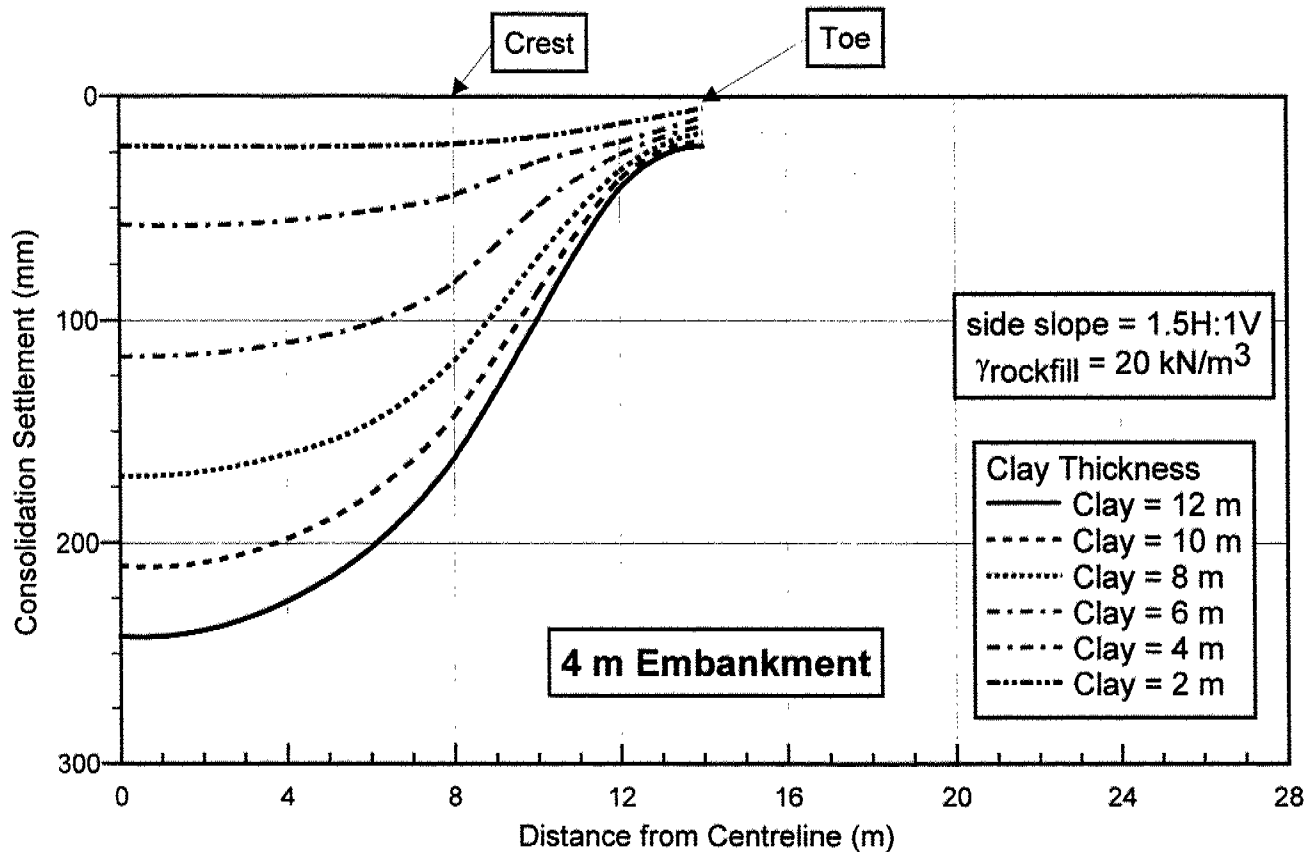
Marshall Macklin Monaghan

**Trout Creek By Pass - South Interchange**

F98179-A/G

Sep 23/98

Figure A15



Trow Consulting Engineers Ltd.  
 Thunder Bay, Ontario

**Estimated Embankment Consolidation  
 Settlement Profile: 4 m and 2 m Height**

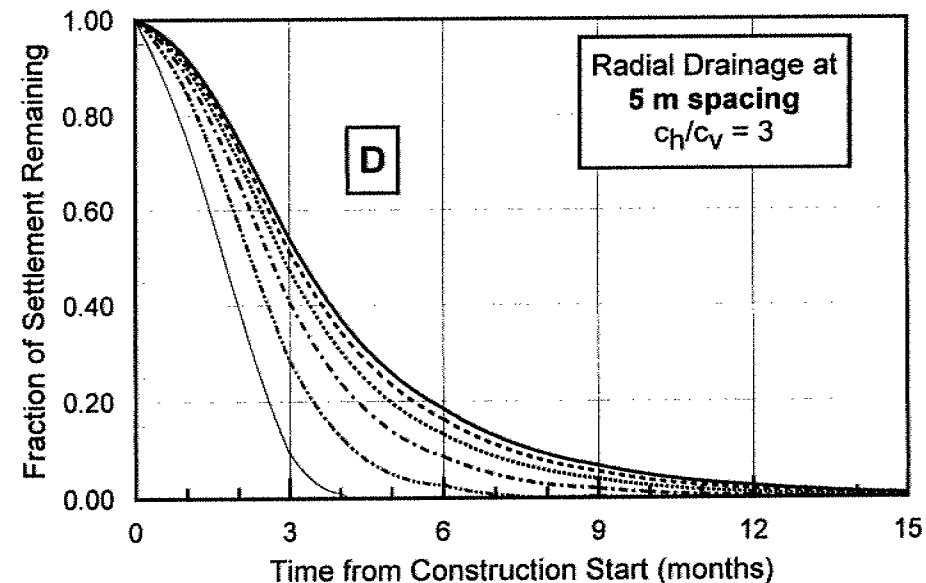
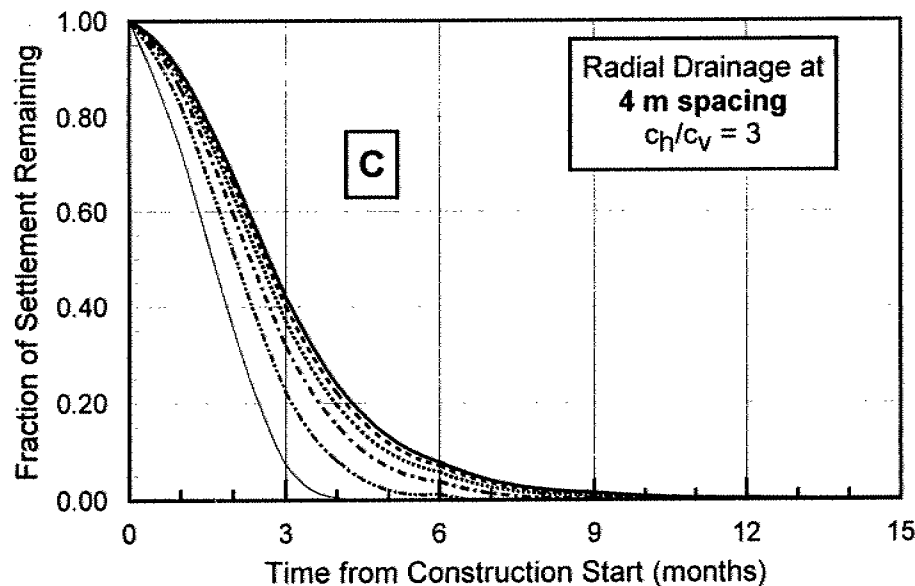
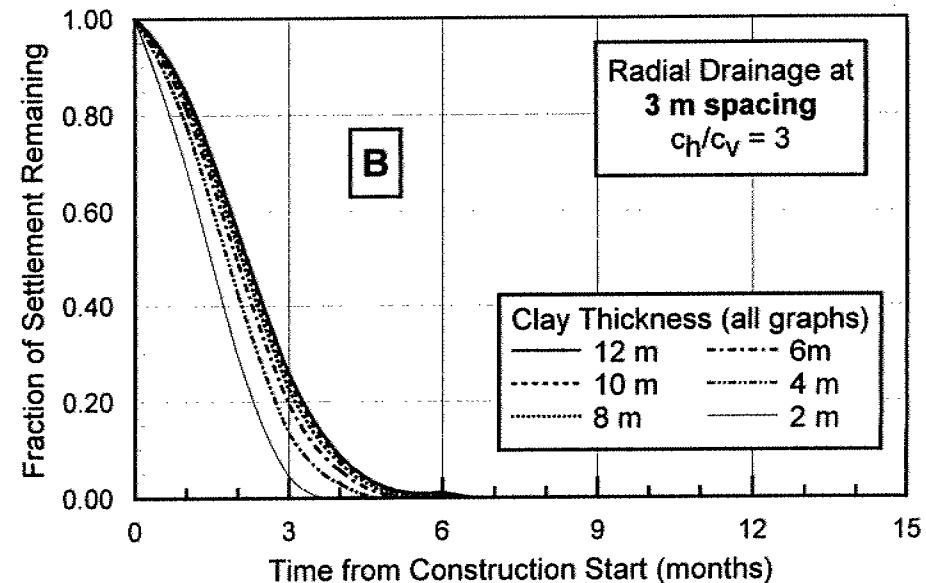
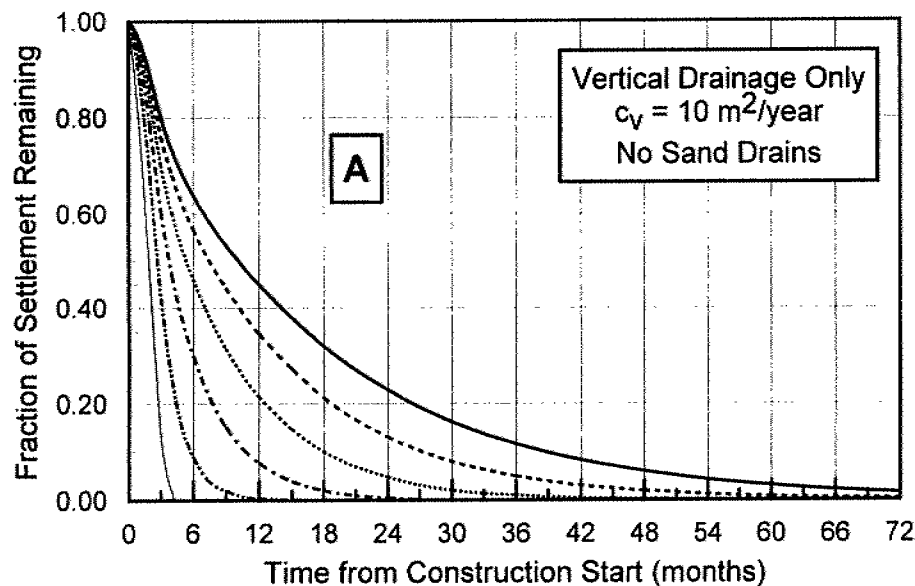
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**Trout Creek By Pass - South Interchange**

Figure A16



Charts based on 3 month (steady loading) construction period



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# Estimated Rate of Consolidation Settlement Various Clay Thickness and Drainage Provisions

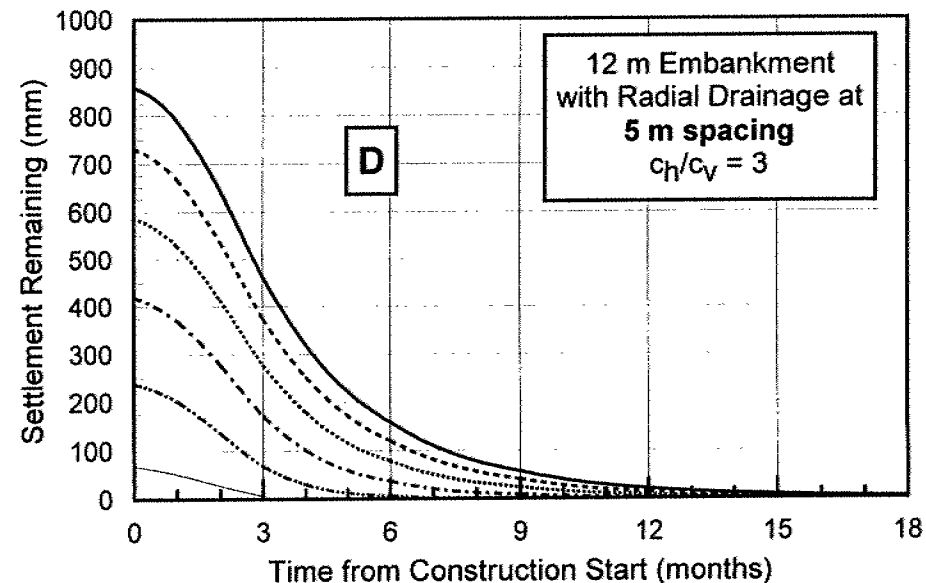
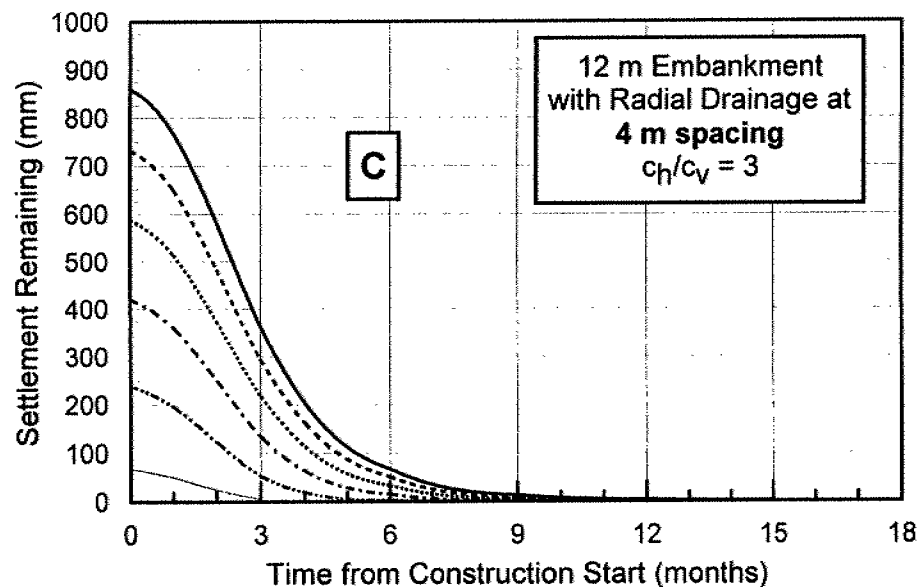
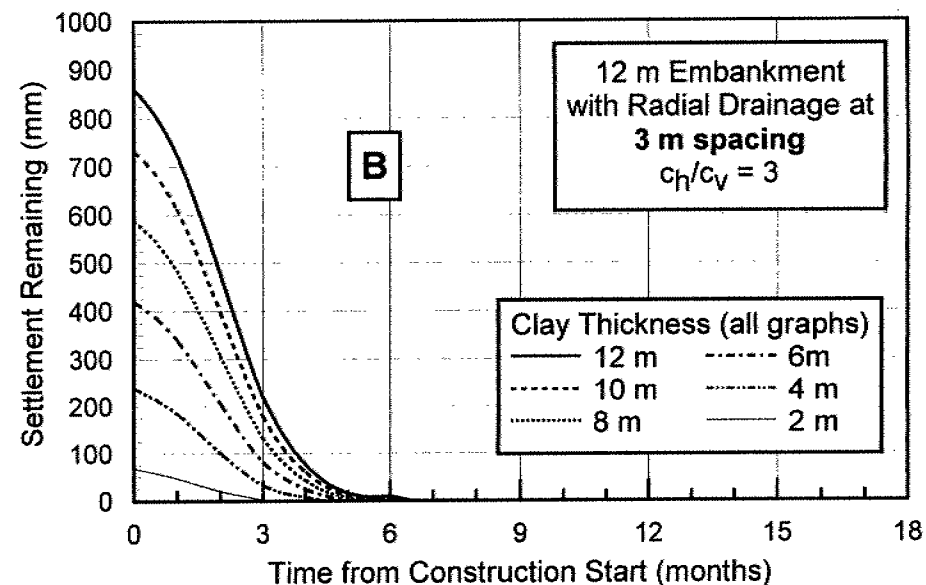
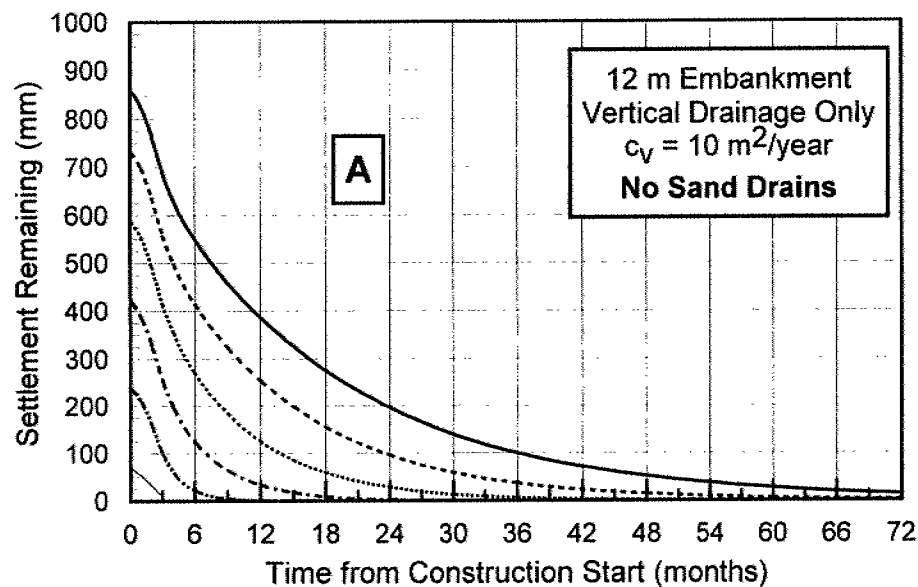
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Trout Creek By Pass - South Interchange

Figure A17



Charts based on 3 month (steady loading) construction period



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Thunder Bay, Ontario

### Estimated Rate of Centerline Consolidation Settlement 12 m High Embankment

F98179-A/G

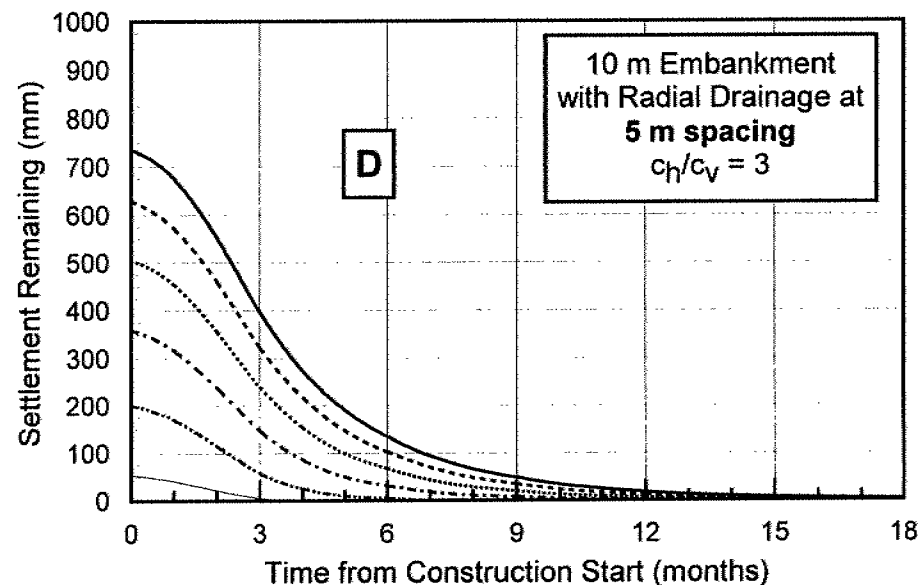
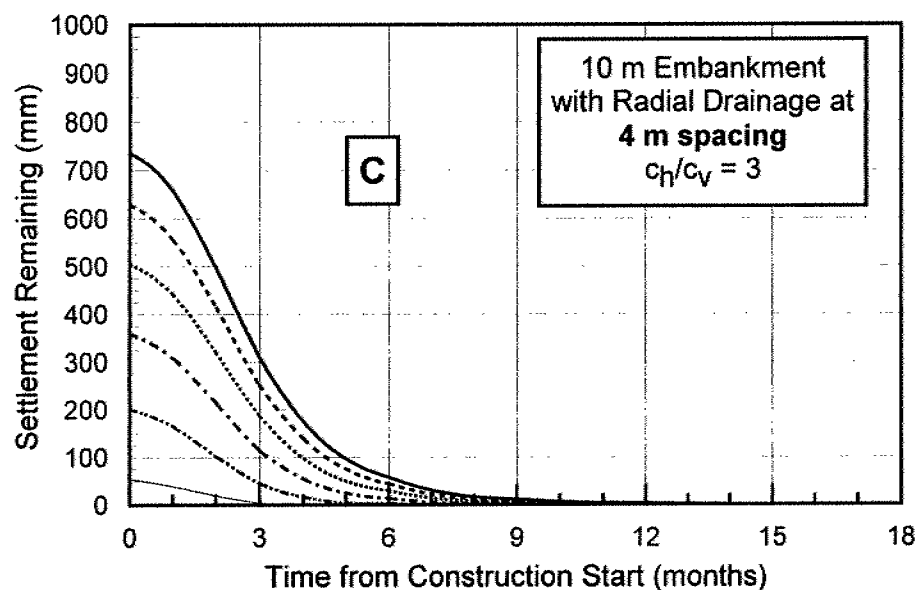
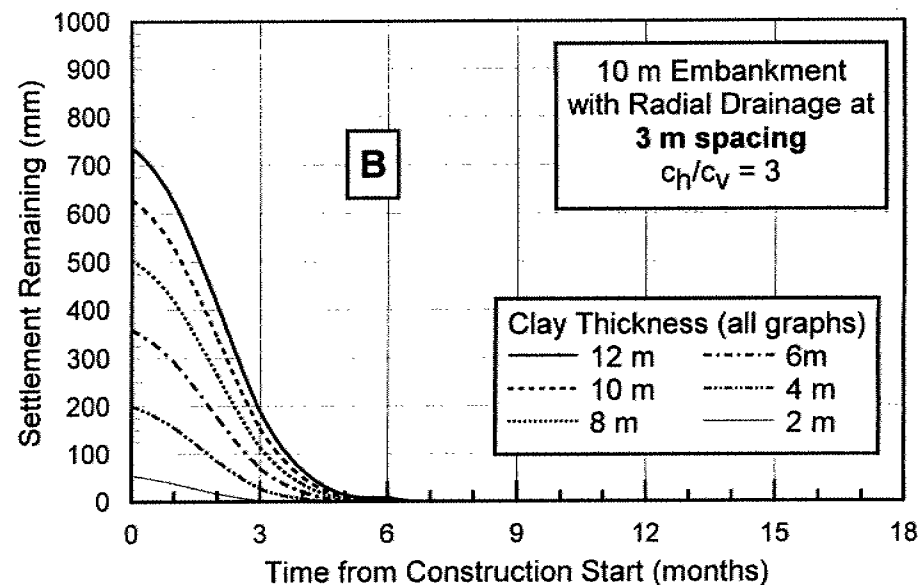
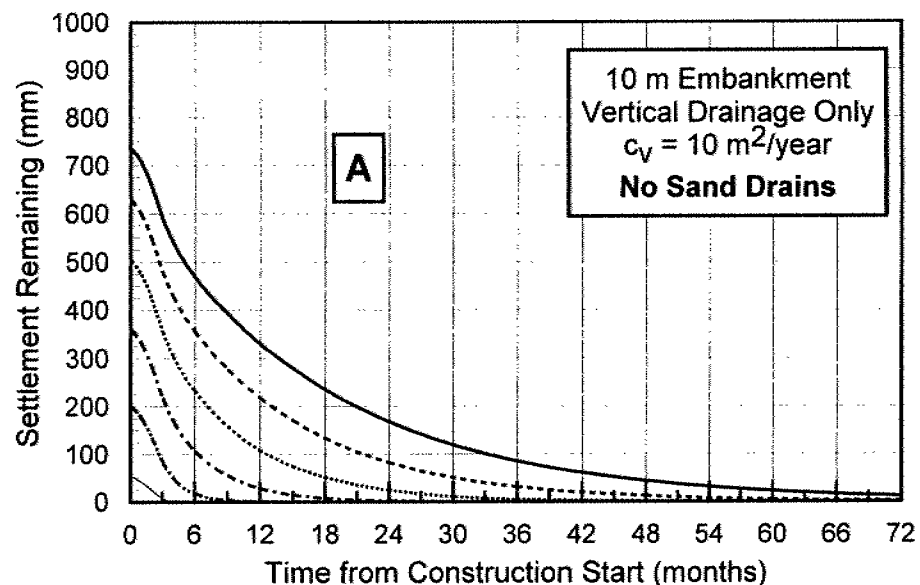
Dec 3/98

Marshall Macklin Monaghan

Trout Creek By Pass - South Interchange

Figure A18





Charts based on 3 month (steady loading) construction period



Trow Consulting Engineers Ltd.  
Thunder Bay, Ontario

Estimated Rate of Centerline Consolidation Settlement  
10 m High Embankment

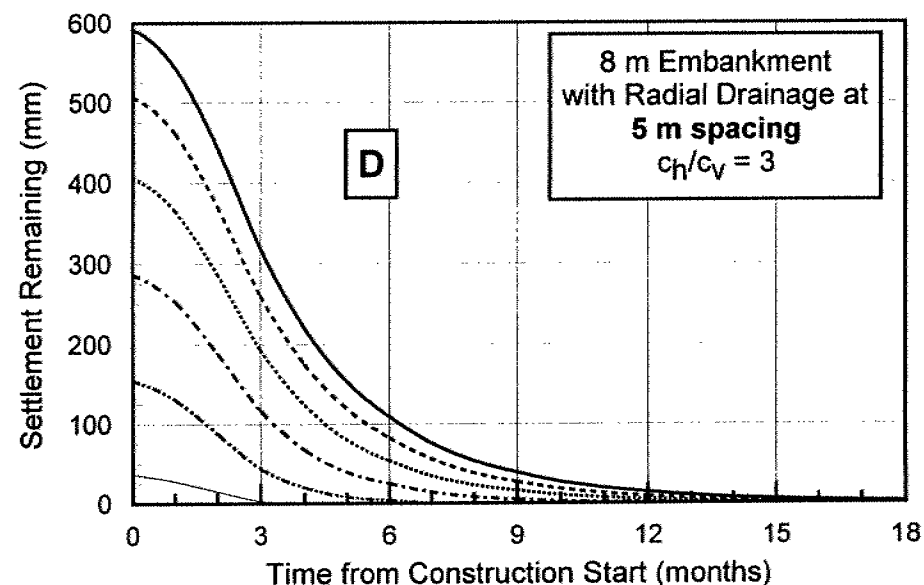
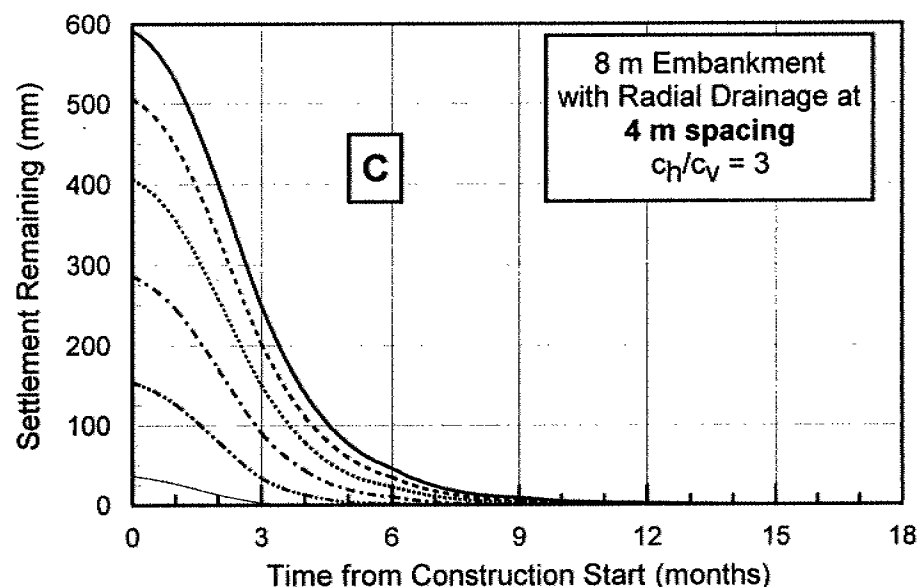
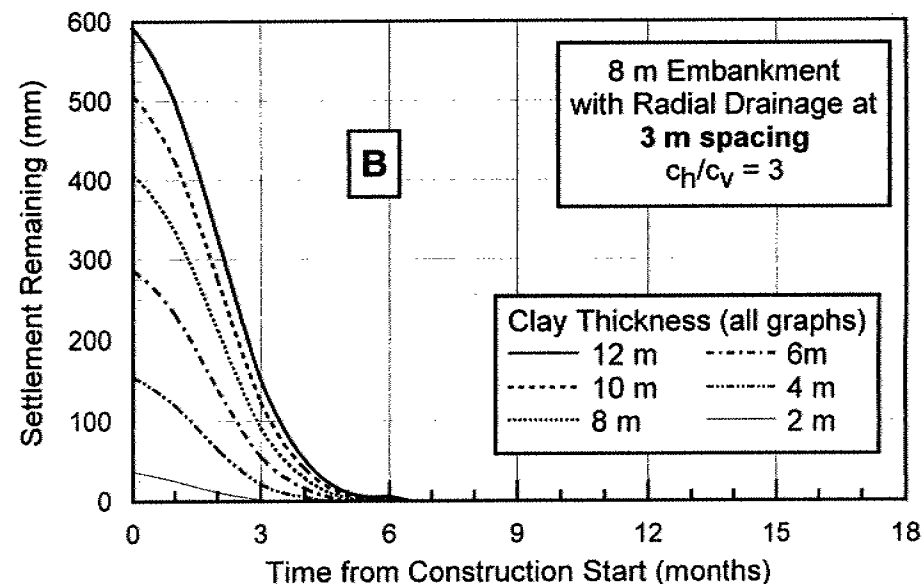
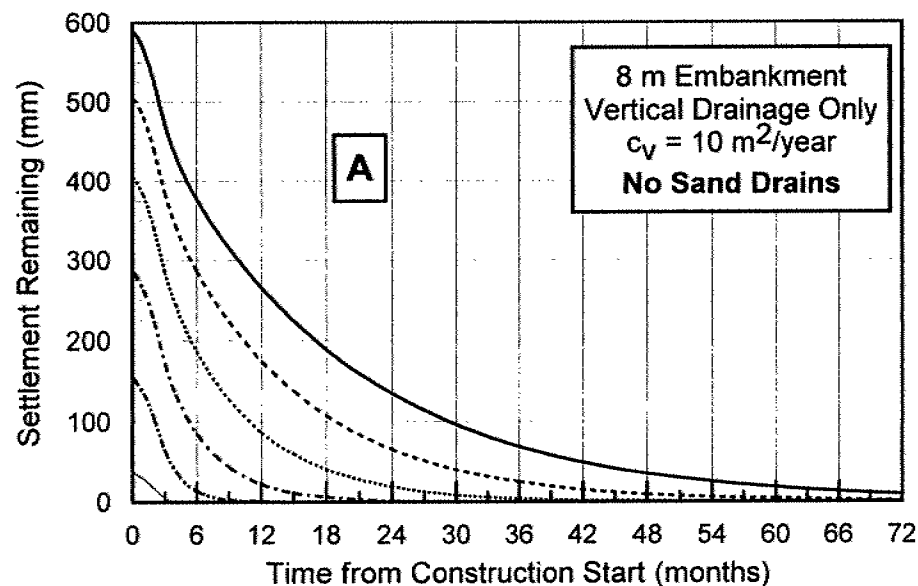
F98179-A/G

Dec 3/98

Marshall Macklin Monaghan

Trout Creek By Pass - South Interchange

Figure A19



Charts based on 3 month (steady loading) construction period



Trow Consulting Engineers Ltd.  
Thunder Bay, Ontario

Estimated Rate of Centerline Consolidation Settlement  
8 m High Embankment

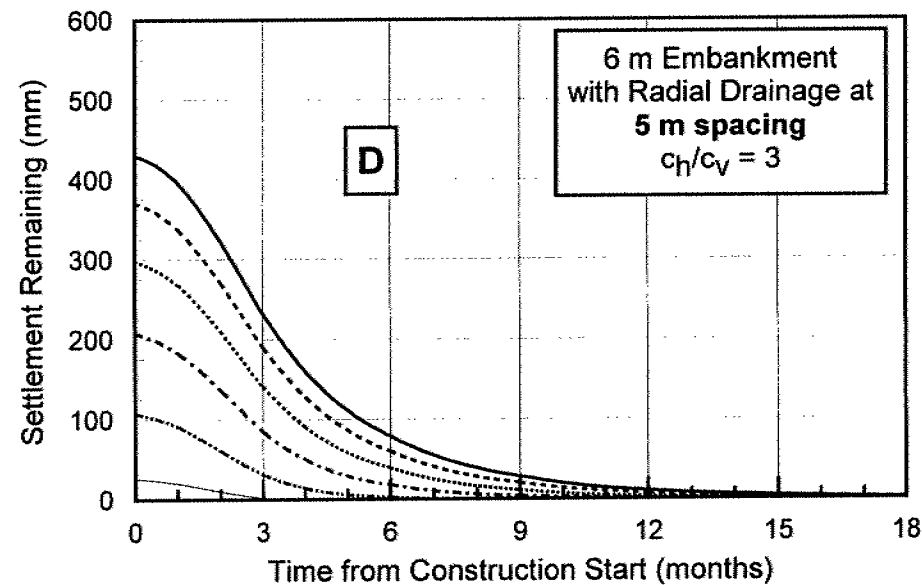
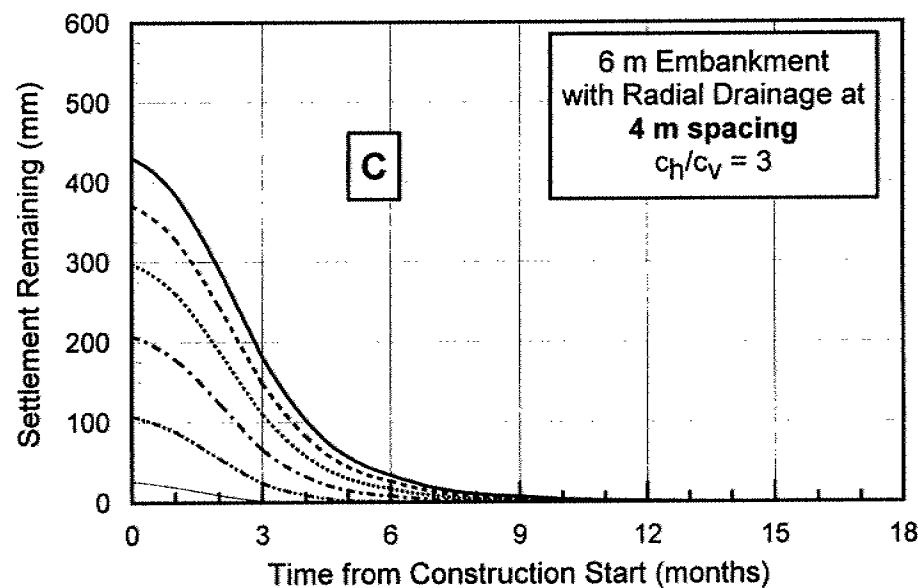
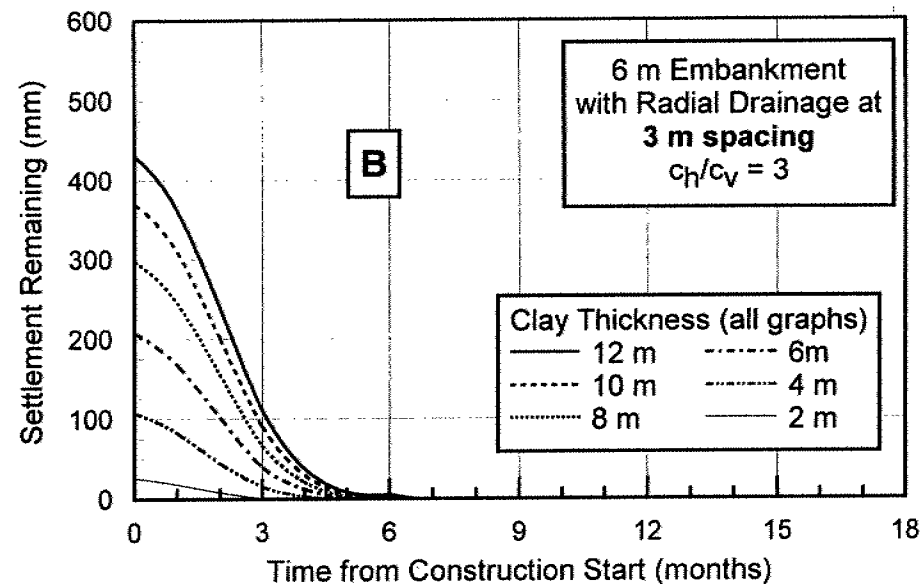
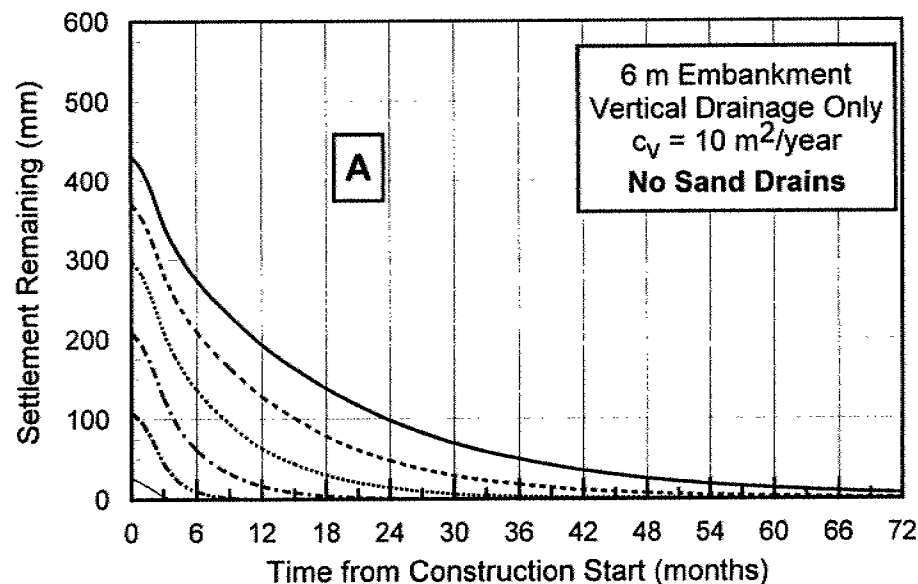
F98179-A/G

Dec 3/98

Marshall Macklin Monaghan

Trout Creek By Pass - South Interchange

Figure A20



Charts based on 3 month (steady loading) construction period



Trow Consulting Engineers Ltd.  
Thunder Bay, Ontario

### Estimated Rate of Centerline Consolidation Settlement 6 m High Embankment

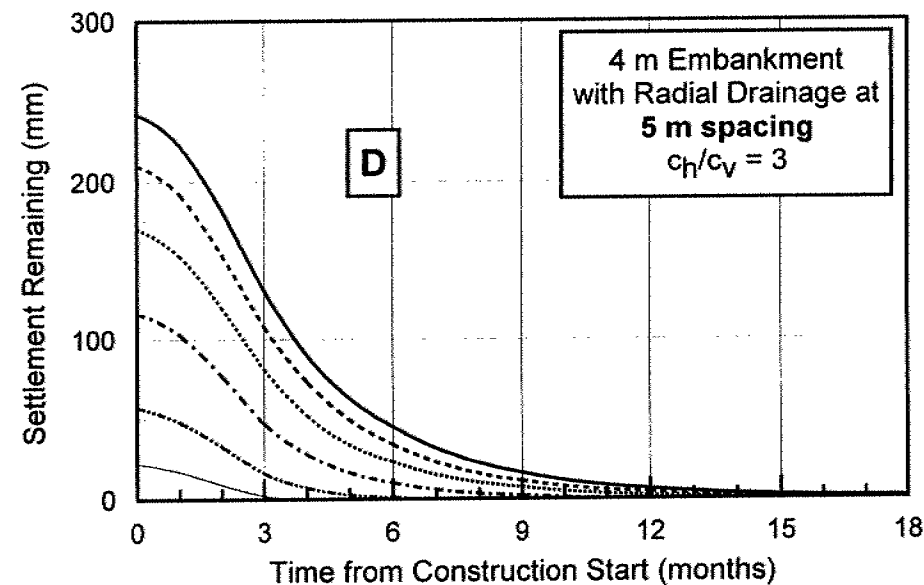
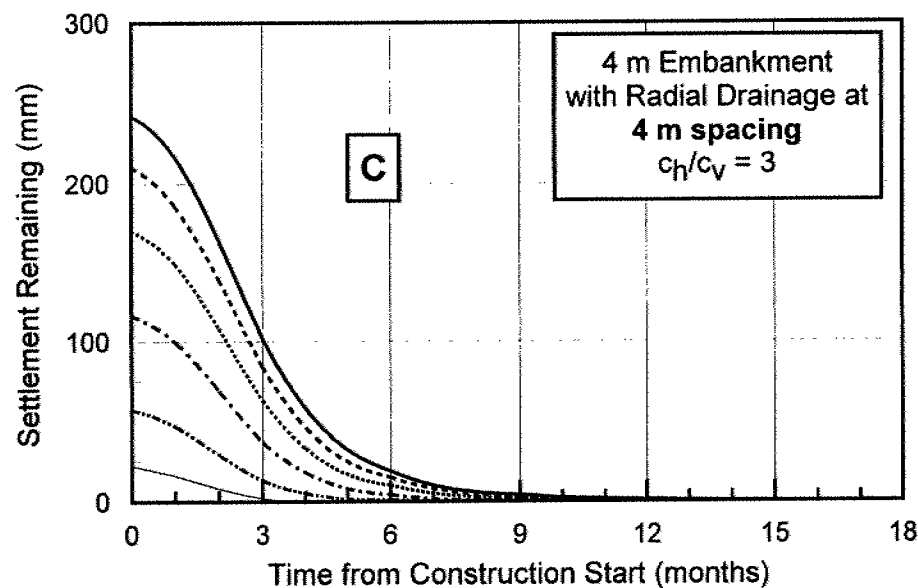
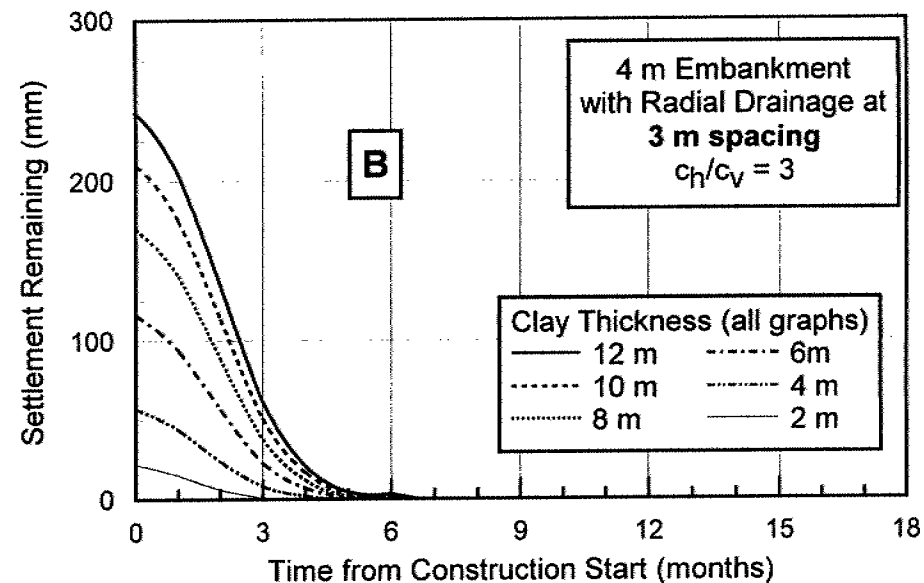
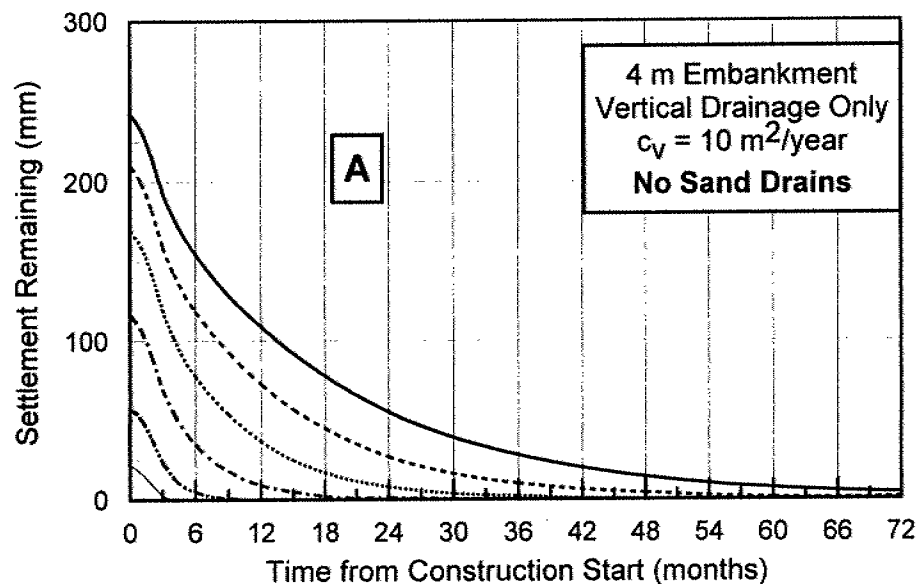
F98179-A/G

Dec 3/98

Marshall Macklin Monaghan

Trout Creek By Pass - South Interchange

Figure A21



Charts based on 3 month (steady loading) construction period



Trow Consulting Engineers Ltd.  
Thunder Bay, Ontario

Estimated Rate of Centerline Consolidation Settlement  
4 m High Embankment

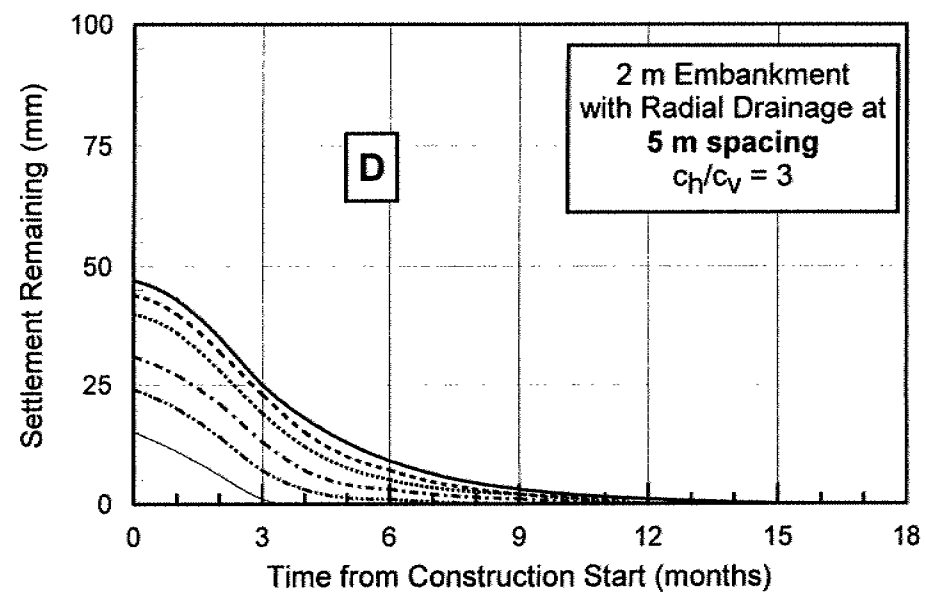
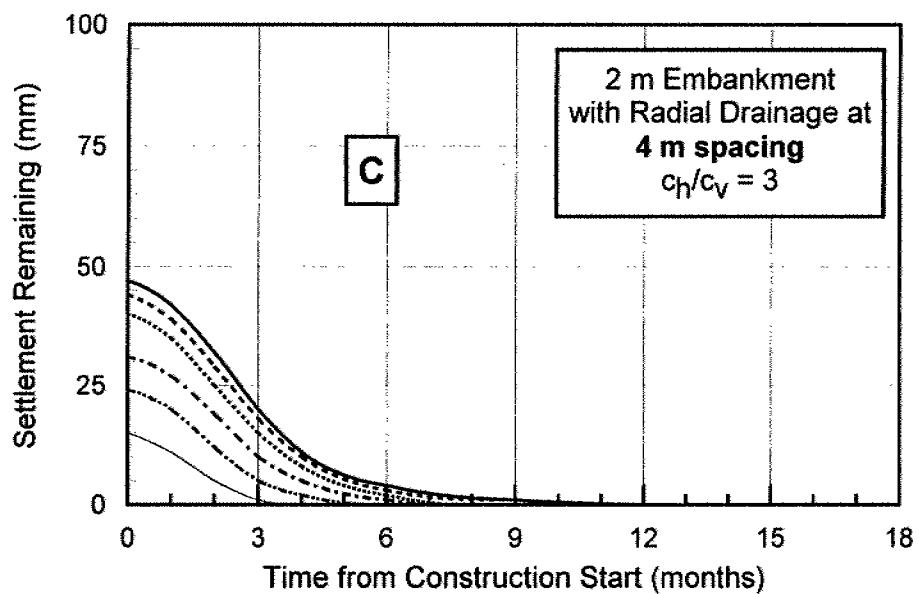
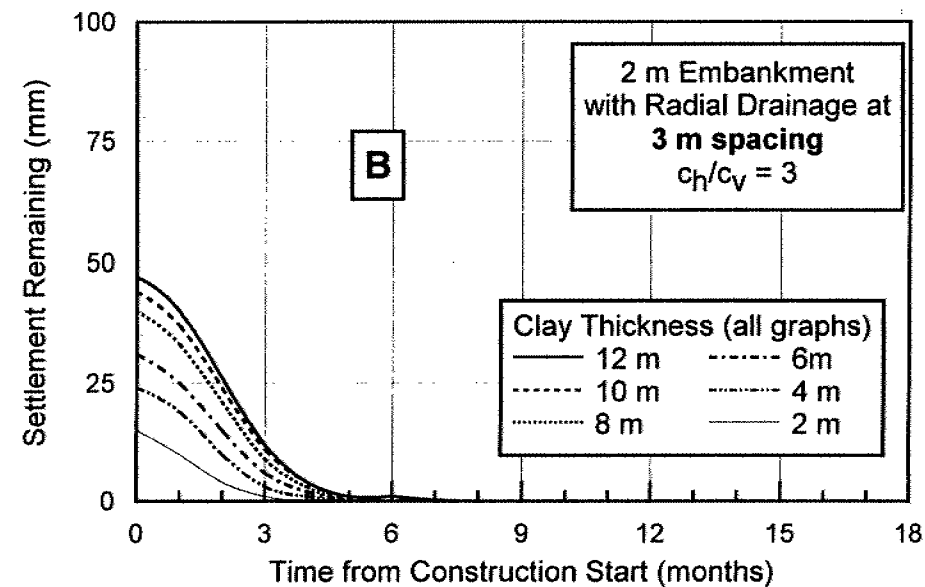
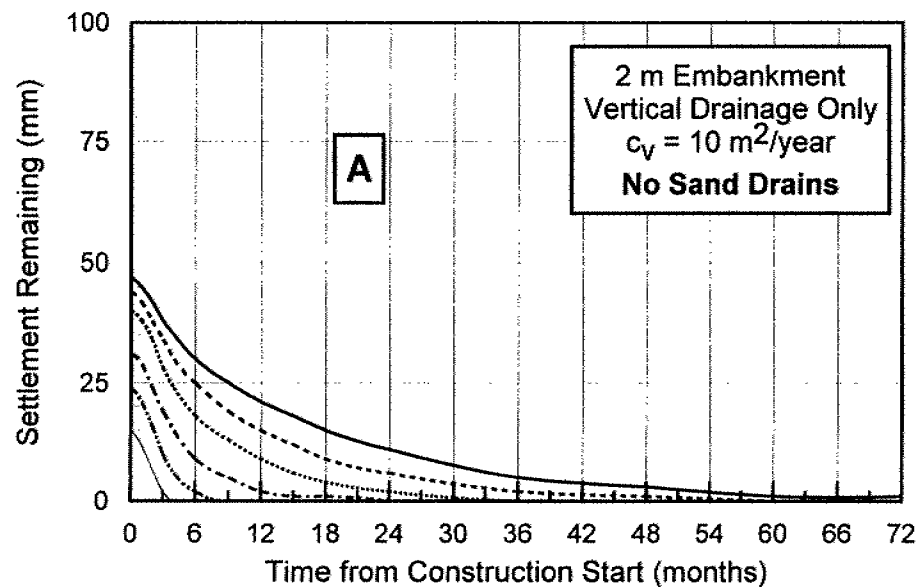
F98179-A/G

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Trout Creek By Pass - South Interchange

Figure A22



Charts based on 3 month (steady loading) construction period



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Thunder Bay, Ontario

### Estimated Rate of Centerline Consolidation Settlement 2 m High Embankment

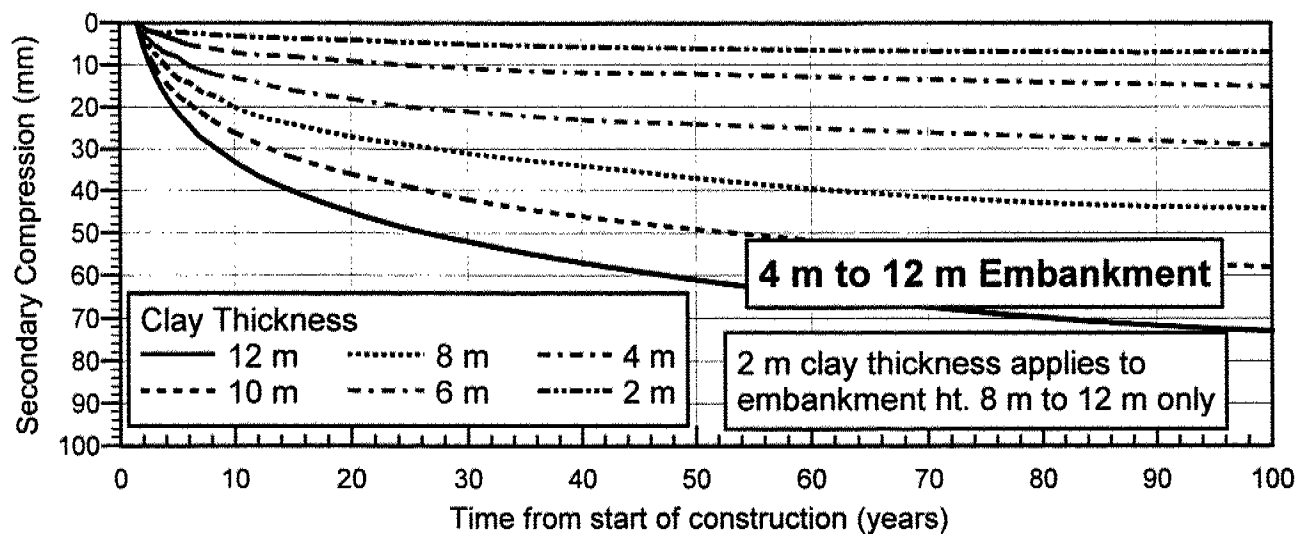
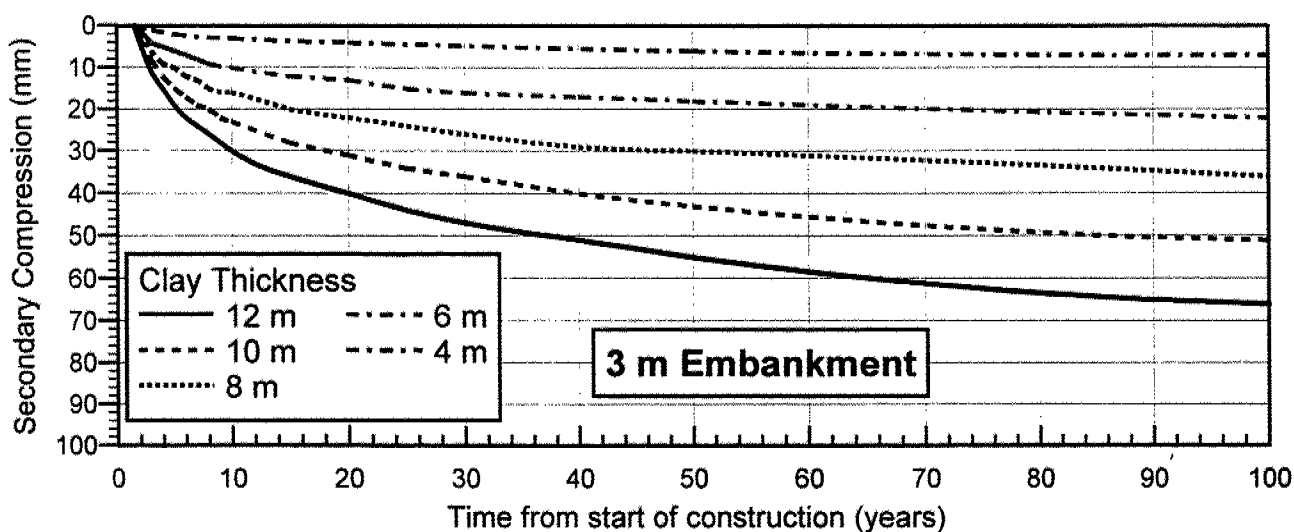
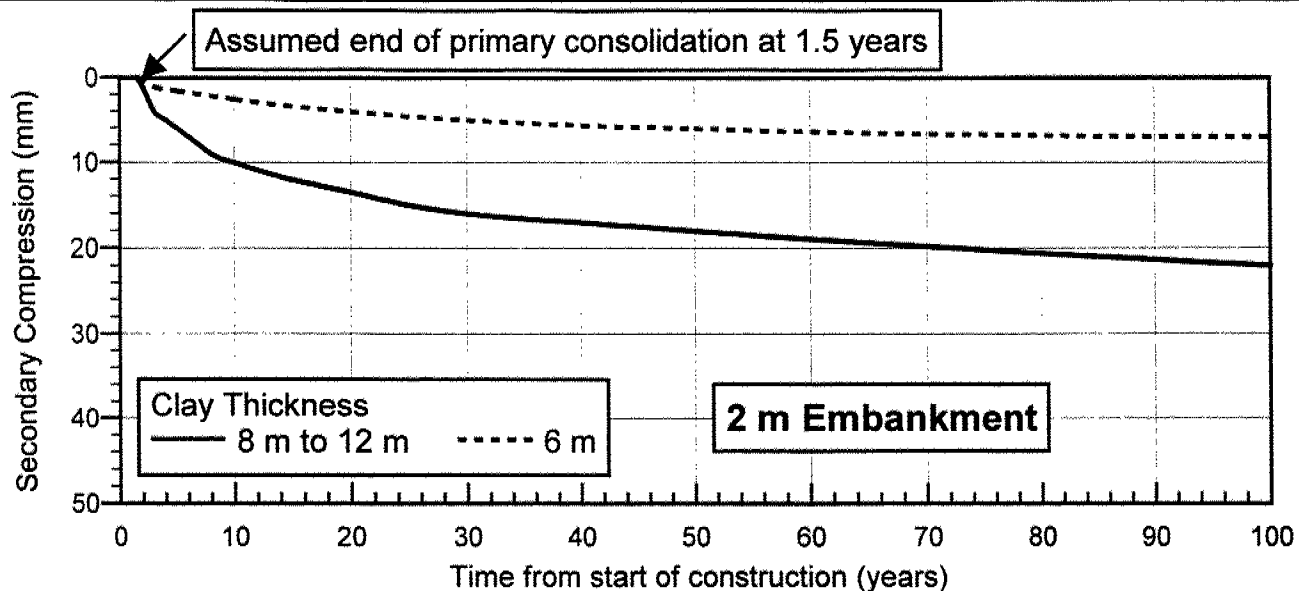
F98179-A/G

Dec 3/98

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Trout Creek By Pass - South Interchange

Figure A23



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Thunder Bay, Ontario

Marshall Macklin Monaghan

### Estimated Embankment Settlement Due to Secondary Compression of Clay

Trout Creek By Pass - South Interchange

F98179-A/G

Oct 21/98

Figure A24

B

# RECORD OF BOREHOLE BH-1FF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 091 692.1 N, 315 215.4 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow Stem Augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE May 29, 1998 CHECKED BY D.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER TYPE BLOWS/0.3m			20 40 60 80	20 40 60 80	wp   w   wl	10 20 30 40		
313.33	GROUND SURFACE										
0.00	PEAT, black, wet. (very soft)										
312.71											
0.62	SILT, grey, some small root fibers, wet. (very loose)		1 SS 3								1% 21% 78%
312.00			2 SS 3								0% 2% 98%
1.33	SILTY CLAY, grey, wet, thinly laminated with SILT. (stiff to very stiff)		3 SS 4								
			4 SS 2								
			5 SS 3								
			6 TW							17.80	
			7 SS 2								
			8 SS 2								
303.33											
10.00	SILT, grey, wet, with thin clay seams, wet. (loose)		9 SS 8								0% 0% 100%
301.83											
11.50	SAND & SILTY SAND, brown, wet, grading to SAND with GRAVEL. (very loose to compact)		10 SS 2								0% 72% 28%
			11 SS 14								
298.33											
15.00	BIOTITE HORNBLENDE GNEISS, excellent rock quality, unweathered.		12 BQ								Rec. 100% RQD 93%
			13 BQ								Rec. 100% RQD 96%
295.18											
18.14	END OF BOREHOLE										
Notes: 1) This borehole forms part of the South Interchange Foundation Investigation. 2) Borehole located at Station 9+955, offset 5 m right of centreline as referenced to Boundary Road. 3) Water level was at 0.2 m & hole was open to 0.5 m depth on completion.											





# RECORD OF BOREHOLE BH-2FF 1 OF 1

## METRIC

W.P. 774-93-00	LOCATION 5 091 726.3 N, 315 242.7 E	ORIGINATED BY I.D.
DIST 54 HWY 11	BOREHOLE TYPE Hollow Stem Augers / CME-55	COMPILED BY M.D.
DATUM Geodetic	DATE June 1, 1998	CHECKED BY D.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		UNIT WEIGHT  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION		
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20    40    60    80				wp    —    w			WATER CONTENT (%)  10    20    30    40	
								SHEAR STRENGTH: Cu, KPa UNCONFINED    QUICK TRIAXIAL    FIELD VANE 20    40    60    80    LAB SHEAR								
313.32	GROUND SURFACE															
0.00	PEAT, black, wet. (very soft)						313									
311.89							312									
1.43	SILT, grey, some small root fibers, wet. (very loose)		1	SS	3		311									
310.82							310									
2.50	SILTY CLAY, grey, wet, thinly laminated with SILT or CLAYEY SILT. (stiff to soft)		2	SS	6		310		X							
			3	SS	6		309									
			4	TW			308									
			5	SS	2		307		X							
			6	SS	2		306									
			7	SS	4		305									
303.52							304									
9.80	SILT, with thin clay seams, grey, wet. (very loose)						303									
301.82							302									
11.50	SAND, brown, wet, grading to SAND with GRAVEL. (loose to compact)		8	SS	3		301									
			9	SS	11		300									
			10	SS	12		299									
			11	SS	6		298									
							297									
	Occasional cobble sizes at base.						296									
294.36			12	SS	7		295									
18.96	BIOTITE HORNBLENDE GNEISS, fair to good rock quality, slightly weathered.		13	BQ			294									
			14	BQ			293									
291.28							292									
22.04	END OF BOREHOLE															
<p>Notes:</p> <p>1) This borehole forms part of the South Interchange Foundation Investigation.</p> <p>2) Borehole located at Station +9 + 999, on centreline as referenced to Boundary Road.</p> <p>3) Water level was at 0.2 m &amp; hole was open to 0.7 m depth on completion.</p>																





# RECORD OF BOREHOLE AP-12FF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 091 698.9 N, 315 209.3 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE July 9, 1998 CHECKED BY D.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT				UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60	80	wp	w	wl		
								SHEAR STRENGTH: Cu, KPa UNCONFINED      FIELD VANE QUICK TRIAXIAL      LAB SHEAR				WATER CONTENT (%)				
						20	40	60	80	10	20	30	40		GR   SA   (SI & CL)	
313.41 0.00	GROUND SURFACE															
312.34 1.07	Probable PEAT															
	Probable SILT															
310.34 3.07	Probable SILTY CLAY															
				VS												
				VS												
				VS												
303.81 9.60	END OF AUGER PROBE															
<div>Notes: 1) This auger probe forms part of the South Interchange Foundation Investigation. 2) Auger probe located at Station 9+956, offset 4 m left of centreline as referenced to Boundary Road.</div>																



# RECORD OF BOREHOLE AP-13FF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 091 729.1 N, 315 242.7 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE July 10, 1998 CHECKED BY D.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT wp	NATURAL MOISTURE CONTENT w	LIQUID LIMIT wl	UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20	40	60	80					
313.29 0.00	GROUND SURFACE														
312.22 1.07	Probable PEAT														
	Probable SILT														
310.24 3.05															
	Probable SILTY CLAY with SILT			VS											
				VS											
				VS											
303.69 9.60	END OF AUGER PROBE														
Notes: 1) This auger probe forms part of the South Interchange Foundation Investigation. 2) Auger probe located at Station +10+001, offset 2 m left of centreline as referenced to Boundary Road.															



# RECORD OF BOREHOLE AP-14FF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 091 754.4 N, 315 276.8 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE July 10, 1998 CHECKED BY D.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLLOT	NUMBER	TYPE			20	40	60	80					
313.07 0.00	GROUND SURFACE					313									
312.00 1.07	Probable PEAT					312									
	Probable SILT					311									
310.02 3.05						310									
	Probable SILTY CLAY with SILT					309									
						308									
						307									
				VS		306									
						305									
				VS		304									
303.47 9.60	END OF AUGER PROBE			VS											
Notes: 1) This auger probe forms part of the South Interchange Foundation Investigation. 2) Auger probe located at Station 10+042, offset 4 m right of centreline as referenced to Boundary Road.															



1 OF 1

METRIC

W.P. 774-93-00

LOCATION 5 091 700.5 N, 315 200.8 E

ORIGINATED BY I.D.

DIST 54 HWY 11

**BOREHOLE TYPE** Standard augers / Dynamic cone

COMPILED BY M.D.

DATUM Geodetic

DATE May 28, 1998

CHECKED BY D.G.

[illegible]

1 OF 1

METRIC

W.P. 774-93-00

LOCATION 5 091 728.4 N, 315 243.4 E

ORIGINATED BY I.D.

DIST 54 HWY 11

**BOREHOLE TYPE** Standard augers / Dynamic cone

COMPILED BY M.D.

DATUM Geodetic

DATE June 1, 1998

CHECKED BY D.G.

[illegible]

# RECORD OF BOREHOLE C-3FF

1 OF 1

METRIC

W.P. 774-93-00

LOCATION 5 091 753.7 N, 315 271.4 E

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Standard augers / Dynamic cone

COMPILED BY M.D.

DATUM Geodetic

DATE June 2, 1998

CHECKED BY D.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60	80	wp	w		
313.07	GROUND SURFACE					313									
0.00	Dynamic cone penetration test only.														
						312									
						311									
						310									
						309									
						308									
						307									
						306									
						305									
						304									
						303									
						302									
						301									
						300									
						299									
						298									
						297									
						296									
						295									
294.48 18.59	<p>END OF CONE TEST BOUNCING REFUSAL ON PROBABLE BEDROCK OR POSSIBLE BOULDER</p> <p>Notes:</p> <p>1) This cone test forms part of the South Interchange Foundation Investigation.</p> <p>2) Cone test located at Station 10+041.5, offset ~5.0 m right of centreline as referenced to Boundary Road.</p>														





# RECORD OF BOREHOLE C-4FP

1 OF 1

METRIC

W.P. 774-93-00

LOCATION 5 091 795.3 N, 315 316.5 E

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Dynamic Cone / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE June 5, 1998

CHECKED BY D.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)				WATER CONTENT (%)			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT			
				BLOWS/0.3m			20	40	60	80	wp	w	wl			
313.40	GROUND SURFACE															
0.00	Dynamic cone penetration test only.					313										
						312										
						311										
						310										
						309										
						308										
						307										
						306										
						305										
						304										
						303										
						302										
301.21	END OF CONE TEST															
12.19	Notes: 1) This cone test forms part of the South Interchange (Boundary Road) Approach Foundation Investigation. 2) Cone test located at station 10+100, on centreline as referenced to Boundary Road.															



# RECORD OF BOREHOLE BH-1FP 1 OF 1

## METRIC

W.P. 774-93-00	LOCATION 5 091 943.9 N, 315 450.0 E	ORIGINATED BY I.D.
DIST 54 HWY 11	BOREHOLE TYPE Hollow Stem Augers / CME-55	COMPILED BY M.D.
DATUM Geodetic	DATE June 4, 1998	CHECKED BY D.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR	wp	w			wl
312.25	GROUND SURFACE													
0.00	PEAT, brown, wet.													
311.33														
0.92	SILT, grey, moist to wet, local thin clay laminations. (compact)		1	SS	13									0% 5% 95%
308.50			2	SS	17									
3.75	SILTY CLAY, grey, wet, thinly laminated with SILT. (soft to firm)		3	SS	2									
			4	SS	2									
			5	SS	1									
			6	SS	1									
300.21														
12.04	SILT, grey, wet, local SILTY CLAY or SANDY SILT layers. (loose to compact)		7	SS	6									
			8	SS	9									
296.55			9	SS	11									
15.70	END OF BOREHOLE													
<p>Notes:</p> <p>1) This borehole forms part of the South Interchange (Boundary Road) Approach Foundation Investigation.</p> <p>2) Borehole located at station ~10+300, on centreline as referenced to Boundary Road.</p> <p>3) Water level was at ~1.5 m &amp; hole was open to ~4.7 m depth on completion.</p> <p>4) Borehole extended to full depth on August 13, 1998.</p>														



# RECORD OF BOREHOLE BH-2FP 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION 5 091 656.6 N, 315 172.3 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow Stem Augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 8, 1998 CHECKED BY D.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80		wp — w — wl				
								SHEAR STRENGTH: Cu, KPa		WATER CONTENT (%)				
								UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR					
								20 40 60 80						
313.91	GROUND SURFACE													
313.61	ORGANICS/PEAT													
0.30	SILT, grey, wet. (loose)													
312.51														
1.40	SILTY CLAY, grey, wet, locally laminated. (stiff to firm)		1	SS	6				X					
			2	SS	4									
			3	SS	4									
			4	SS	6									
306.91														
7.00	SILT, grey, wet, trace of clay. (loose to compact)													0% 10% 90%
			5	SS	9									
305.21														
8.70	SAND, brown, wet. (compact) Grading to SAND with GRAVEL. (compact to dense)		6	SS	10									
			7	SS	13									
302.40														3% 72% 25%
11.51	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER													
	Notes: 1) This borehole forms part of the South Interchange (Boundary Road) Approach Foundation Investigation. 2) Borehole located at station +9+900, on centreline as referenced to Boundary Road. 3) Water level was at 0.7 m & hole was open to 4.8 m depth on completion.													



# RECORD OF BOREHOLE BH-3FP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 091 670.8 N, 315 158.3 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow Stem Augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 9, 1998 CHECKED BY D.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20	40	60	80					
314.18	GROUND SURFACE						314									
313.92	ORGANICS/PEAT															
0.46	SILT, grey, moist, trace of CLAY. (compact)															
312.68							313									
1.50	SILTY CLAY, grey, wet, thinly laminated with SILT. (stiff to firm)		1	SS	9		312				120					
			2	TW			311								18.30	
309.68							310									
4.50	SILT, grey, wet, local thin CLAY or fine SAND layers. (loose)		3	SS	6		309									
			4	SS	6		308									0% 27% 73%
306.98							307									
7.20	SAND, brown, wet grading to SILTY SAND with GRAVEL. (compact to dense)		5	SS	11		306									
			6	SS	13		305									
302.75			7	SS	31		304									34% 50% 16%
11.43	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER						303									
Notes: 1) This borehole forms part of the South Interchange (Boundary Road) Approach Foundation Investigation. 2) Borehole located at station +9 + 900, offset ~20 m left of centreline as referenced to Boundary Road. 3) Water level was at ~2.0 m & hole was open to ~6.0 m depth on completion.																



# RECORD OF BOREHOLE BH-4FP 1 OF 1

## METRIC

W.P. 774-93-00	LOCATION 5 091 642.4 N, 315 186.4 E	ORIGINATED BY I.D.
DIST 54 HWY 11	BOREHOLE TYPE Hollow Stem Augers / CME-55	COMPILED BY M.D.
DATUM Geodetic	DATE June 9, 1998	CHECKED BY D.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80		wp  ---  w  ---  wl				
								SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL X FIELD VANE LAB SHEAR		WATER CONTENT (%)				
						20 40 60 80		10 20 30 40			kN/m <sup>3</sup>	GR SA (SI & CL)		
313.62	GROUND SURFACE													
0.00	ORGANICS/PEAT													
312.85														
0.77	SILT, grey, wet. (compact)													
312.12														
1.50	SILTY CLAY, grey, wet, thinly laminated with SILT. (stiff to soft)		1	SS	8									
			2	SS	3									
			3	TW										
			4	SS	2									
306.62														
7.00	SILT, grey, wet, trace of clay. (loose)		5	SS	7									
304.82														
8.80	SAND, grey, wet, grading to SAND with GRAVEL. (very loose to compact)		6	SS	2									
			7	SS	5									
301.28														
12.34	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER													
<p>Notes:</p> <p>1) This borehole forms part of the South Interchange (Boundary Road) Approach Foundation investigation.</p> <p>2) Borehole located at station +9+900, offset ~20 m right of centreline as referenced to Boundary Road.</p> <p>3) Water level was at ~0.5 m &amp; hole was open to ~5.6 m depth on completion.</p>														



# RECORD OF BOREHOLE BH-5FP 1 OF 1

METRIC

W.P. 774-93-00

LOCATION 5 091 691.8 N, 315 207.9 E

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Hollow Stem Augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE June 8, 1998

CHECKED BY D.G.

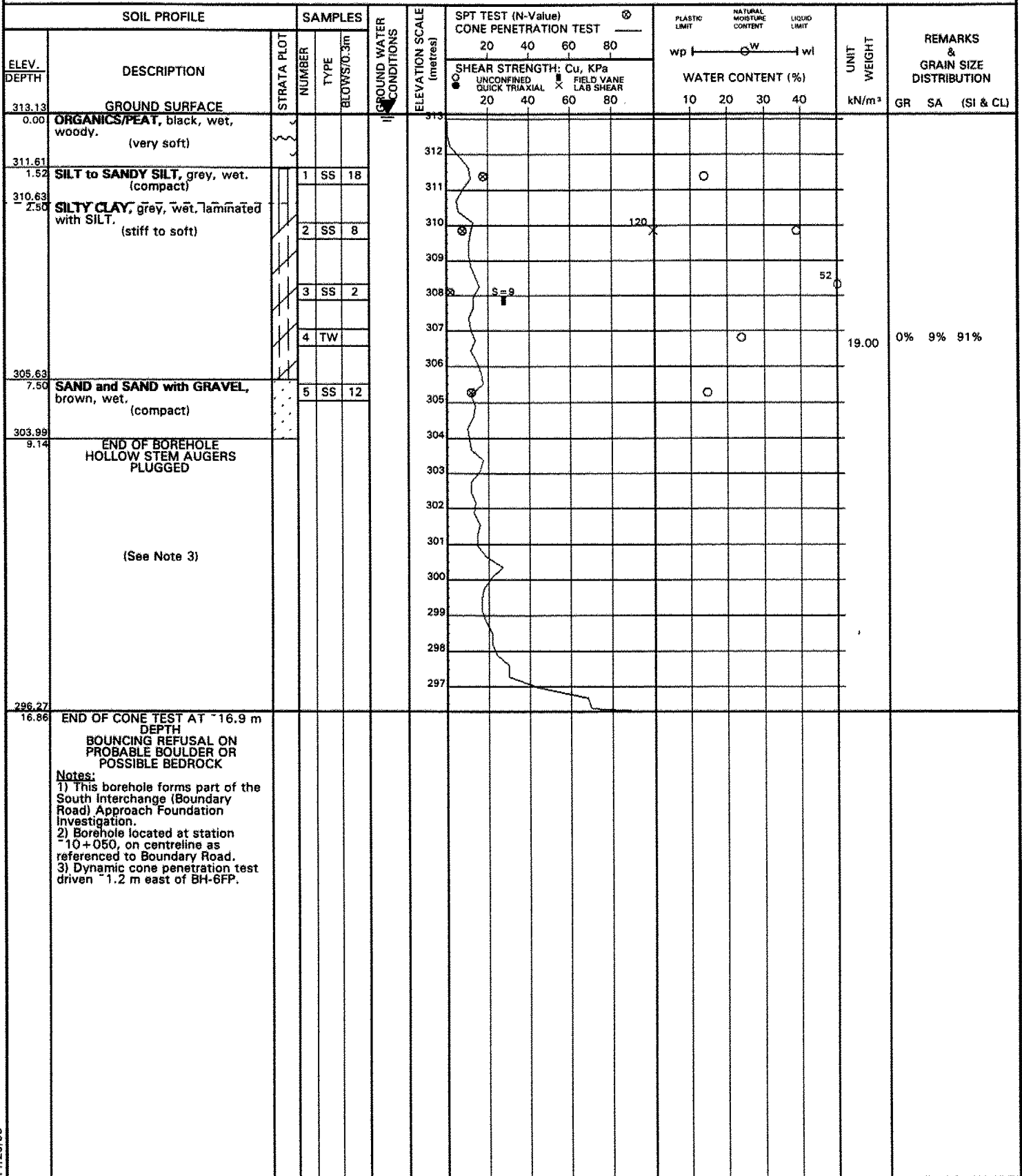
SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20	40	60	80	wp	w	wl		
313.33	GROUND SURFACE														
0.00	PEAT, black to brown, wet. (soft)														
312.23															
1.10	SILT, grey, wet, thin CLAY laminations. (loose)		1	SS	5										
311.23															
2.10	SILTY CLAY, grey, wet, thinly laminated with SILT. (firm to soft)		2	SS	2										
			3	SS	1										
			4	SS	0										
			5	SS	5										
304.83															
8.50	SILT, grey, wet, trace of CLAY. (loose)		6	SS	4										
303.33															
10.00	SAND, brown, wet, grading to SAND with GRAVEL. (loose to compact)		7	SS	9										
			8	SS	10										
			9	SS	13										
299.15															
14.17	END OF BOREHOLE														
Notes: 1) This borehole forms part of the South Interchange (Boundary Road) Approach Foundation Investigation. 2) Borehole located at station +9+950, on centreline as referenced to Boundary Road. 3) Water level was at 1.0 m & hole was open to 4.7 m depth on completion.															



# RECORD OF BOREHOLE BH-6FP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 091 762.2 N, 315 278.9 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow Stems / Cone / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 4, 1998 CHECKED BY D.G.



# RECORD OF BOREHOLE BH-7FP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 091 797.4 N, 315 314.4 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow Stem Augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 5, 1998 CHECKED BY D.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)			
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20	40	60	80			wp	w	wl
								SHEAR STRENGTH: Cu, KPa		WATER CONTENT (%)						
								UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR							
313.44	GROUND SURFACE															
0.00	ORGANICS/PEAT, brown, wet, woody. (very soft)															
311.76			1	SS	15											
1.68	SILT to SANDY SILT, grey, wet. (compact)															
310.74			2	SS	12											
2.70	SILTY CLAY, grey, wet, laminated with SILT. (stiff to soft)															
			3	SS	2											
			4	TW												
			5	SS	1											
			6	SS	2											
302.94																
10.50	SILT, grey, wet, local SILTY CLAY or SANDY SILT layers. (loose)		7	SS	6											
300.79			8	SS	7											
12.65	END OF BOREHOLE															
Notes: 1) This borehole forms part of the South Interchange (Boundary Road) Approach Foundation Investigation. 2) Borehole located at station 10+100, on centreline as referenced to Boundary Road. 3) Water level was at 0.7 m & hole was open to 5.7 m depth on completion.																





# RECORD OF BOREHOLE BH-8FP 1 OF 1

METRIC

W.P. 774-93-00

LOCATION 5 091 811.6 N, 315 300 3 E

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Hollow Stem Augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE June 8, 1998

CHECKED BY D.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20 40 60 80		wp — w — wl			
								SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL X FIELD VANE LAB SHEAR 20 40 60 80		WATER CONTENT (%) 10 20 30 40			
313.65	GROUND SURFACE												
0.00	ORGANICS/PEAT, brown, wet, woody. (very soft)												
312.13													
1.52	SILT to SANDY SILT, grey, wet. (compact)		1	SS	10								
311.15													
2.50	SILTY CLAY, grey, wet, laminated with SILT. (stiff to soft)		2	SS	9								
			3	TW									
			4	SS	3								
			5	SS	4								
			6	SS	5								
303.65											16.70		
10.00	SILT, grey, wet, local SILTY CLAY or SANDY SILT layers. (loose)		7	SS	6							0% 6% 94%	
			8	SS	5								
301.00	END OF BOREHOLE												
12.65	Notes: 1) This borehole forms part of the South Interchange (Boundary Road) Approach Foundation Investigation. 2) Borehole located at station -10+100, offset -25 m left of centreline as referenced to Boundary Road. 3) Water level was at -0.8 m & hole was open to -5.5 m depth on completion.												



# RECORD OF BOREHOLE BH-9FP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 091 783.2 N, 315 328.5 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow Stem Augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 8, 1998 CHECKED BY D.G.

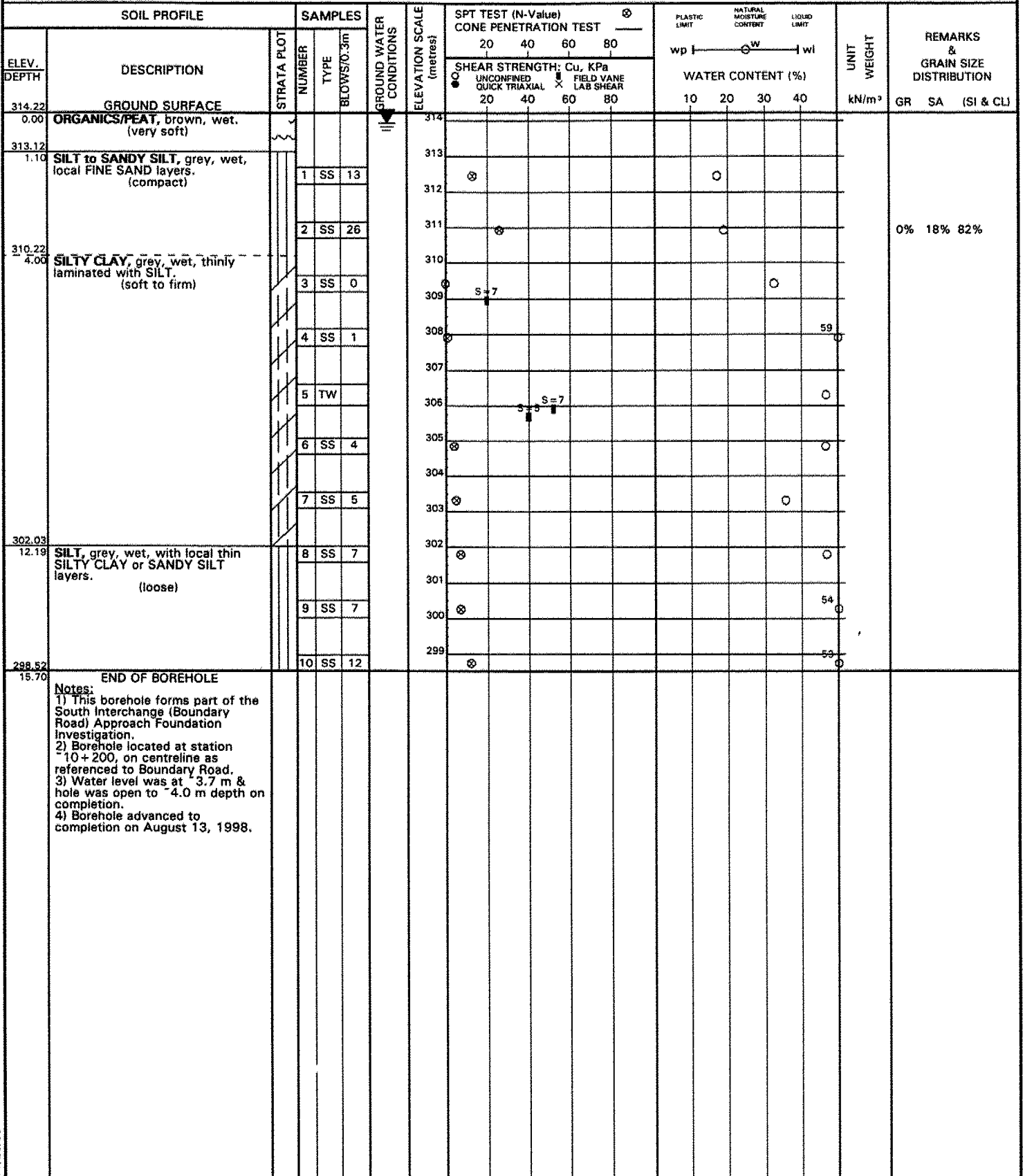
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80				wp — w — wl				
								SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)				
								UNCONFINED QUICK TRIAXIAL    FIELD VANE LAB SHEAR				20 40 60 80				
312.92	GROUND SURFACE															
0.00	ORGANICS/PEAT, brown, wet, woody. (very soft)															
311.24			1	SS	11										0% 25% 75%	
1.68	SILT to SANDY SILT, grey, wet. (compact)															
310.42	SILTY CLAY, grey, wet, thinly laminated with SILT. (firm to soft)		2	TW												
2.50																
			3	SS	1											
			4	SS	3											
			5	SS	2											
			6	SS	4											
302.92																
10.00	SILT, grey, wet, local SILTY CLAY or SANDY SILT layers. (loose)		7	SS	9											



# RECORD OF BOREHOLE BH-10FP 1 OF 1

## METRIC

W.P. 774-93-00	LOCATION 5 091 865.8 N, 315 385.4 E	ORIGINATED BY I.D.
DIST 54 HWY 11	BOREHOLE TYPE Hollow Stem Augers / CME-55	COMPILED BY M.D.
DATUM Geodetic	DATE June 5, 1998	CHECKED BY D.G.



# RECORD OF BOREHOLE BH-11FP 1 OF 1

METRIC

W.P. 774-93-00

LOCATION 5 091 663.7 N, 315 270.8 E

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Hollow stem augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE July 9, 1998

CHECKED BY D.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20	40	60	80					
313.16	GROUND SURFACE						313									
0.00	PEAT, brown, wet. (very soft)						312									
311.94							311									
1.22	SILT, grey, wet. (loose)		1	SS	6		310									
310.66							309									
2.50	SILTY CLAY, grey, wet, locally thinly laminated with SILT. (stiff to soft)		2	SS	7		308									
			3	TW			307									
			4	SS	2		306									
			5	SS	1		305									
			6	TW			304									
			7	SS	3		303									
301.66							302									
11.50	SILT, grey, wet, local thin SILTY CLAY or SANDY SILT layers. (loose to compact)		8	SS	6		301									
			9	SS	11		300									
298.66							299									
14.50	SAND, brown, wet, grading to SAND with GRAVEL. (loose to dense)		10	SS	7		298									
			11	SS	43		297									
294.93							296									
18.23	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER						295									
Notes: 1) This borehole forms part of the South Interchange (Boundary Road) Approach Foundation Investigation. 2) Borehole located at station +8+665, on centreline as referenced to the Highway 11 Median. 3) Water level was at ~0.5 m & hole was open to ~11.1 m depth on completion.																



# RECORD OF BOREHOLE BH-15FP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 091 795.8 N, 315 404.9 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE July 13, 1998 CHECKED BY D.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20 40 60 80				wp — w — wl					
								SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL    FIELD VANE LAB SHEAR				WATER CONTENT (%) 10 20 30 40					
312.55	GROUND SURFACE																
0.00	PEAT, brown, wet. (very soft)																
311.48	SILT to SANDY SILT, grey, wet. (compact to dense)		1	SS	11												
1.07			2	SS	36												
308.75	SILTY CLAY, grey, wet, locally thinly laminated with SILT. (firm to soft)		3	SS	2												
3.80			4	TW													
			5	SS	2												
			6	TW													
302.30	SILT, grey, wet, locally thin SILTY CLAY or SANDY SILT layers. (loose to compact)		7	SS	6												
10.25			8	SS	9												
			9	SS	10												
297.35	SAND, grey, wet, grading to SAND with GRAVEL. (compact to very dense)		10	SS	9												
15.20			11	SS	25												
			12	SS	156												
291.52	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER																
21.03	Notes: 1) This borehole forms part of the South Interchange (Boundary Road) Approach Foundation Investigation. 2) Borehole located at Station 8+400.0, offset ~4.0 m left of centreline as referenced to the EW-N Ramp. 3) Water level was at ~0.2 m & hole was open to ~7.0 m depth on completion.																



# RECORD OF BOREHOLE BH-16FP 1 OF 1

METRIC

W.P. 774-93-00

LOCATION 5 091 706.5 N, 315 438.0 E

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Hollow stem augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE July 13, 1998

CHECKED BY D.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20	40	60	80					
312.27	GROUND SURFACE														
0.00	PEAT, brown, wet. (very soft)														
311.42															
0.85	SILTY SAND, grey, wet. (compact)		1	SS	10										
309.77															
2.50	SILT to SANDY SILT, grey, wet. (compact to loose)		2	SS	31										
306.77															
5.50	SILTY CLAY, grey, wet, thinly laminated with SILT. (soft to firm)		3	SS	7										
			4	SS	2										
			5	TW											
			6	SS	2										
			7	SS	2										
			8	SS	3										
			9	SS	6										
297.27															
15.00	SILT, grey, wet, local thin SILTY CLAY or SANDY SILT layers. (loose to compact)		10	SS	9										
			11	SS	14										
291.07															
21.20	SAND to SAND WITH GRAVEL, grey, wet. (compact to dense)		12	SS	29										
			13	SS	38										
285.60															
26.67	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER														
	Notes: 1) This borehole forms part of the South Interchange (Boundary Road) Approach Foundation Investigation. 2) Borehole located at Station														



8+500.0, on centreline as  
referenced to the EW-N Ramp.  
3) Water level was at -0.3 m &  
hole was open to -20.9 m depth  
on completion.

## METRIC

ORIGINATED BY I.D.

COMPILED BY M.D.

CHECKED BY D.G.



# RECORD OF BOREHOLE BH-18FP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 091 651.9 N, 315 083.6 E ORIGINATED BY R.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE July 15, 1998 CHECKED BY D.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT			UNIT WEIGHT  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60	80	wp	w		
316.26	GROUND SURFACE														
0.00	TOPSOIL														
315.66	SAND WITH GRAVEL, brown, wet.  (compact to dense)														
0.60			1	SS	13										
			2	SS	38										
312.09	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER														
4.17	Notes: 1) This borehole forms part of the South Interchange (Boundary Road) Approach Foundation Investigation. 2) Borehole located at Station 9+075.0, on centreline as referenced to the W-S Ramp. 3) Water level was at surface & hole was open to ~3.0 m depth on completion.														



# RECORD OF BOREHOLE BH-19FP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 091 723.4 N, 315 023.1 E ORIGINATED BY R.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE July 15, 1998 CHECKED BY D.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)				PLASTIC LIMIT			NATURAL MOISTURE CONTENT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	CONE PENETRATION TEST				w <sub>p</sub> — w — w <sub>L</sub>			WATER CONTENT (%)			
							20	40	60	80								
							SHEAR STRENGTH: Cu, KPa											
							UNCONFINED QUICK TRIAXIAL X FIELD VANE LAB SHEAR											
							20	40	60	80								
321.83	GROUND SURFACE																	
321.60	TOPSOIL																	
0.30	SAND WITH GRAVEL, brown, wet.																	
	(compact)																	
319.85			1	SS	25													
1.98	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER																	
	Notes: 1) This borehole forms part of the South Interchange (Boundary Road) Approach Foundation Investigation. 2) Borehole located at Station 8+990.0, offset ~20.0 m right of centreline as referenced to the EW-S Ramp. 3) Water level was at ~0.9 m & hole was open to ~1.8 m depth on completion.																	



# RECORD OF BOREHOLE BH-20FP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 092 031.6 N, 315 042.8 E ORIGINATED BY R.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE July 15, 1998 CHECKED BY D.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m	20	40	60	80	wp	w	wl	WATER CONTENT (%)		
319.18	GROUND SURFACE														
318.99	TOPSOIL														
0.15	SAND WITH GRAVEL, brown, damp. (dense to very dense)		1	SS	56										
315.52	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER		2	SS	50										
3.66															
	Notes: 1) This borehole forms part of the South Interchange Approach Foundation Investigation. 2) Borehole located at Station 9+100.0, on centreline of Highway 11 Median. 3) Borehole was dry & open to 3.0 m depth on completion.														



# RECORD OF BOREHOLE BH-21FP 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION 5 091 954.4 N, 315 106.3 E ORIGINATED BY R.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE July 15, 1998 CHECKED BY D.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT				UNIT WEIGHT kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLLOT	NUMBER			TYPE	BLOWS/0.3m	SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)			
						UNCONFINED QUICK TRIAXIAL      FIELD VANE LAB SHEAR				wp ——— w ——— wl					
						20   40   60   80				10   20   30   40					
314.68	GROUND SURFACE														
313.68 0.30	TOPSOIL														
	SILTY SAND, brown, damp. (compact)		1	SS	15										
311.93 2.75	SAND to SAND WITH GRAVEL, damp to wet. (compact to dense)		2	SS	31										
			3	SS	16										
			4	SS	50										
307.97 6.71	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER														
	<b>Notes:</b> 1) This borehole forms part of the South Interchange Approach Foundation Investigation. 2) Borehole located at Station 9+000.0, on centreline of Highway 11 Median. 3) Water level was at ~0.9 m & hole was open to ~5.8 m depth on completion.														



# RECORD OF BOREHOLE BH-22FP 1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 091 872.4 N, 315 163.5 E ORIGINATED BY R.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE July 15, 1998 CHECKED BY D.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20	40	60	80					
313.55	GROUND SURFACE														
0.00	ORGANICS/PEAT														
312.95															
0.60	SILT, grey, moist, trace of CLAY. (loose)		1	SS	9										
311.05															
2.50	SILTY CLAY, stratified, grey, moist. (soft)		2	SS	4										
309.05															
4.50	SILT, grey, wet, local thin SILTY CLAY or SANDY SILT layers. (loose)		3	SS	7										
308.05															
4.50			4	SS	2										
306.05															
7.50	SAND WITH GRAVEL, grey, wet. (very dense)		5	SS	56										
304.41															
9.14	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER														
Notes: 1) This borehole forms part of the South Interchange (Boundary Road) Approach Foundation Investigation. 2) Borehole located at Station 8+900.0, on centreline of Median as referenced to Boundary Road. 3) Water level was at 1.2 m & hole was open to 6.1 m depth on completion.															



# RECORD OF BOREHOLE BH-23FP 1 OF 1

METRIC

W.P. 774-93-00

LOCATION 5 091 786.1 N, 315 213.9 E

ORIGINATED BY R.M.

DIST 54 HWY 11

BOREHOLE TYPE Hollow stem augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE July 16, 1998

CHECKED BY D.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20	40	60	80					
313.41	GROUND SURFACE														
0.00	PEAT, black, wet. (very soft)														
312.01															
1.40	SILT, grey, moist, with SILTY CLAY laminations. (compact)		1	SS	11										
310.91	SILTY CLAY, grey, wet, locally thinly laminated with SILT. (firm to soft)		2	SS	3										
2.50			3	SS	2										
307.51	SILT, grey, wet, local thin SILTY CLAY layers. (loose to very loose)		4	SS	4										
5.90			5	SS	2										
304.61	SAND, brown, wet, fine to medium grained. (loose)		6	SS	3										
8.80															
302.28	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER														
11.13	Notes: 1) This borehole forms part of the South Interchange (Boundary Road) Approach Foundation Investigation. 2) Borehole located at Station 8+800.0, on centreline of the Highway 11 Median. 3) Water level was at ~1.1 m & hole was open to ~9.1 m depth on completion.														



# RECORD OF BOREHOLE BH-24FP 1 OF 1

METRIC

W.P. 774-93-00

LOCATION 5 091 768.6 N. 315 180.4 E

ORIGINATED BY R.M.

DIST 54 HWY 11

BOREHOLE TYPE Hollow stem augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE July 16, 1998

CHECKED BY D.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20 40 60 80				wp — w — wl				
								SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL X FIELD VANE LAB SHEAR 20 40 60 80				WATER CONTENT (%) 10 20 30 40				
313.57	GROUND SURFACE															
0.00	PEAT, black, wet. (very soft)						313									
312.20							312									
1.37	SILT, grey, wet, with SILTY CLAY laminations. (loose)		1	SS	8		311									
311.07	SILTY CLAY, grey, wet, locally laminated with SILT. (firm)		2	SS	6		310									
2.50							309									
308.37	SILT, grey, wet, local thin SILTY CLAY layers. (loose)		3	SS	3		308									
5.20			4	SS	9		307									
306.07	SAND to SAND WITH GRAVEL, grey, wet. (loose to compact)		5	SS	9		306									
7.50			6	SS	13		305									
302.75			7	SS	50		304									
10.82	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR PROBABLE BOULDER						303									
<p>Notes:</p> <p>1) This borehole forms part of the South Interchange (Boundary Road) Approach Foundation investigation.</p> <p>2) Borehole located at Station 8+800.0, on centreline of EW-S Ramp. 3) Water level was at ~0.8 m &amp; hole was open to ~9.1 m depth on completion.</p>																



# RECORD OF BOREHOLE BH-25FP 1 OF 1

## METRIC

W.P. 774-93-00	LOCATION 5 091 596.7 N, 315 275.6 E	ORIGINATED BY S.M.
DIST 54 HWY 11	BOREHOLE TYPE Hollow stem augers / CME-55	COMPILED BY M.D.
DATUM Geodetic	DATE August 17, 1998	CHECKED BY D.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION			
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20						40	60	80
313.15	GROUND SURFACE															
0.00	PEAT, black, wet. (very soft)															
312.39	SILT to SANDY SILT, grey, wet. (loose)		1	SS	6											
0.76																
310.10			2	SS	6											
3.05		SILTY CLAY, grey, wet, locally thinly laminated with SILT. (stiff to soft)		3	SS	1										
				4	TW											
				5	SS	1										
				6	SS	0										
			7	SS	4											
301.87	SILT, grey, wet, local thin SILTY CLAY or SANDY SILT layers. (loose)		8	SS	6											
11.28			9	SS	4											
297.78	SAND, brown, wet, grading to SAND with GRAVEL. (loose to compact)		10	SS	7											
15.39																
295.93	END OF BOREHOLE		11	SS	22											
17.22	<p>Notes:</p> <p>1) This borehole forms part of the South Interchange (Boundary Road) Approach Foundation Investigation.</p> <p>2) Borehole located at Station +8+600, on centreline of southbound lane.</p> <p>3) Water level was at ~1.1 m &amp; hole was open to ~9.9 m depth on completion.</p>															





# RECORD OF BOREHOLE BH-26FP 1 OF 1

METRIC

W.P. 774-93-00

LOCATION 5 091 511.5 N, 315 339.9 E

ORIGINATED BY S.M.

DIST 54 HWY 11

BOREHOLE TYPE Hollow stem augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE September 17, 1998

CHECKED BY D.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20	40	60	80	wp			w
313.15	GROUND SURFACE													
0.00	PEAT, black, wet. (very soft)													
311.63														
1.52	SILT to SANDY SILT, grey, wet, local fine SAND layers. (loose)		1	SS	4									
309.65			2	SS	7									
3.50	SILTY CLAY, grey, wet, thinly laminated with SILT. (firm to soft)		3	SS	1									
			4	SS	0									
			5	SS	0									
			6	TW										
302.25			7	SS	0									
10.90	SILT, grey, wet, local thin SILTY CLAY or SANDY SILT layers. (loose to compact)		8	SS	7									
			9	SS	11									
297.91			10	SS	22									
15.24	SAND, grey brown, wet, grading to SAND with GRAVEL. (compact)		11	SS	27									
295.93	END OF BOREHOLE													
17.22	Notes: 1) This borehole forms part of the South Interchange (Boundary Road) Approach Foundation Investigation. 2) Borehole located at Station +8+500, on centreline of northbound lane. 3) Water level was at ~1.2 m & hole was open to ~7.2 m depth on completion.													



# RECORD OF BOREHOLE BH-27FP 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION 5 091 412.3 N, 315 361.0 E ORIGINATED BY S.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE September 18, 1998 CHECKED BY D.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)		CONE PENETRATION TEST	PLASTIC LIMIT wp	NATURAL MOISTURE CONTENT w	LIQUID LIMIT wl	UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	UNCONFINED QUICK TRIAXIAL						
313.24	GROUND SURFACE													
0.00	PEAT, black, wet. (very soft)													
311.56			1	SS	5									
1.68	SILT to SILTY SAND, grey, wet. (loose)													
310.19			2	SS	5									
3.05	SILTY CLAY, grey, wet, thinly laminated with SILT. (firm to soft)													
			3	TW										
			4	SS	2									
			5	SS	0									
			6	TW										
302.57			7	SS	3									
10.67	SILT, grey, wet, local thin SILTY CLAY layers. (loose)													
300.92			8	SS	16									
12.32	SAND, brown, wet, grading to SAND with GRAVEL. (compact to dense)													
299.07			9	SS	35									
14.17	END OF BOREHOLE													
<b>Notes:</b> 1) This borehole forms part of the South Interchange (Boundary Road) Approach Foundation Investigation. 2) Borehole located at Station "8+400, on centreline of northbound lane. 3) Water level was at ~1.2 m & hole was open to ~7.2 m depth on completion.														



# RECORD OF BOREHOLE BH-28FP 1 OF 1

## METRIC

W.P. 774-93-00	LOCATION 5 091 311.6 N, 315 374.0 E	ORIGINATED BY S.M.
DIST 54 HWY 11	BOREHOLE TYPE Hollow stem augers / CME-55	COMPILED BY M.D.
DATUM Geodetic	DATE September 18, 1998	CHECKED BY D.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)		CONE PENETRATION TEST		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR	wp	wl					
313.09	GROUND SURFACE														
0.00	PEAT, black, wet. (very soft)														
311.41			1	SS	3										
1.68	SILTY SAND to SILT, grey, wet. (loose)														
310.04			2	SS	11										
3.05	SILTY CLAY, grey, wet, thinly laminated with SILT. (stiff to soft)														
			3	SS	2										
			4	TW											
			5	SS	0										
			6	SS	4										
			7	SS	2										
300.90															
12.19	SILT, grey, wet. (loose)		8	SS	5										
299.99															
13.10	SAND, brown, wet, grading to SAND with GRAVEL, local cobbles. (dense)		9	SS	44										
297.39			10	SS	49										
15.70	END OF BOREHOLE														
<p><b>Notes:</b></p> <p>1) This borehole forms part of the South Interchange (Boundary Road) Approach Foundation Investigation.</p> <p>2) Borehole located at Station +8 + 300, on centreline of northbound lane.</p> <p>3) Water level was at surface &amp; hole was open to ~6.8 m depth on completion.</p>															



# RECORD OF BOREHOLE BH-29FP 1 OF 1

**METRIC**

W.P. 774-93-00 LOCATION 5 091 209.9 N, 315 341.5 E ORIGINATED BY S.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Hollow stem augers / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE September 21, 1998 CHECKED BY D.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20	40	60	80	wp	w	wl	10		
313.48	GROUND SURFACE															
0.00	PEAT, black, wet. (very soft)															
312.87																
0.61	SANDY CLAY to SILTY CLAY, grey, moist, grey, moist to wet. (stiff)		1	SS	9											
310.43																
3.05	SILTY CLAY, grey, wet, thinly laminated with SILT. (firm to soft)		2	SS	4											
			3	SS	3											
			4	TW												
			5	SS	3											
304.03																
9.45	SILT to SANDY SILT, grey brown, wet. (loose)		6	SS	5											
			7	SS	5											
302.08																
11.40	SAND to SAND with GRAVEL, brown, wet, local cobbles.															
			8	SS	20											
299.31			9	SS	43											
14.17	END OF BOREHOLE															
Notes: 1) This borehole forms part of the South Interchange (Boundary Road) Approach Foundation Investigation. 2) Borehole located at Station 8 + 200, on centreline of southbound lane. 3) Water level was at ~2.5 m & hole was open to ~7.8 m depth on completion.																



# RECORD OF BOREHOLE BH-30FP 1 OF 1

## METRIC

W.P. 774-93-00

LOCATION

ORIGINATED BY S.M.

DIST 54 HWY 11

BOREHOLE TYPE Hollow stem augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE September 21, 1998

CHECKED BY D.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA	NUMBER	TYPE		20	40	60	80	wp	w	wl		
313.00	GROUND SURFACE													
0.00	PEAT, black, wet. (very soft)													
311.32	SILTY SAND to SANDY SILT, grey, wet. (loose to compact)		1	SS	5									
1.68			2	SS	12									
			3	SS	26									
307.52	SILTY CLAY, grey, wet, thinly laminated with SILT. (firm to soft)		4	SS	0									
5.48			5	TW										
			6	SS	0									
			7	SS	0									
299.89	SILT, grey, wet, local thin CLAY or SILTY CLAY layers in upper zone, thin SANDY layers with depth. (loose to compact)		8	SS	0									
13.11			9	SS	5									
			10	SS	7									
			11	SS	11									
294.45	SAND to SAND with GRAVEL, brown, wet, local cobbles. (compact to dense)		12	SS	13									
18.55			13	SS	54									
292.73	END OF BOREHOLE													
20.27	Notes: 1) This borehole forms part of the South Interchange (Boundary Road) Approach Foundation Investigation. 2) Borehole located at Station +8 + 500, on centreline of S-E/W ramp. 3) Water level was at ~1.4 m & hole was open to ~10.2 m depth on completion.													



# RECORD OF BOREHOLE BH-31FP 1 OF 1

METRIC

W.P. 774-93-00

LOCATION 5 091 606.8 N, 315 417.2 E

ORIGINATED BY S.M.

DIST 54 HWY 11

BOREHOLE TYPE Hollow stem augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE September 22, 1998

CHECKED BY D.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20 40 60 80				wp — w — wl				
								SHEAR STRENGTH: Cu, KPa UNCONFINED QUICK TRIAXIAL X FIELD VANE LAB SHEAR				WATER CONTENT (%)				
312.57	GROUND SURFACE															
0.00	PEAT, black, wet. (very soft)															
311.35																
1.22	SILTY SAND to SANDY SILT, grey, wet. (loose to compact)		1	SS	7											
			2	SS	24											
308.30																
4.27	SILTY CLAY, grey, wet, thinly laminated with SILT. (stiff to soft)		3	SS	6											
			4	TW	0											
			5	SS	0											
			6	SS	0											
			7	TW												
			8	SS	1											
			9	SS	5											
			10	SS	2											
			11	SS	3											
294.32																
18.25	SILT, grey, wet, local thin CLAY or SILTY CLAY layers. (loose)		12	SS	7											
			13	SS	5											
291.27																
21.30	SILT to SILTY SAND, grey, wet. (loose to compact)		14	SS	9											
			15	SS	11											
287.92																
24.65	SAND to SAND with GRAVEL, brown, wet. (compact to dense)		16	SS	16											
286.20			17	SS	33											
26.37	END OF BOREHOLE															
<b>Notes:</b> 1) This borehole forms part of the South Interchange (Boundary Road) Approach Foundation Investigation. 2) Borehole located at Station +8+600, on centreline of S-E/W ramp. 3) Water level was at ~1.4 m & hole was open to ~11.8 m depth																



on completion.

# RECORD OF BOREHOLE BH-32FP 1 OF 1

## METRIC

W.P. 774-93-00	LOCATION 5 091 904.5 N, 315 419.4 E	ORIGINATED BY S.M.
DIST 54 HWY 11	BOREHOLE TYPE Hollow stem augers / CME-55	COMPILED BY M.D.
DATUM Geodetic	DATE September 23, 1998	CHECKED BY D.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60	80	wp	w		
313.07	GROUND SURFACE														
0.00	PEAT, black, wet. (very soft)														
312.16	SILT to SANDY SILT, grey, wet. (compact)														
0.91			1	SS	12										
			2	SS	24										
308.96	SILTY CLAY, grey, wet, thinly laminated with SILT. (firm to soft)		3	SS	2										
4.11			4	TW											
			5	SS	0										
			6	SS	1										
			7	SS	0										
			8	TW											
			9	SS	3										
297.83	SILT, grey, wet, local thin SILTY CLAY or SANDY SILT layers. (loose)		10	SS	8										
15.24			11	SS	6										
294.78	SAND, brown to grey, wet, local thin SILTY CLAY or SANDY SILT layers. (loose to dense)		12	SS	1										
18.29			13	SS	39										
292.65	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER														
20.42	Notes: 1) This borehole forms part of the South Interchange (Boundary Road) Approach Foundation Investigation. 2) Borehole located at Station +10+250, on centreline of Boundary Road. 3) Water level was at ~1.4 m & hole was open to ~10.5 m depth on completion.														





# RECORD OF BOREHOLE BH-33FP 1 OF 1

METRIC

W.P. 774-93-00

LOCATION

ORIGINATED BY S.M.

DIST 54 HWY 11

BOREHOLE TYPE Hollow stem augers / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE November 26, 1998

CHECKED BY D.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT NATURAL MOISTURE CONTENT		UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20 40 60 80	20 40 60 80	wp	wl		
0.00	GROUND SURFACE					0						
0.00	GRAVEL (FILL)											
0.45	PEAT, black, wet. (very soft)											
1.20	SILT to SANDY SILT, grey, wet. (loose to compact)											
1.20			1	SS	27							
-4.57	SILTY CLAY, grey, wet, thinly laminated with SILT. (firm to soft)											
4.57			2	SS	2							
			3	TW								
			4	SS	3							
			5	TW								
-16.50	SILT, grey, wet. (loose)											
16.50			6	SS	4							
-18.60	SAND (loose) .M = SAND END OF BOREHOLE											
18.60												
-18.75												
18.75												

Notes:  
 1) This borehole forms part of the South Interchange (Boundary Road) Approach Foundation Investigation.  
 2) Borehole located at Station -10+350, offset -26 m right of centreline of Boundary Road.  
 3) Water level was at -1.8 m & hole was open to -13.6 m depth on completion.



# RECORD OF BOREHOLE C-1FP

1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 091 621.4 N, 315 136.8 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Dynamic Cone / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 5, 1998 CHECKED BY D.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m					
315.46	GROUND SURFACE											
0.00	Dynamic cone penetration test only.											
309.58	END OF CONE TEST BOUNCING REFUSAL ON PROBABLE BEDROCK OR POSSIBLE BOULDER											
5.88	Notes: 1) This cone test forms part of the South Interchange (Boundary Road) Approach Foundation Investigation. 2) Cone test located at station "9+850, on centreline as referenced to Boundary Road.											



# RECORD OF BOREHOLE C-2FP

1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 091 832.6 N, 315 349.9 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Dynamic Cone / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 5, 1998 CHECKED BY D.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)		CONE PENETRATION TEST		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA	NUMBER			TYPE	BLOWS/0.3m	20	40	60	80	wp	w	wl	10		
314.90	GROUND SURFACE																
0.00	Dynamic cone penetration test only.																
305.76	END OF CONE TEST																
9.14	Notes: 1) This cone test forms part of the South Interchange (Boundary Road) Approach Foundation Investigation. 2) Cone test located at station 10+150, on centreline as referenced to Boundary Road.																



# RECORD OF BOREHOLE C-3FP

1 OF 1

METRIC

W.P. 774-93-00 LOCATION 5 091 904.5 N, 315 419.4 E ORIGINATED BY I.D.  
 DIST 54 HWY 11 BOREHOLE TYPE Dynamic Cone / CME-55 COMPILED BY M.D.  
 DATUM Geodetic DATE June 5, 1998 CHECKED BY D.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLAT	NUMBER			TYPE	BLOWS/0.3m					
313.07	GROUND SURFACE											
0.00	Dynamic cone penetration test only.											
312												
311												
310												
309												
308												
307												
306												
305.45	END OF CONE TEST											
7.62	Notes: 1) This cone test forms part of the South Interchange (Boundary Road) Approach Foundation Investigation. 2) Cone test located at station 10+250, on centreline as referenced to Boundary Road.											



1 OF 1

METRIC

W.P. 774-93-00

LOCATION 5 091 654.5 N, 315 151.6 E

ORIGINATED BY I.D.

DIST 54 HWY 11

BOREHOLE TYPE Dynamic Cone / CME-55

COMPILED BY M.D.

DATUM Geodetic

DATE June 5, 1998

**CHECKED BY** D.G.

SOIL PROFILE						
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	SAMPLES NUMBER TYPE BLOWS/0.3m	GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST
313.91	GROUND SURFACE					
0.00	Dynamic cone penetration test only.					
302.51	END OF CONE TEST BOUNCING REFUSAL ON PROBABLE BEDROCK OR POSSIBLE BOULDER					
11.40	Notes: 1) This cone test forms part of the South Interchange (Boundary Road) Approach Foundation Investigation. 2) Cone test located at station -9+898, offset ~1 m right of centreline as referenced to Boundary Road.					



**S07524G/F/G**

**TABLE 1-1**  
**ROCK CORE DESCRIPTION**

BH#	Core Recovery				Core Description	
	Rock Core No.	Depth (m)	% CR*	% RQD**	Depth (m)	Description
<b>SOUTH INTERCHANGE FOUNDATIONS</b>						
1FF	12	15.00 to 16.52	100	93	15.00 to 18.14	<b>Biotite Hornblende Gneiss (Garnetigerous)</b> , white to grey pink, medium to coarse grained, some sulphate inclusions unweathered, fractures widely spaced, dipping at 45° from vertical, planar, smooth
	13	16.52 to 18.14	100	96		
2FF	13	18.96 to 20.39	100	69	18.96 to 22.04	<b>Biotite Hornblende Gneiss</b> , white to pink, medium to coarse grained, slightly unweathered, strong fractures widely spaced, dipping at 45° to 90° from vertical, planar, smooth
	14	20.39 to 22.04	100	72		
3FF	15	18.53 to 20.06	100	89	18.53 to 21.58	<b>Biotite Hornblende Gneiss (Garnetigerous)</b> , white to pink, medium to coarse grained, unweathered, strong fractures widely spaced, dipping at 90° from vertical, planar, smooth
	16	20.56 to 21.58	100	88		

\*CR = Core Recovery  
\*\*RQD = Rock Quality Designation

c

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT

SAND

GRAVEL

FINE

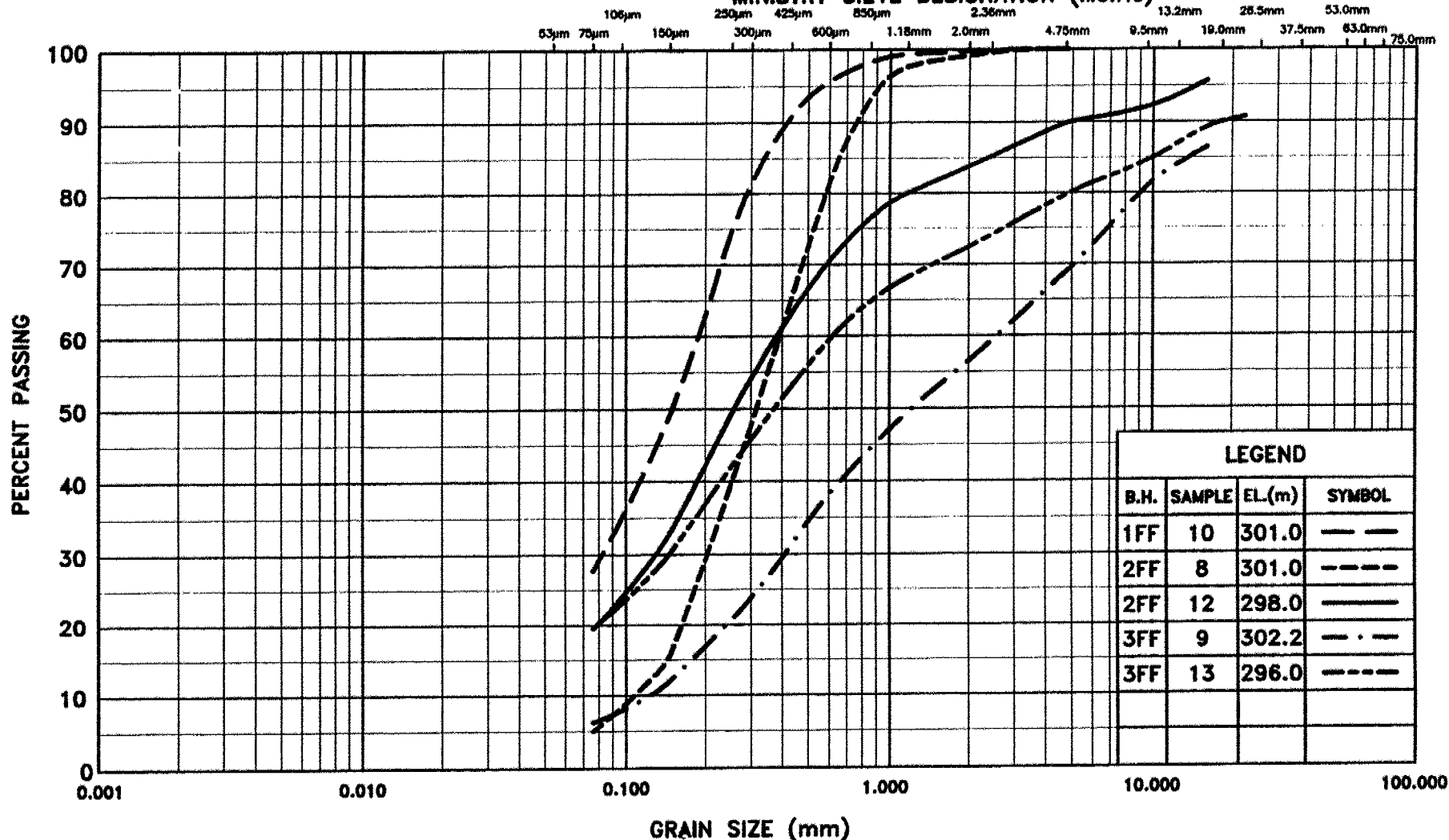
MEDIUM

COARSE

FINE

COARSE

MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION

SAND / SAND & GRAVEL

FIGURE C1

W.P 774-93-00



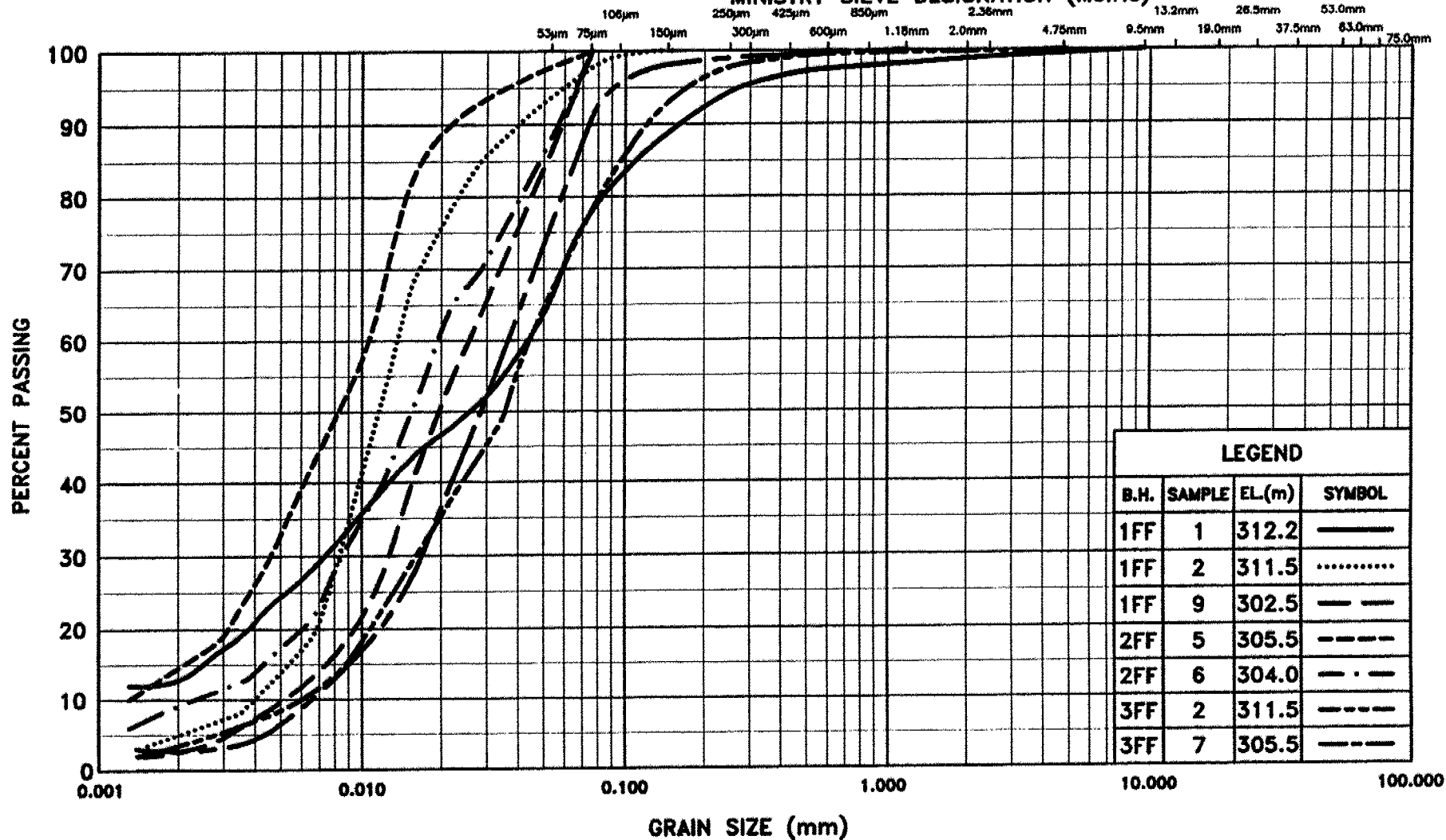
S07524GFF / F98179-A/G



# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION  
SILT & CLAY

FIGURE C2

W.P 774-93-00

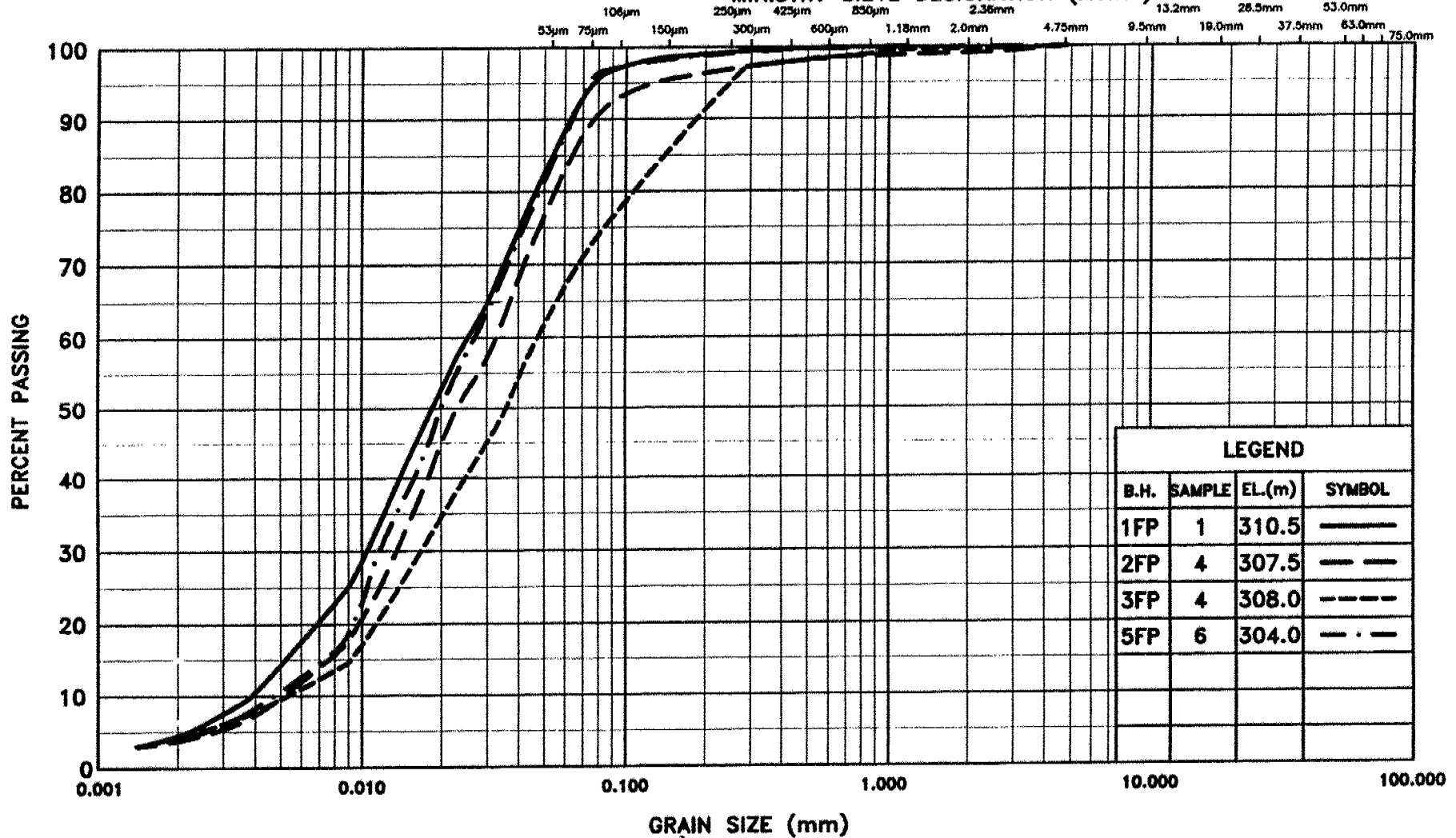


S07524GFF / F98179-A/G

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



LEGEND			
B.H.	SAMPLE	EL.(m)	SYMBOL
1FP	1	310.5	————
2FP	4	307.5	—— —
3FP	4	308.0	- - - -
5FP	6	304.0	- . -

Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION

CLAY & SILT

FIGURE C3

W.P. 774-93-00

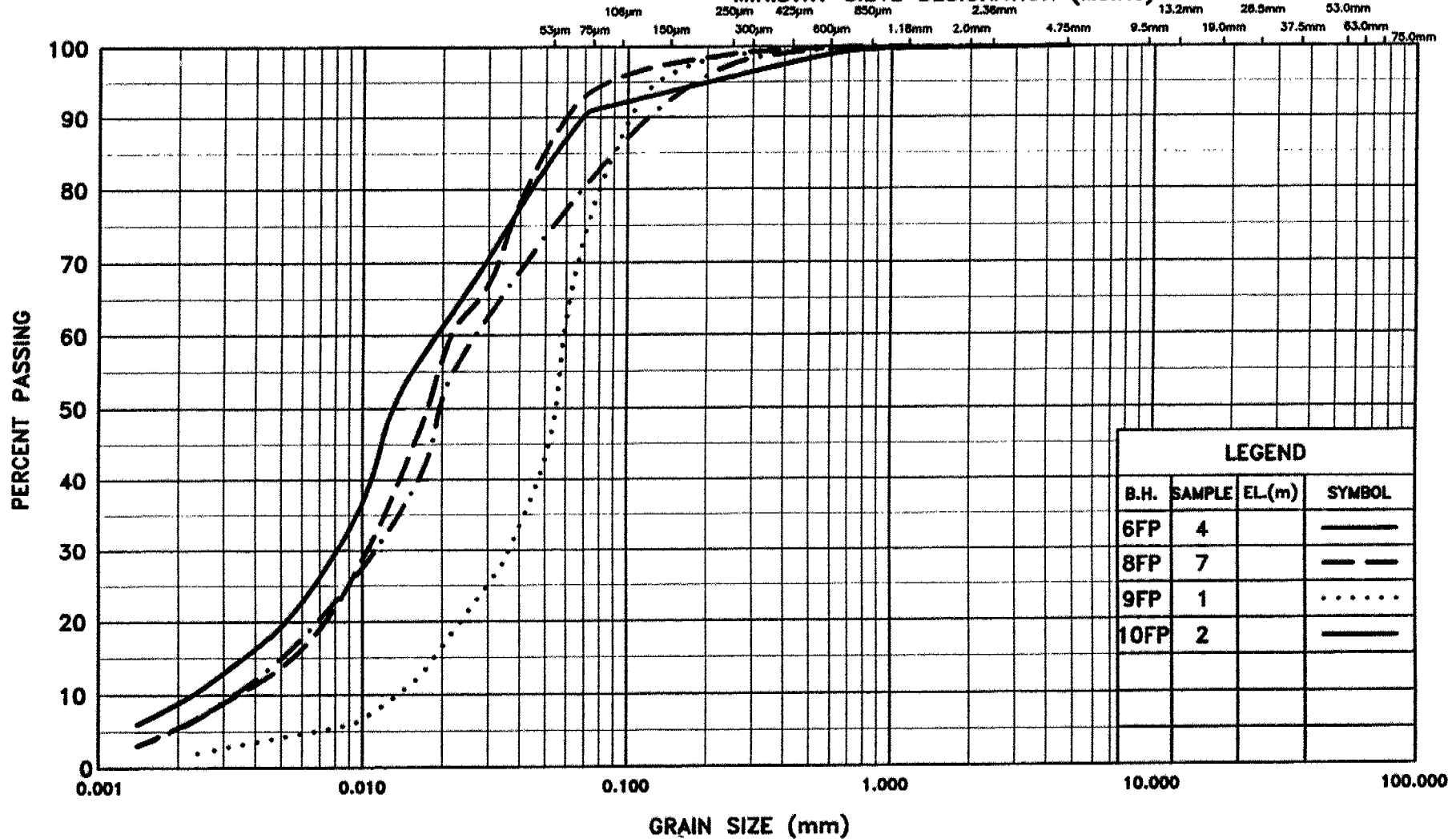


S07524GFP / F98179-A/G

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION

CLAY & SILT

FIGURE C4

W.P. 774-93-00

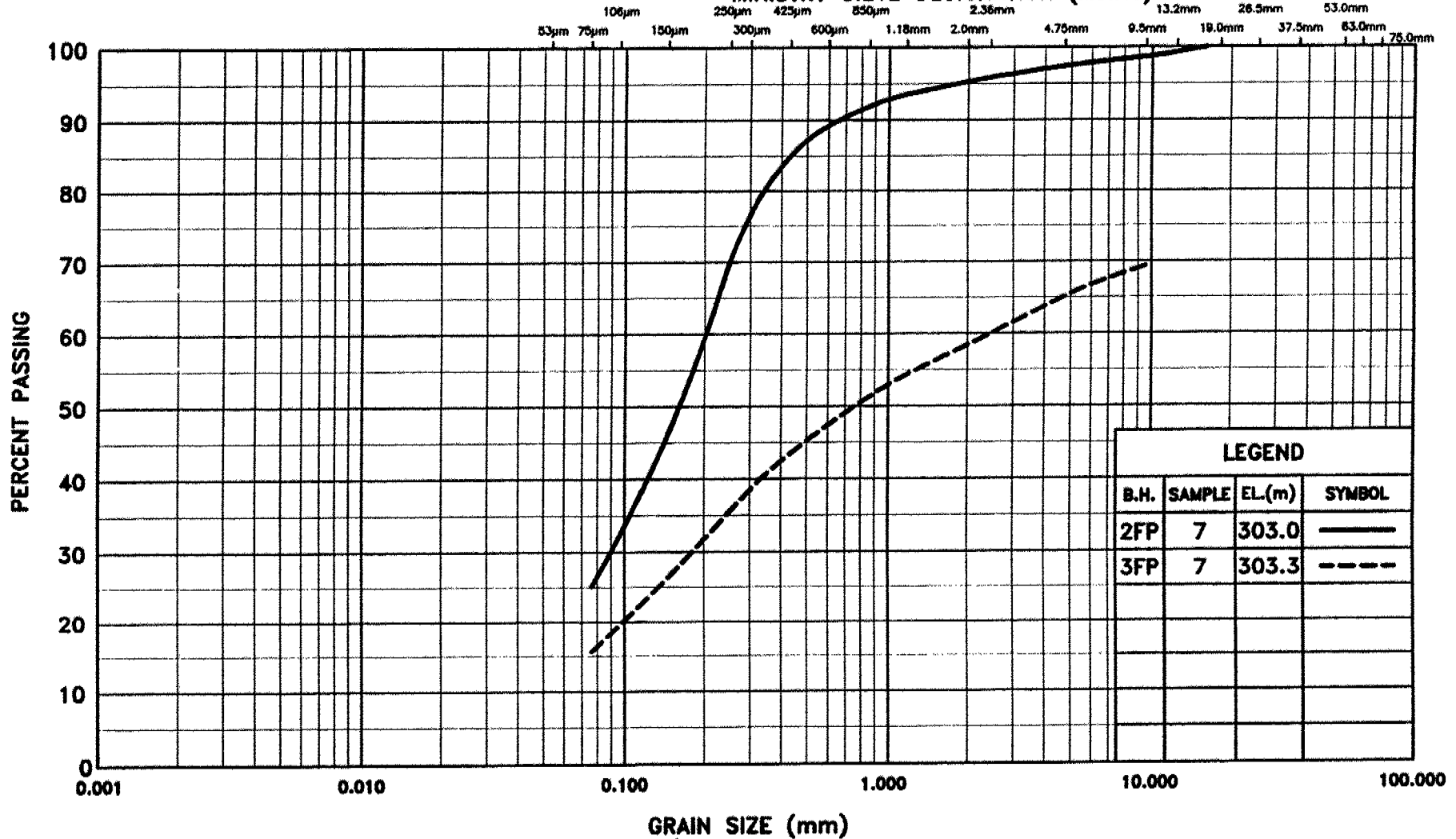


S07524GFP / F98179-A/G

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

### GRAIN SIZE DISTRIBUTION

BH-2FP, SS-7  
BH-3FP, SS-7

SAND  
SAND

FIGURE C5

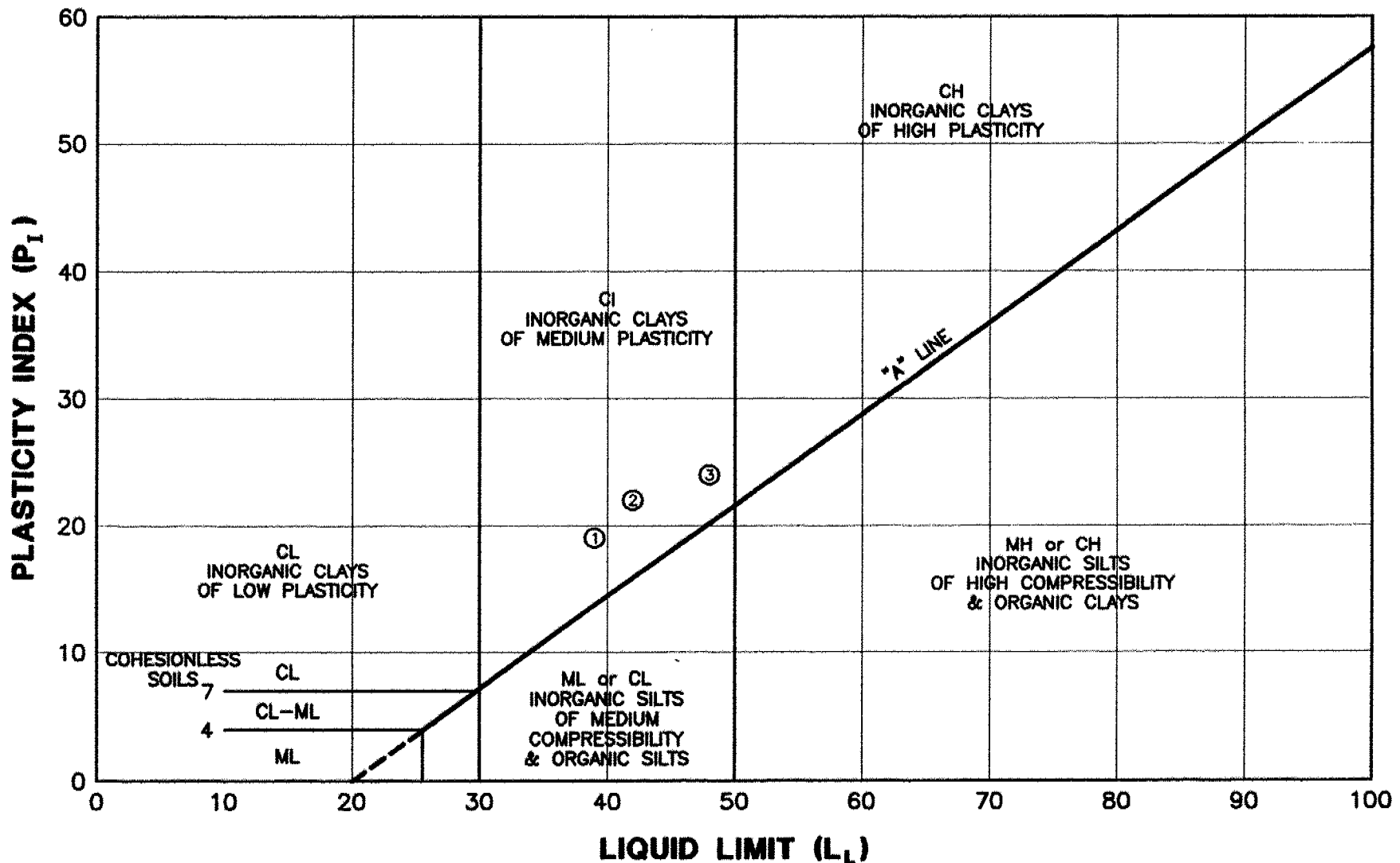
W.P. 774-93-00



S07524GFP / F98179-A/G

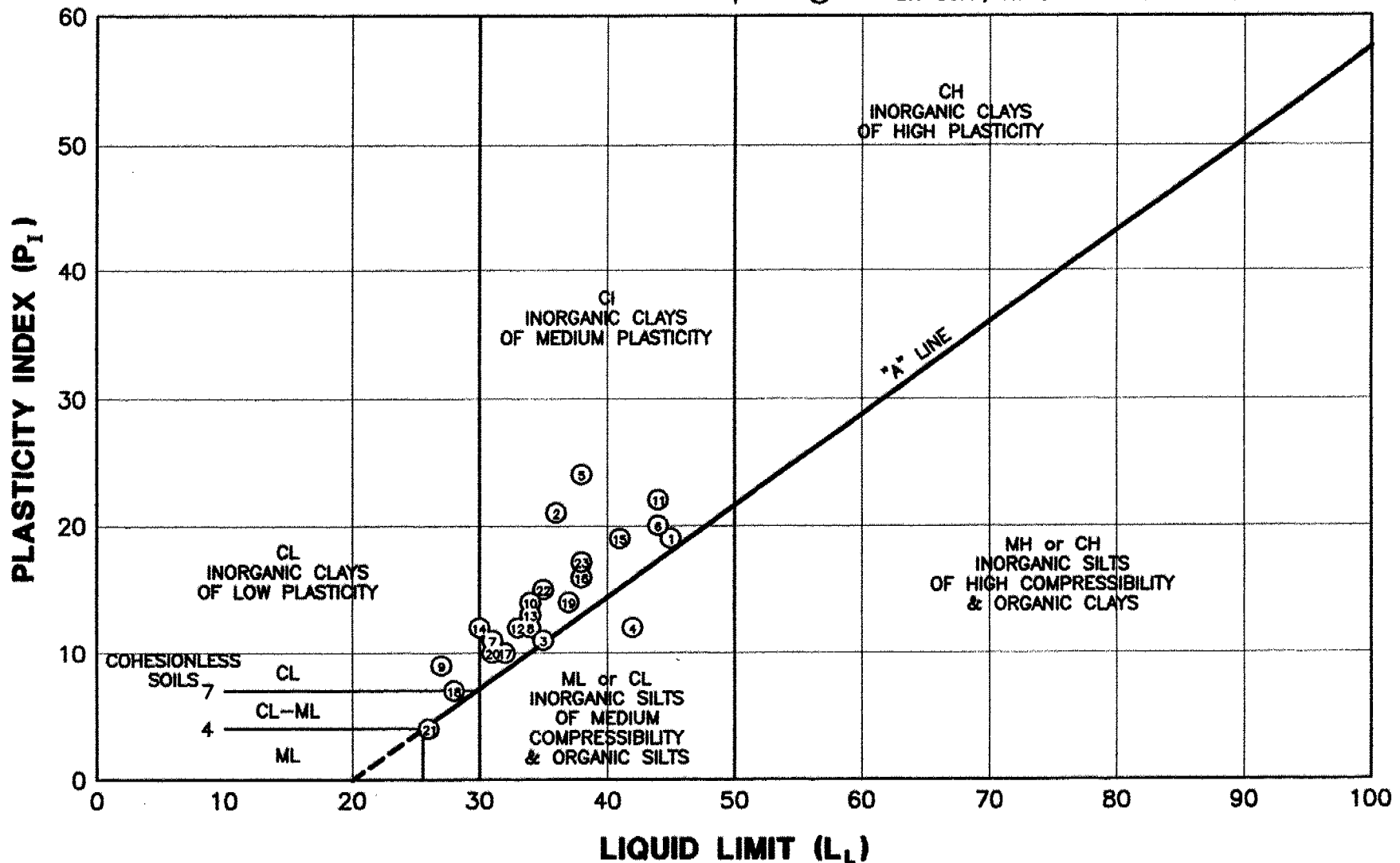
# **ATTERBERG LIMITS - PLASTICITY CHART**

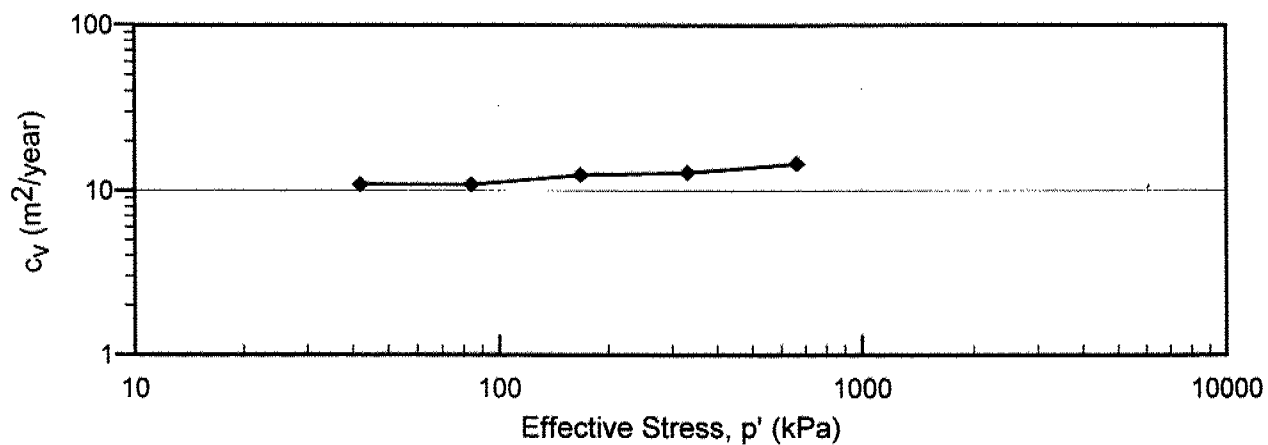
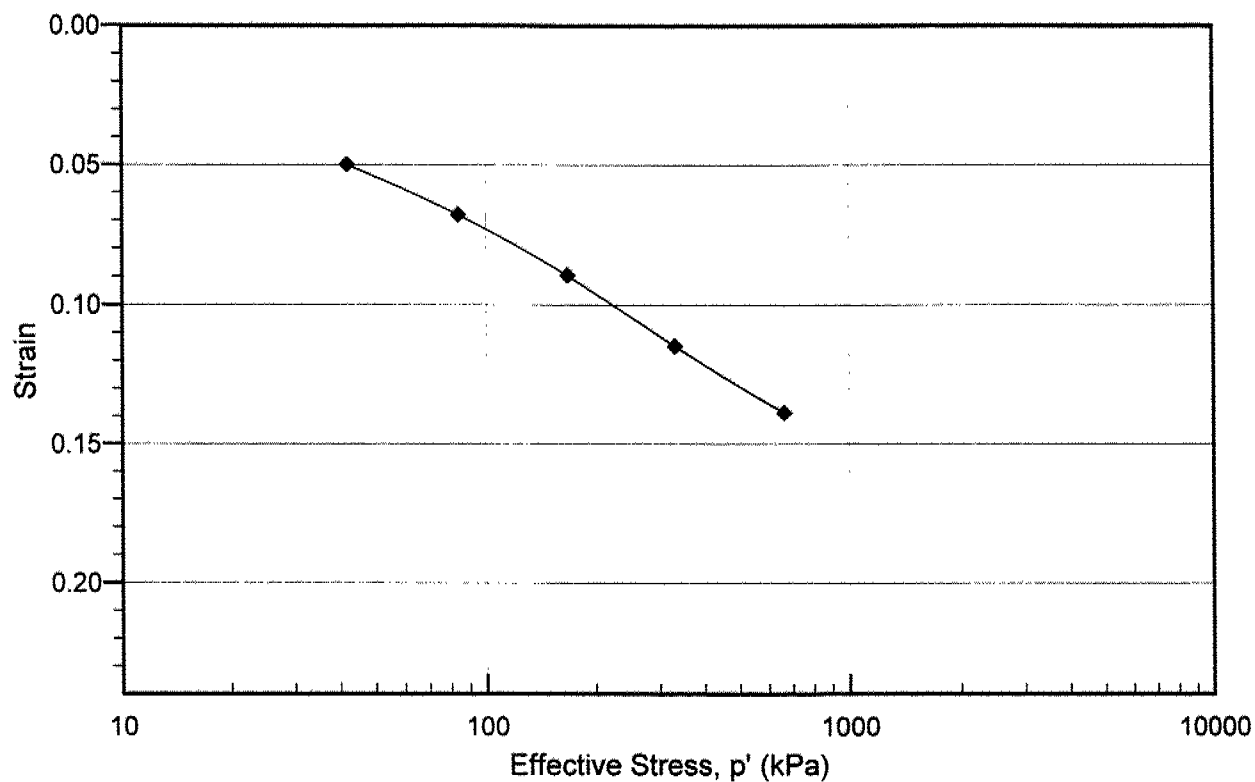
SYMBOL	DESCRIPTION	SOIL TYPE	LL	PI
①	BH-1FF, TW-6	CI	39	19
②	BH-2FF, TW-4	CI	42	22
③	BH-3FF, SS-3	CI	48	24



# **ATTERBERG LIMITS - PLASTICITY CHART**

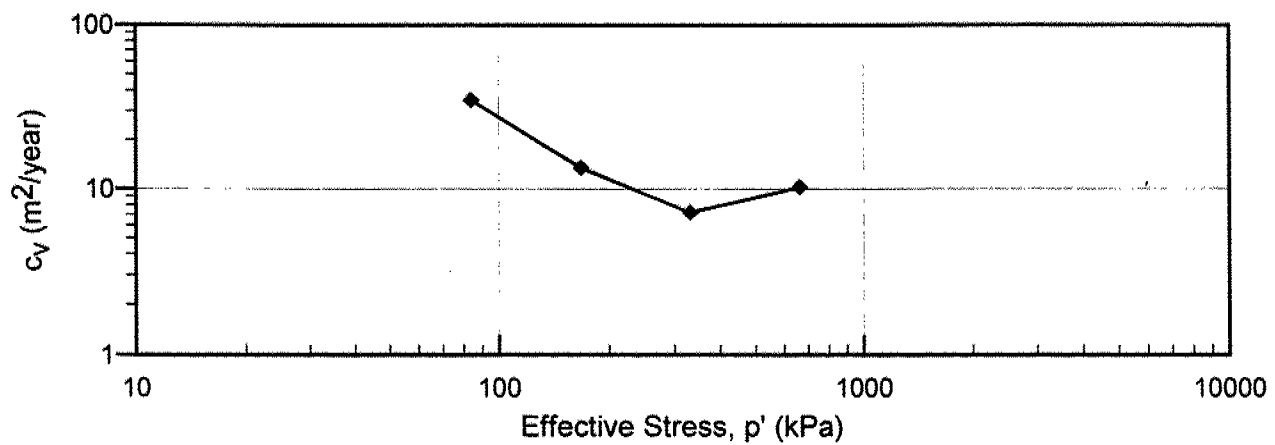
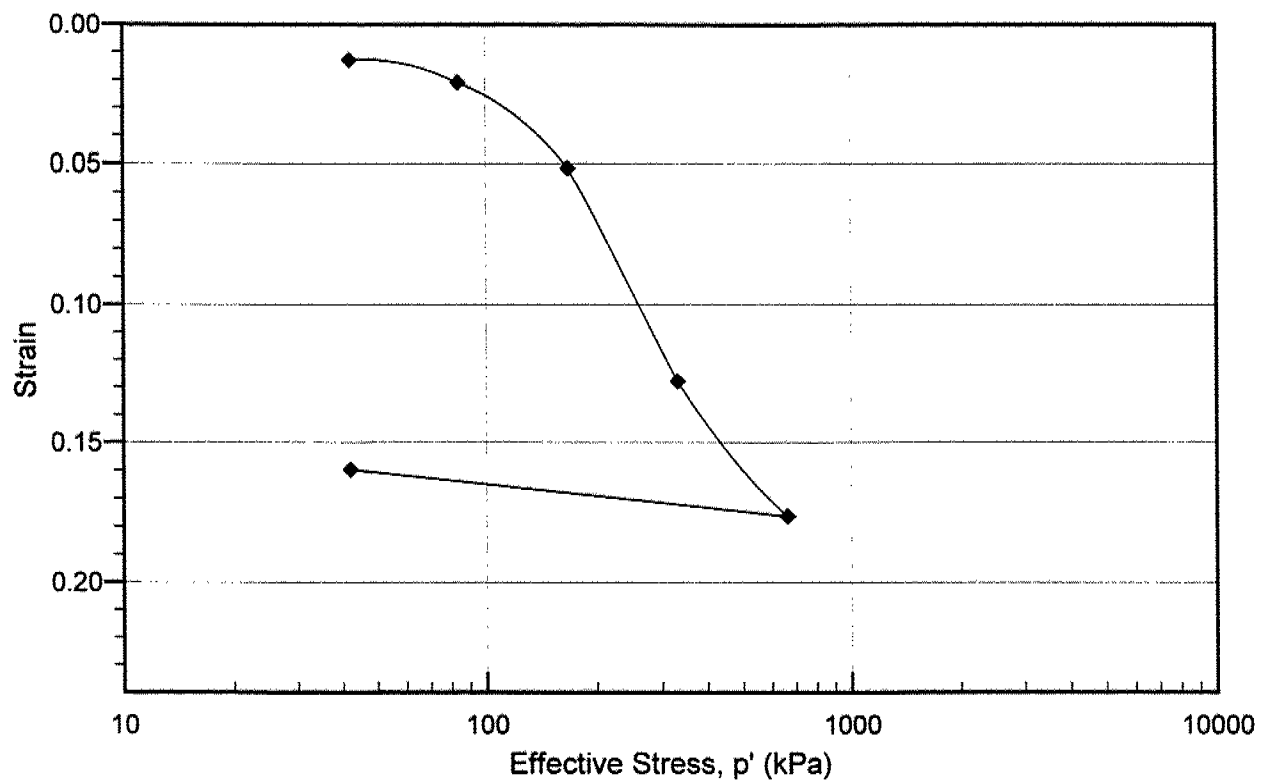
SYMBOL	DESCRIPTION	SOIL TYPE	LL	PI	SYMBOL	DESCRIPTION	SOIL TYPE	LL	PI
①	BH-1FP, SS-4	CI	45	19	⑫	BH-11FP, SS-5	CI	33	12
②	BH-2FP, SS-2	CI	36	21	⑬	BH-11FP, TW-6	CI	34	13
③	BH-3FP, TW-2	CI-CL	35	11	⑭	BH-16FP, SS-4	CI	30	12
④	BH-4FP, SS-2	CL-ML	42	12	⑮	BH-16FP, SS-6	CI	41	19
⑤	BH-7FP, SS-3	CI	38	24	⑯	BH-16FP, SS-7	CI	38	16
⑥	BH-8FP, SS-2	CI	44	20	⑰	BH-16FP, SS-9	CI	32	10
⑦	BH-8FP, SS-4	CI	31	11	⑱	BH-16FP, SS-10	CL-ML	28	7
⑧	BH-8FP, SS-5	CI	34	12	⑲	BH-23FP, SS-2	CI	37	14
⑨	BH-8FP, SS-6	CL	27	9	⑳	BH-23SP, SS-3	CI	31	10
⑩	BH-11FP, SS-2	CI	34	14	㉑	BH-23FP, SS-4	CL-ML	26	4
⑪	BH-11FP, SS-4	CI	44	22	㉒	BH-26FP, TW-6	CI	35	15
					㉓	BH-30FP, TW-5	CI	38	17





$$C_c/(1+e_0) = 0.08$$

$$\sigma'_{v0} = 45 \text{ kPa}$$



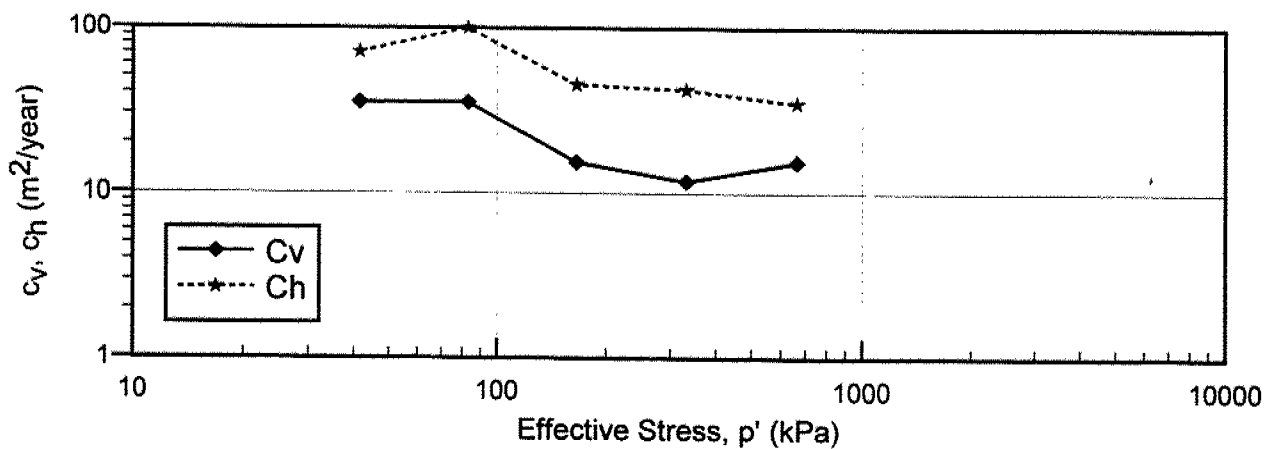
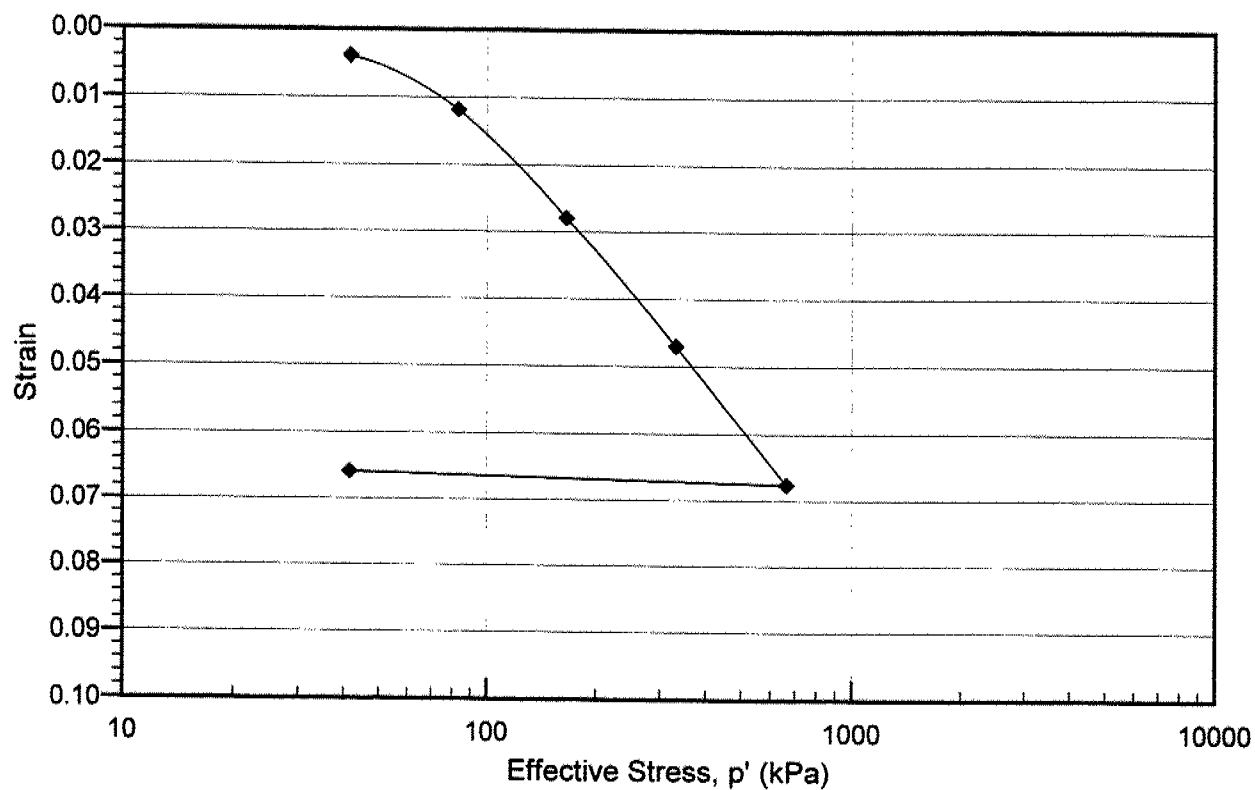
$$C_c/(1+e_0) = 0.20$$

$$C_r/(1+e_0) = 0.015$$

$$\sigma'_{v0} = 35 \text{ kPa}$$

$$\sigma'_{pc} = 110 \text{ kPa}$$



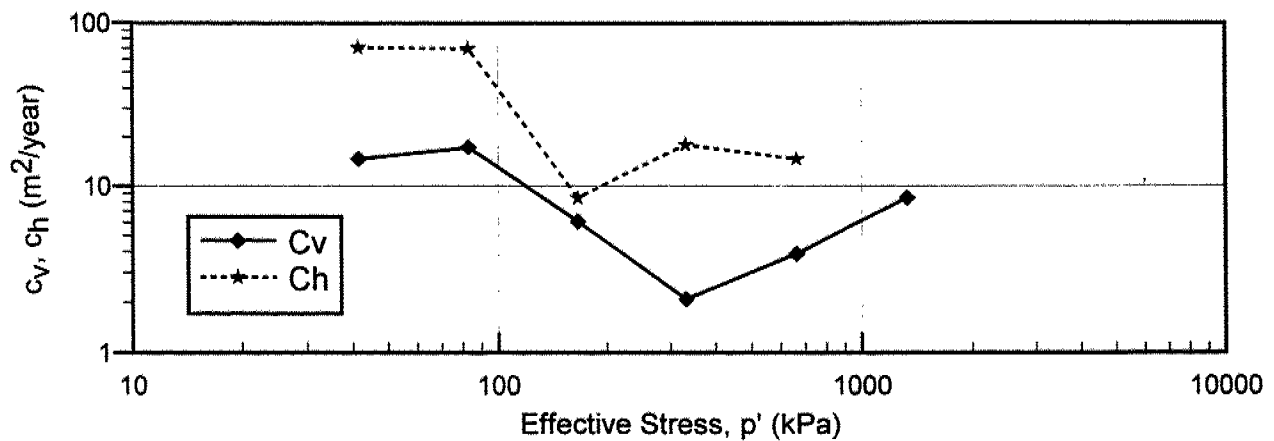
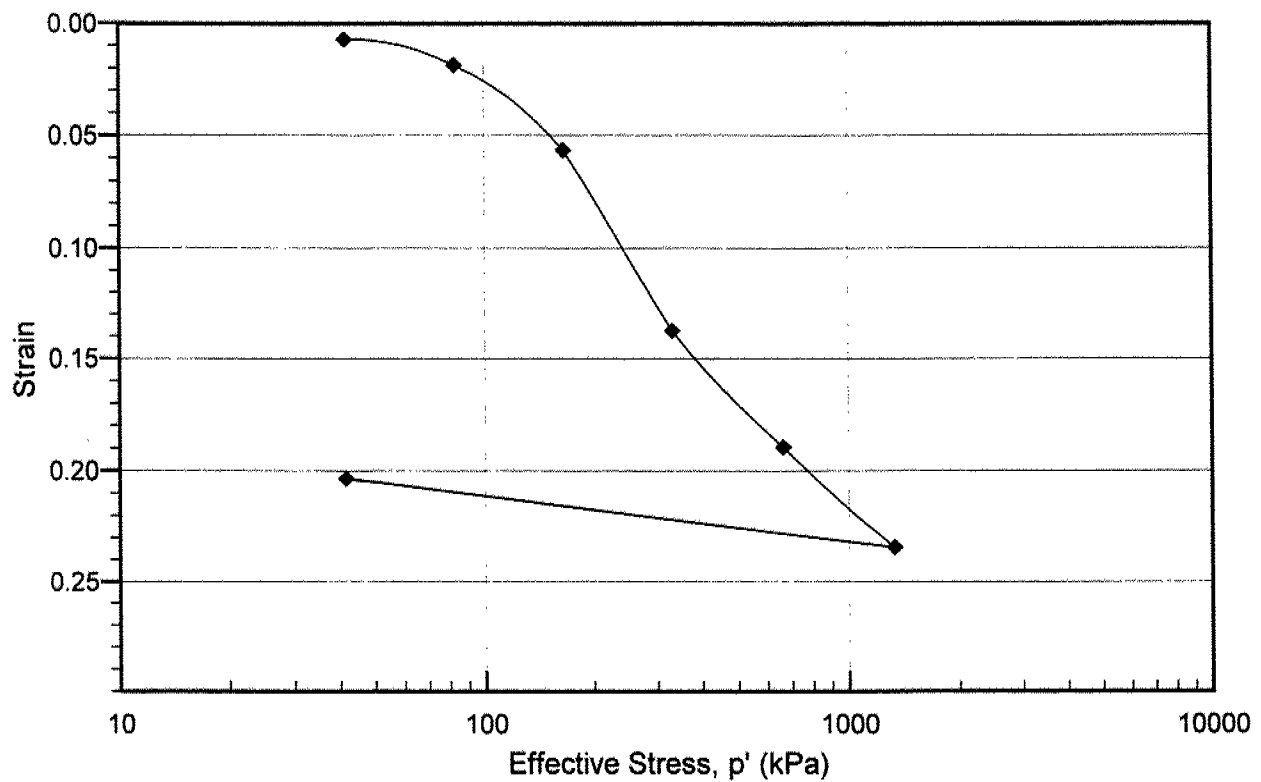


$$C_c/(1+e_0) = 0.07$$

$$C_r/(1+e_0) = 0.002$$

$$\sigma'_{vo} = 76 \text{ kPa}$$

$$\sigma'_{pc} = 90 \text{ kPa}$$

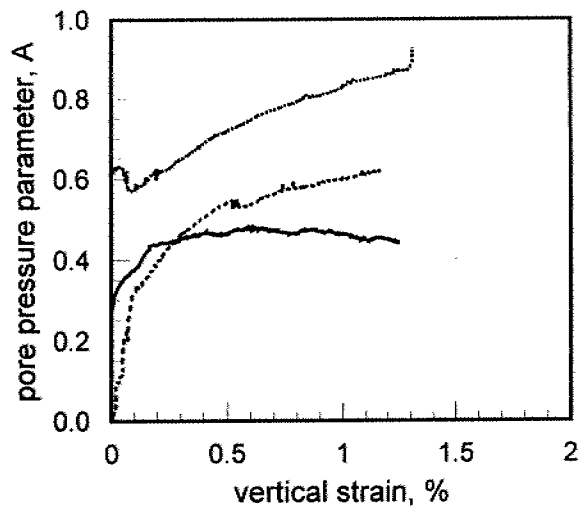
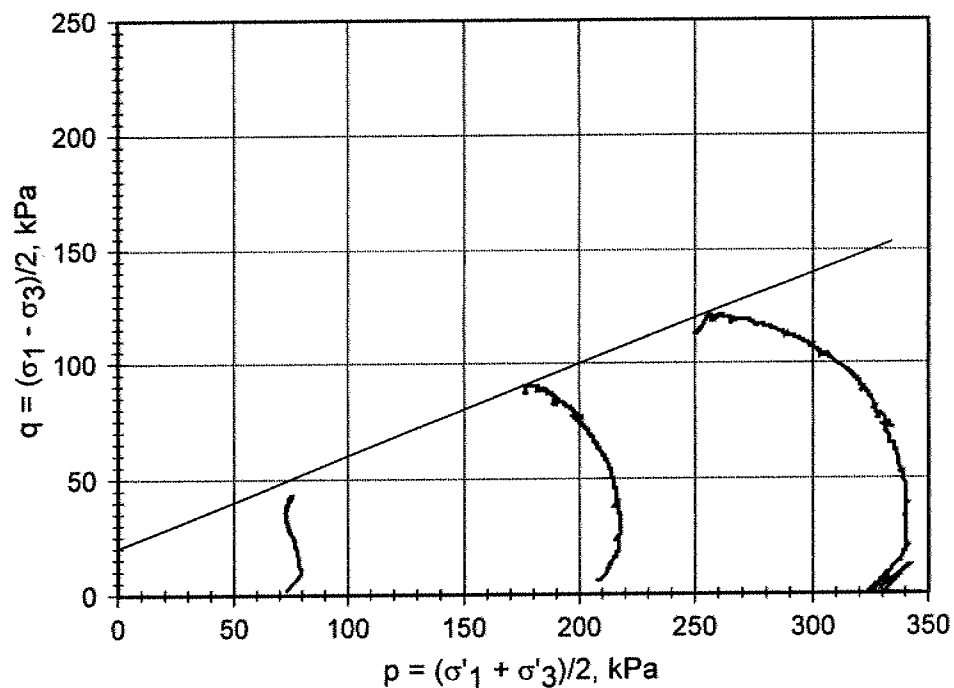
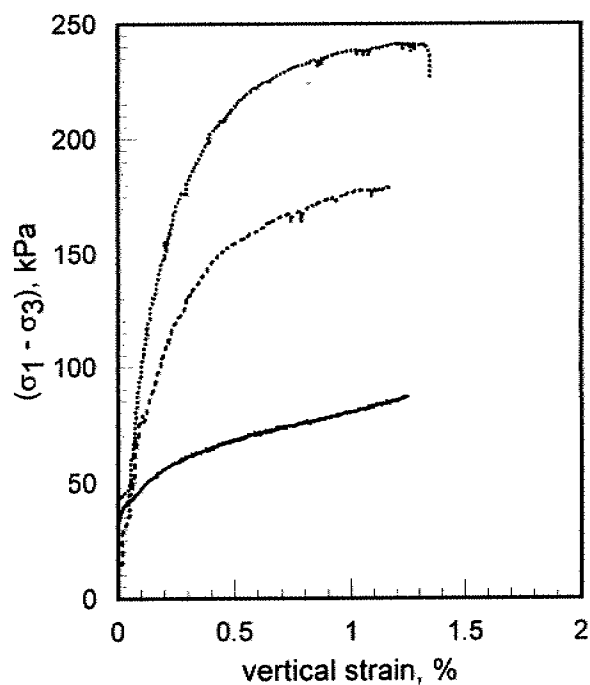


$$C_c/(1+e_0) = 0.21$$

$$C_r/(1+e_0) = 0.035$$

$$\sigma'_{v0} = 65 \text{ kPa}$$

$$\sigma'_{pc} = 90 - 100 \text{ kPa}$$



Trow Consulting Engineers Ltd.  
Thunder Bay, Ont.

Client: Marshall Macklin Monaghan

## Triaxial Test Data 1

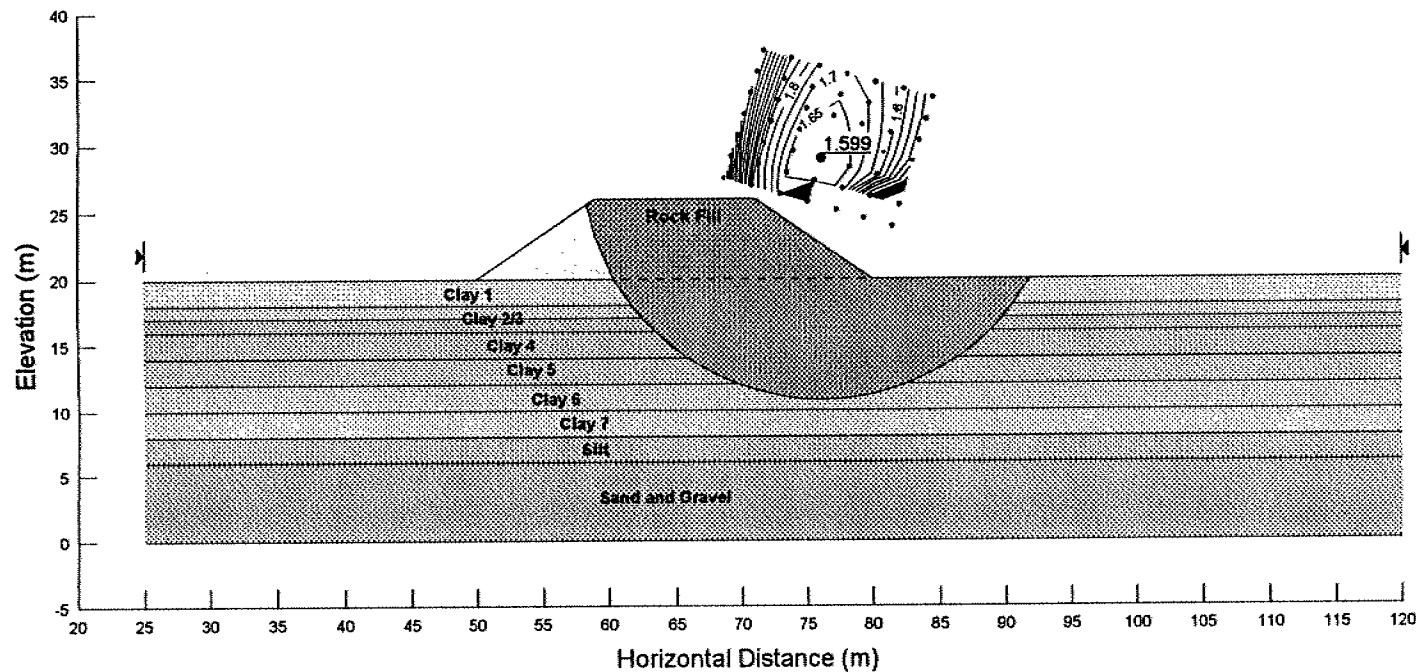
Trout Creek Project - Trout Creek

F98179-A/G

Aug. 24/98

Figure C12

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 6 metre embankment height, 1.5:1 overall side slope  
 12 metre clay foundation  
 H06T12CU.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Clay 5  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

Soil 7 - Clay 6  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 27.5  
 Rate of Increase 2.5  
 Cv - Maximum 32.5  
 Ch/Cv Ratio 1

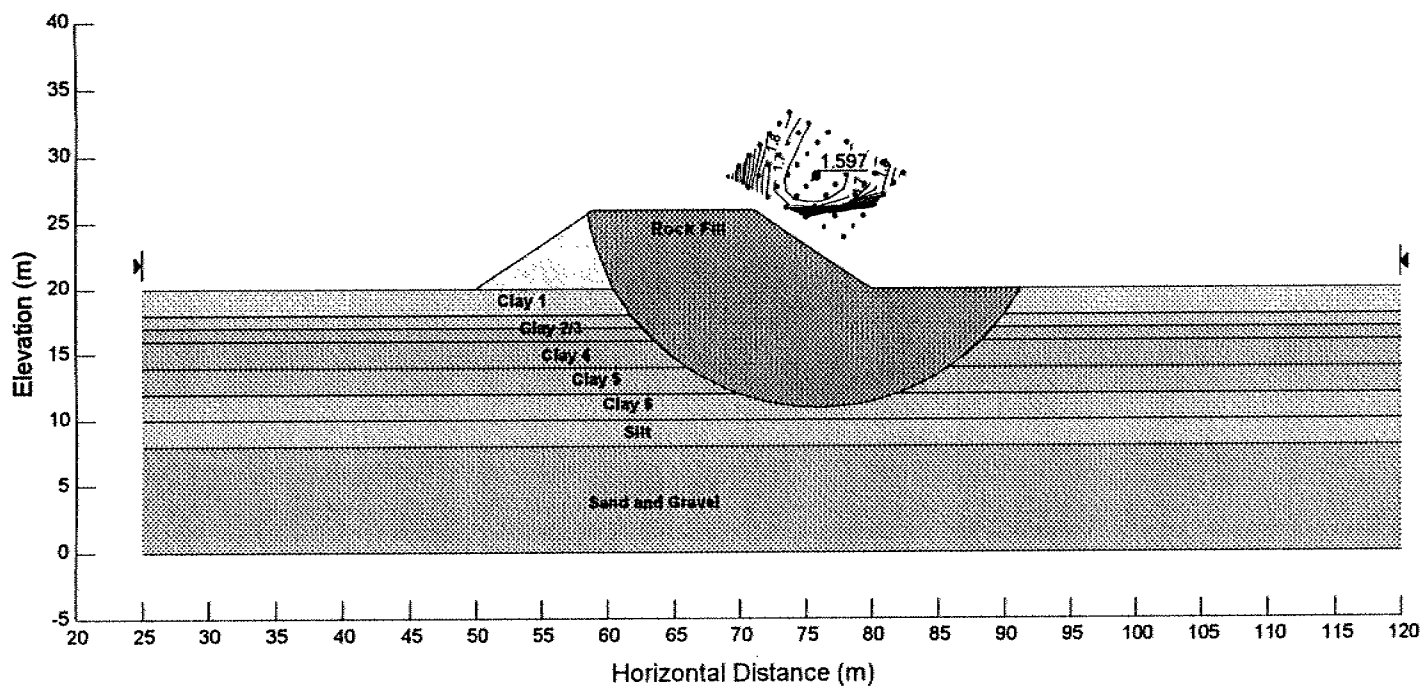
Soil 8 - Clay 7  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 32.5  
 Rate of Increase 2.5  
 Cv - Maximum 37.5  
 Ch/Cv Ratio 1

Soil 9 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 10 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 11 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 6 metre embankment height, 1.5:1 overall side slope  
 10 metre clay foundation  
 H06T10CU.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Clay 5  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

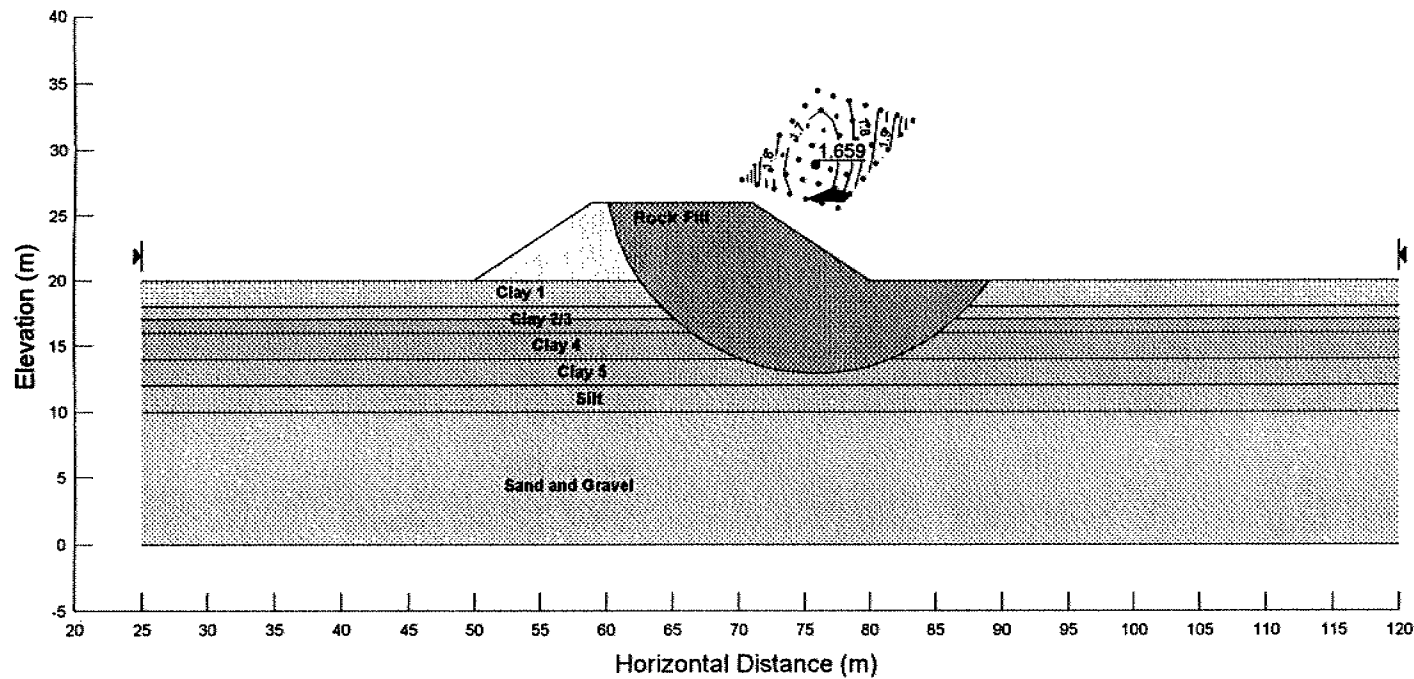
Soil 7 - Clay 6  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 27.5  
 Rate of Increase 2.5  
 Cv - Maximum 32.5  
 Ch/Cv Ratio 1

Soil 8 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 9 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 10 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 6 metre embankment height, 1.5:1 overall side slope  
 8 metre clay foundation  
 H06T08CU.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

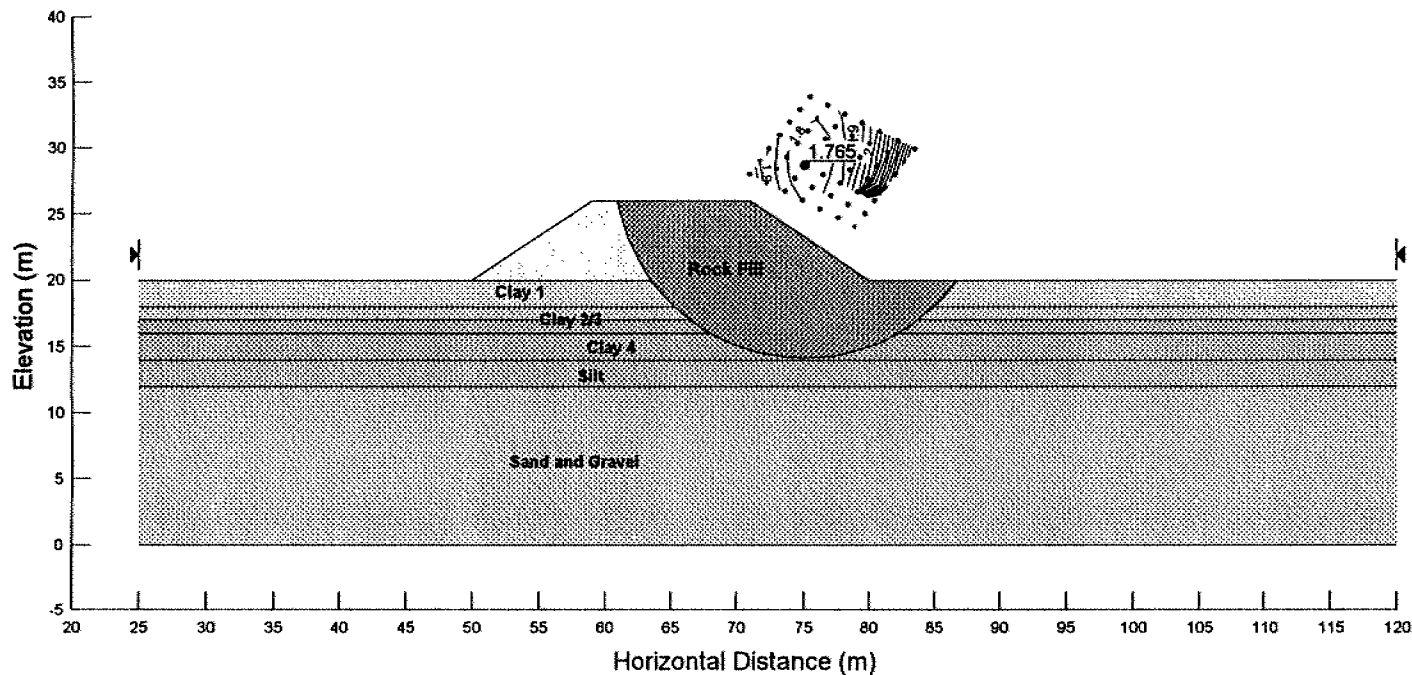
Soil 6 - Clay 5  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

Soil 7 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 8 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 9 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 6 metre embankment height, 1.5:1 overall side slope  
 6 metre clay foundation  
 H06T06CU.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 7 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 8 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 6 metre embankment height, 1.5:1 overall side slope  
 4 metre clay foundation  
 H06T04CU.SLP

Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

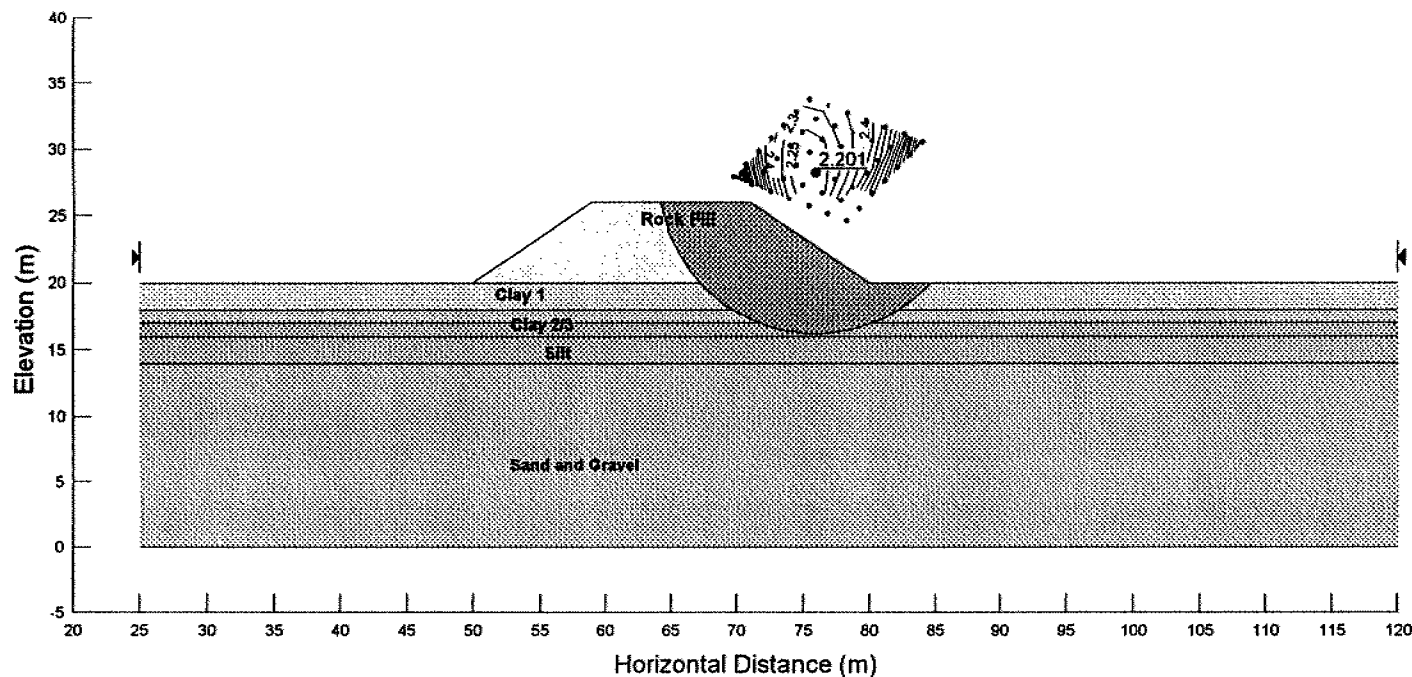
Soil 3 - Clay 2  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

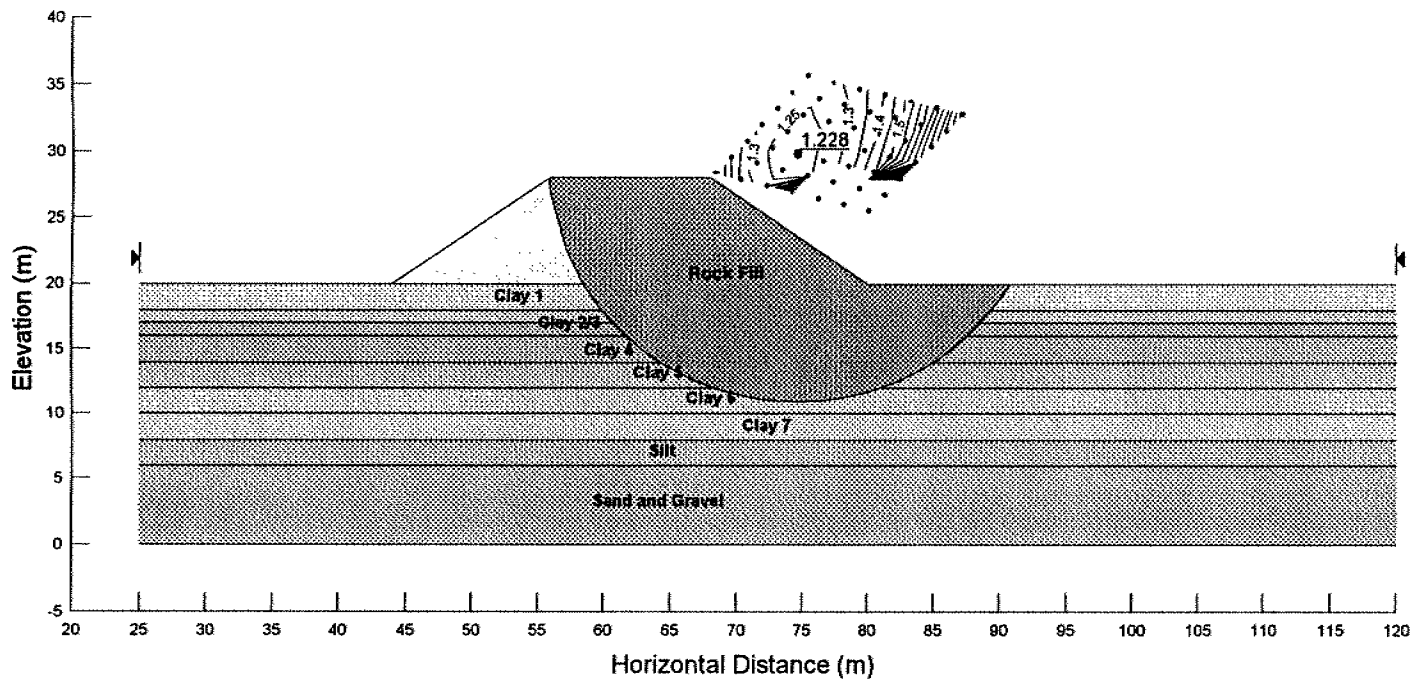
Soil 6 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 7 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1





Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 8 metre embankment height, 1.5:1 overall side slope  
 12 metre clay foundation  
 H08T12CU.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Clay 5  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

Soil 7 - Clay 6  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 27.5  
 Rate of Increase 2.5  
 Cv - Maximum 32.5  
 Ch/Cv Ratio 1

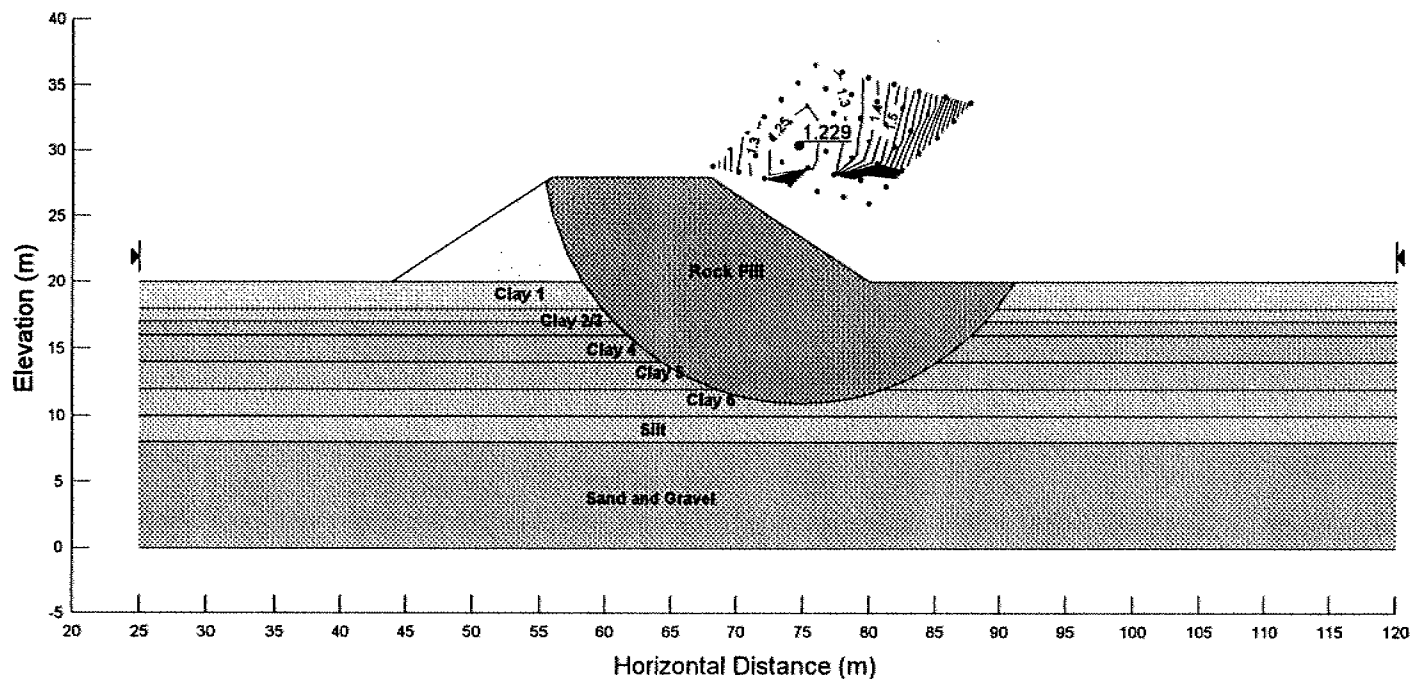
Soil 8 - Clay 7  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 32.5  
 Rate of Increase 2.5  
 Cv - Maximum 37.5  
 Ch/Cv Ratio 1

Soil 9 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 10 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 11 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 8 metre embankment height, 1.5:1 overall side slope  
 10 metre clay foundation  
 H8T10CU.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Clay 5  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

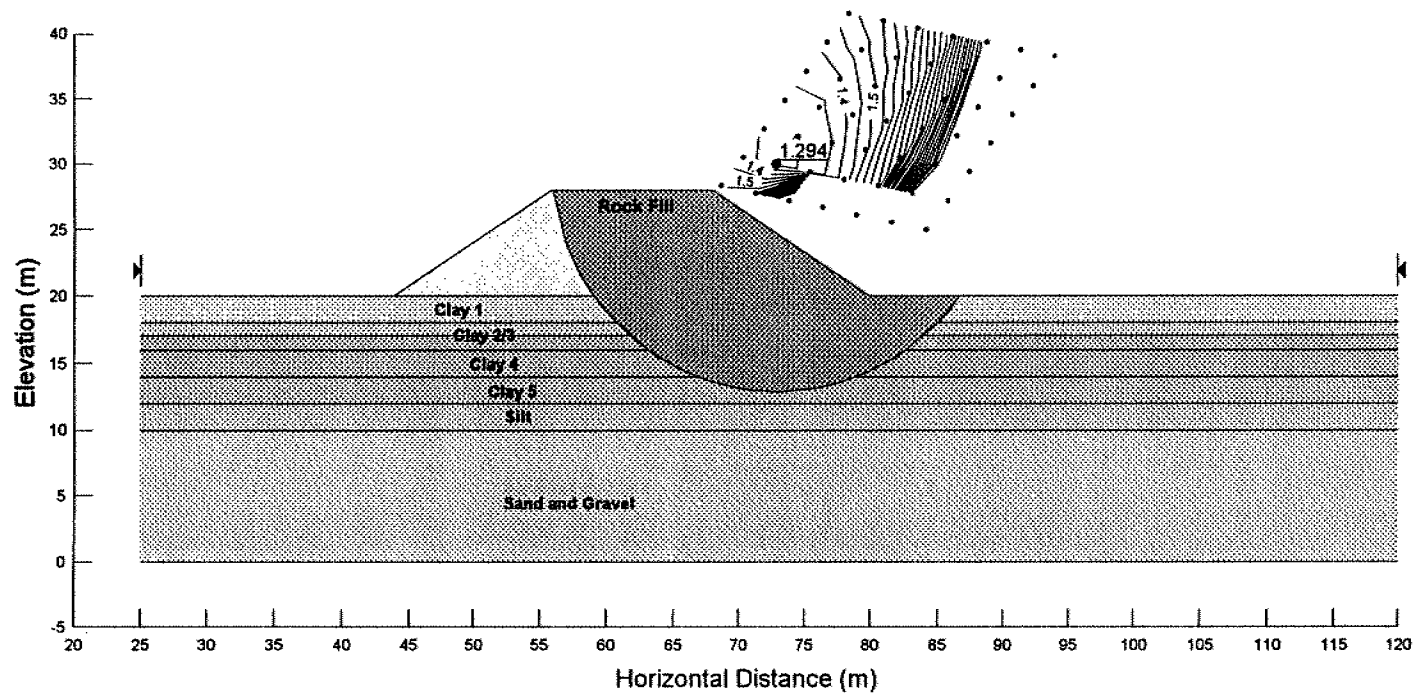
Soil 7 - Clay 6  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 27.5  
 Rate of Increase 2.5  
 Cv - Maximum 32.5  
 Ch/Cv Ratio 1

Soil 8 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 9 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 10 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 8 metre embankment height, 1.5:1 overall side slope  
 8 metre clay foundation  
 H08T08CU.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

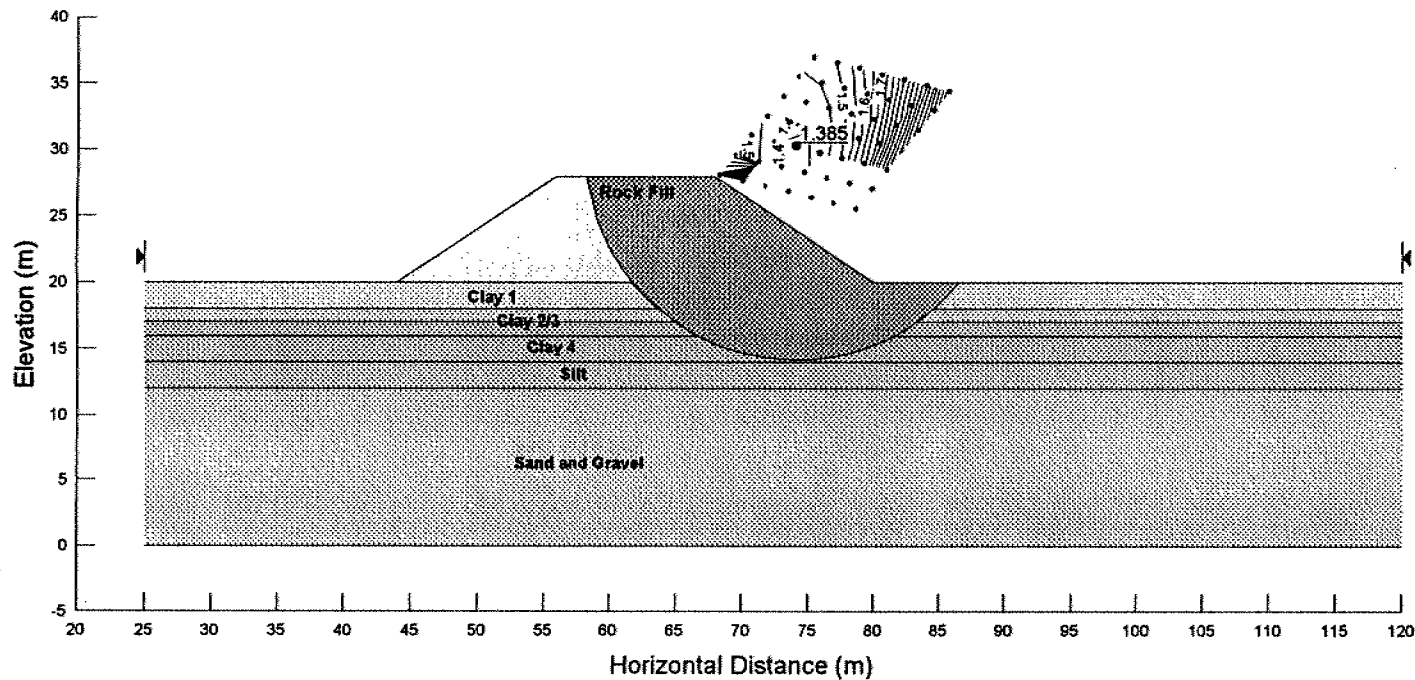
Soil 6 - Clay 5  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

Soil 7 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 8 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 9 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 8 metre embankment height, 1.5:1 overall side slope  
 6 metre clay foundation  
 H08T06CU.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

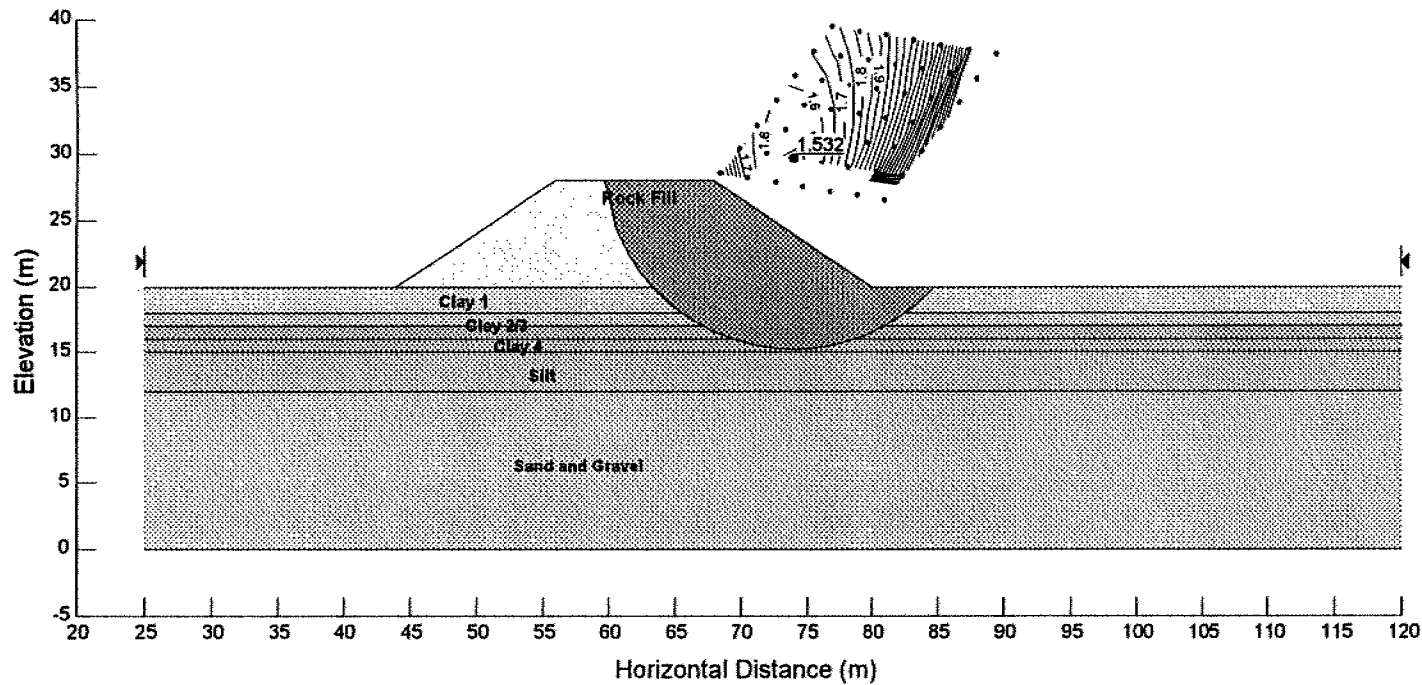
Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 7 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 8 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 8 metre embankment height, 1.5:1 overall side slope  
 5 metre clay foundation  
 H08T05CU.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

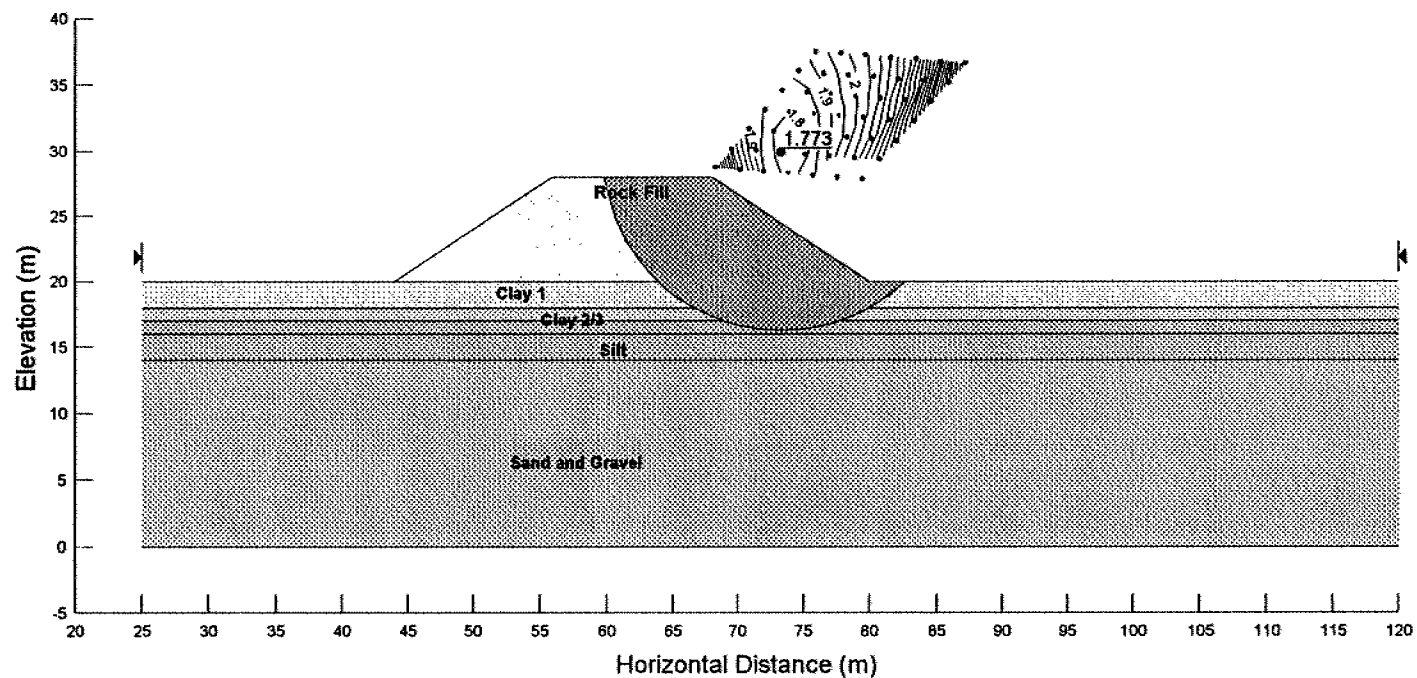
Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 7 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 8 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 8 metre embankment height, 1.5:1 overall side slope  
 4 metre clay foundation  
 H08T04CU.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

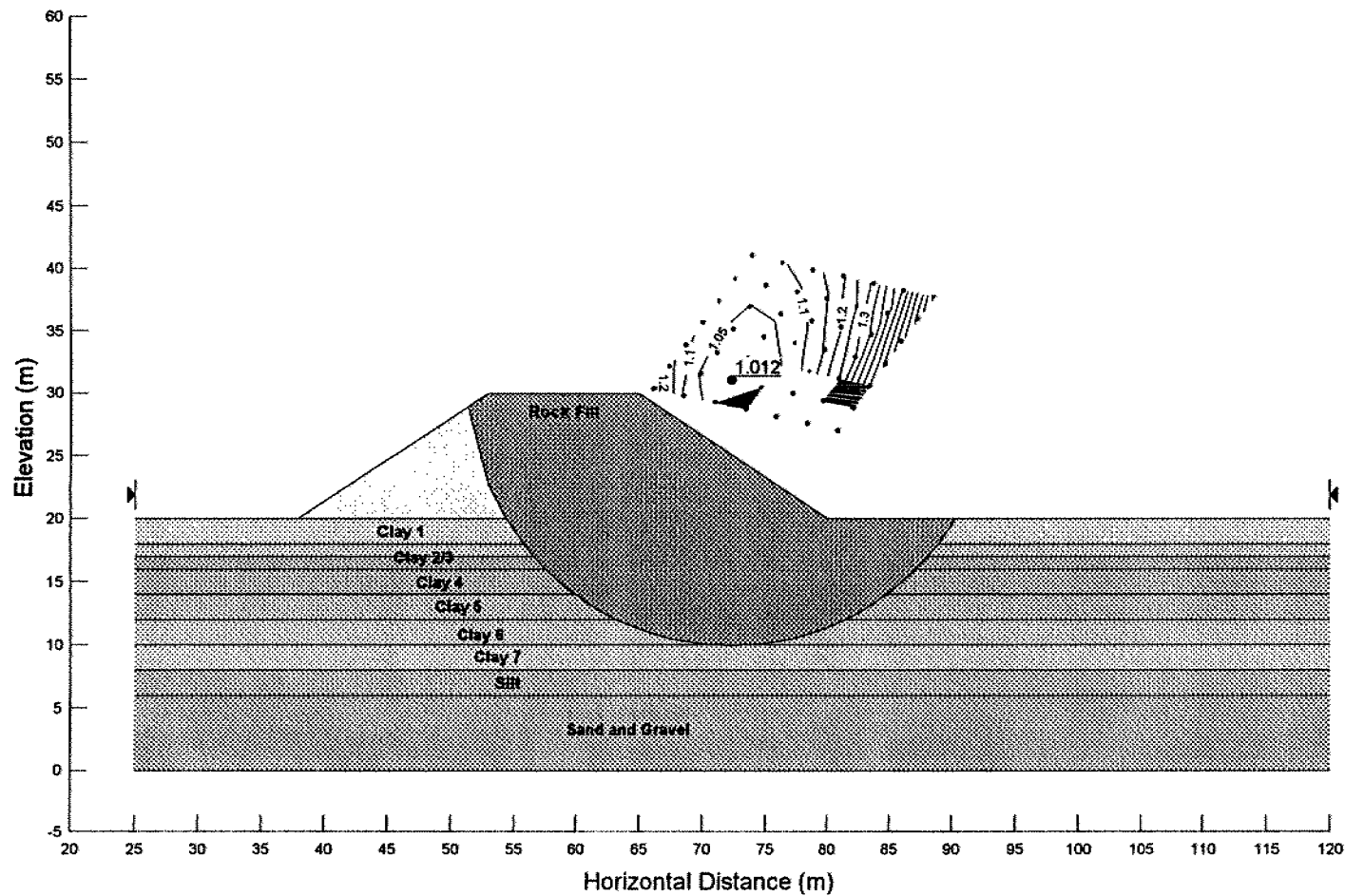
Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 6 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 7 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 10 metre embankment height, 1.5:1 overall side slope  
 12 metre clay foundation  
 H10T12CU.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Clay 5  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

Soil 7 - Clay 6  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 27.5  
 Rate of Increase 2.5  
 Cv - Maximum 32.5  
 Ch/Cv Ratio 1

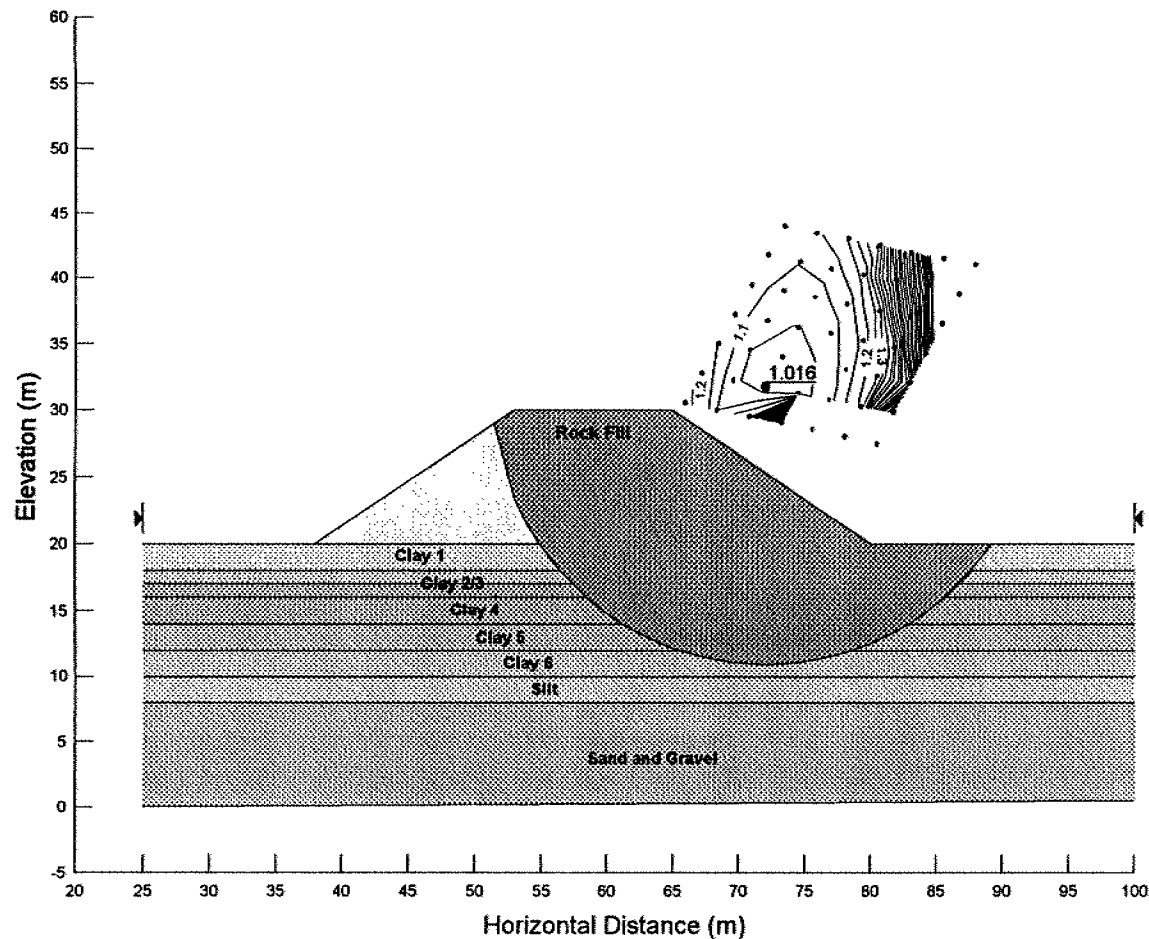
Soil 8 - Clay 7  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 32.5  
 Rate of Increase 2.5  
 Cv - Maximum 37.5  
 Ch/Cv Ratio 1

Soil 9 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 10 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 11 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 10 metre embankment height, 1.5:1 overall side slope  
 10 metre clay foundation  
 H10T10CU.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Clay 5  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

Soil 7 - Clay 6  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 27.5  
 Rate of Increase 2.5  
 Cv - Maximum 32.5  
 Ch/Cv Ratio 1

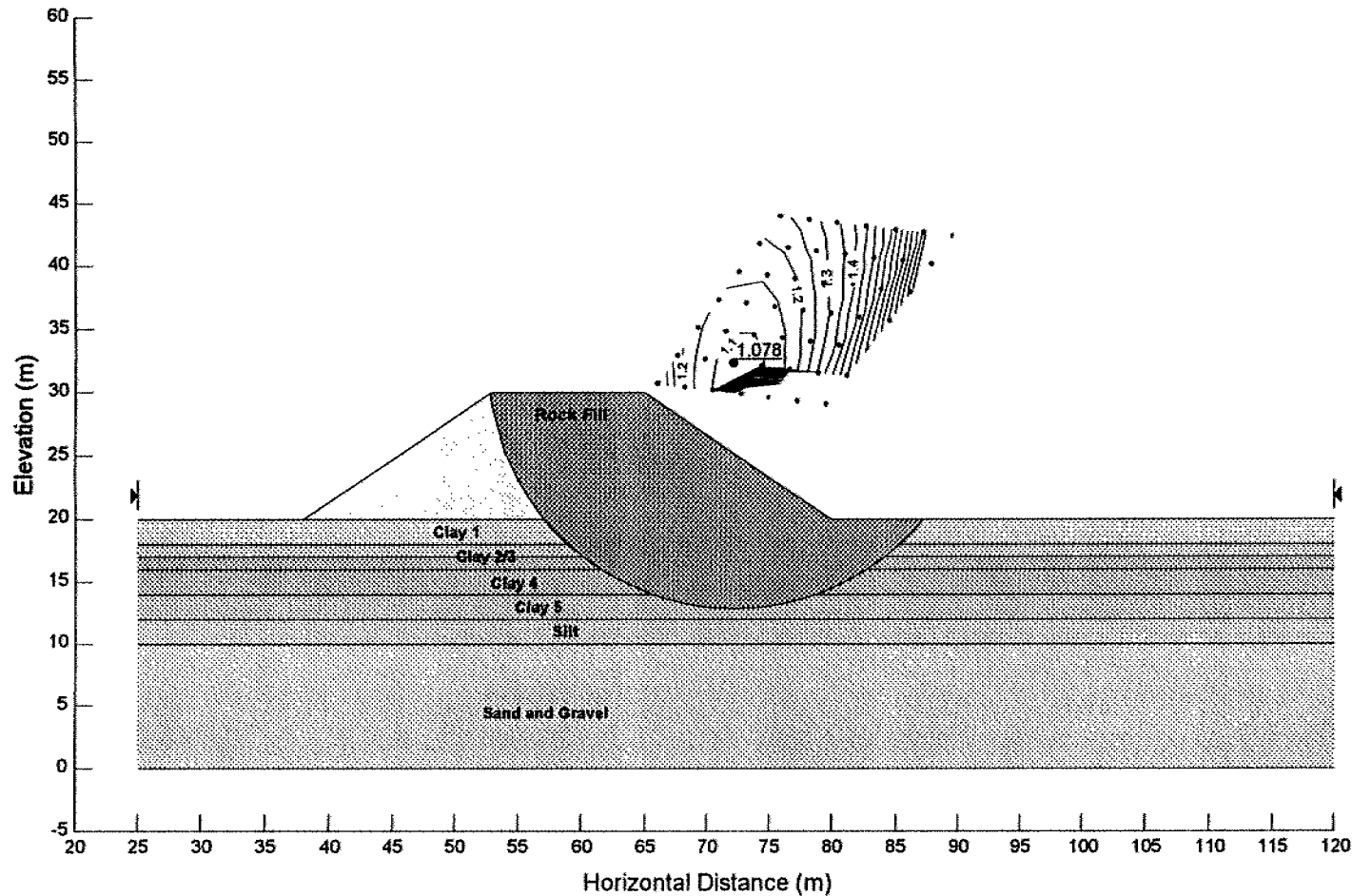
Soil 8 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 9 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 10 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1



Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 10 metre embankment height, 1.5:1 overall side slope  
 8 metre clay foundation  
 H10T08CU.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

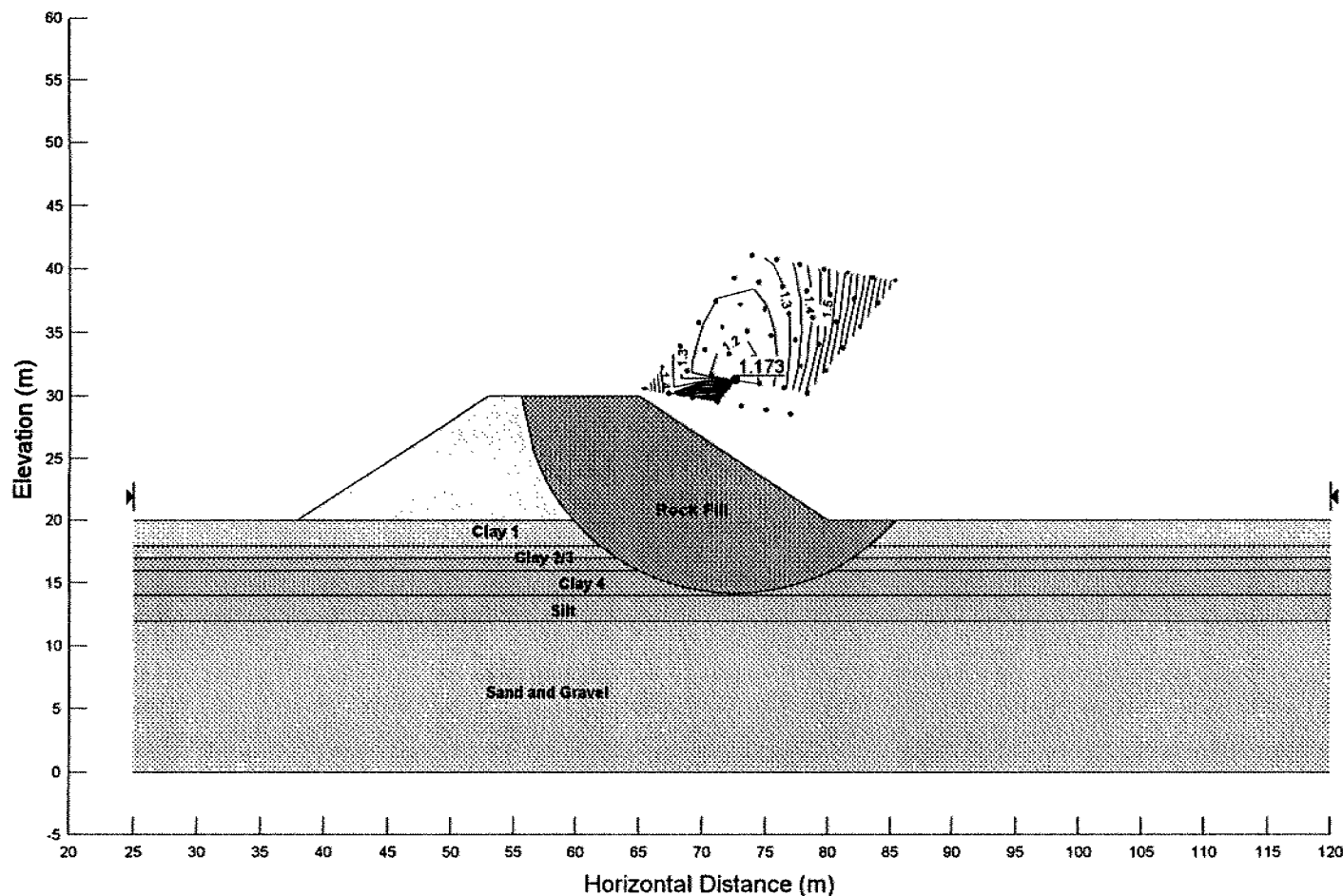
Soil 6 - Clay 5  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

Soil 7 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 8 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 9 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 10 metre embankment height, 1.5:1 overall side slope  
 6 metre clay foundation  
 H10T06CU.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

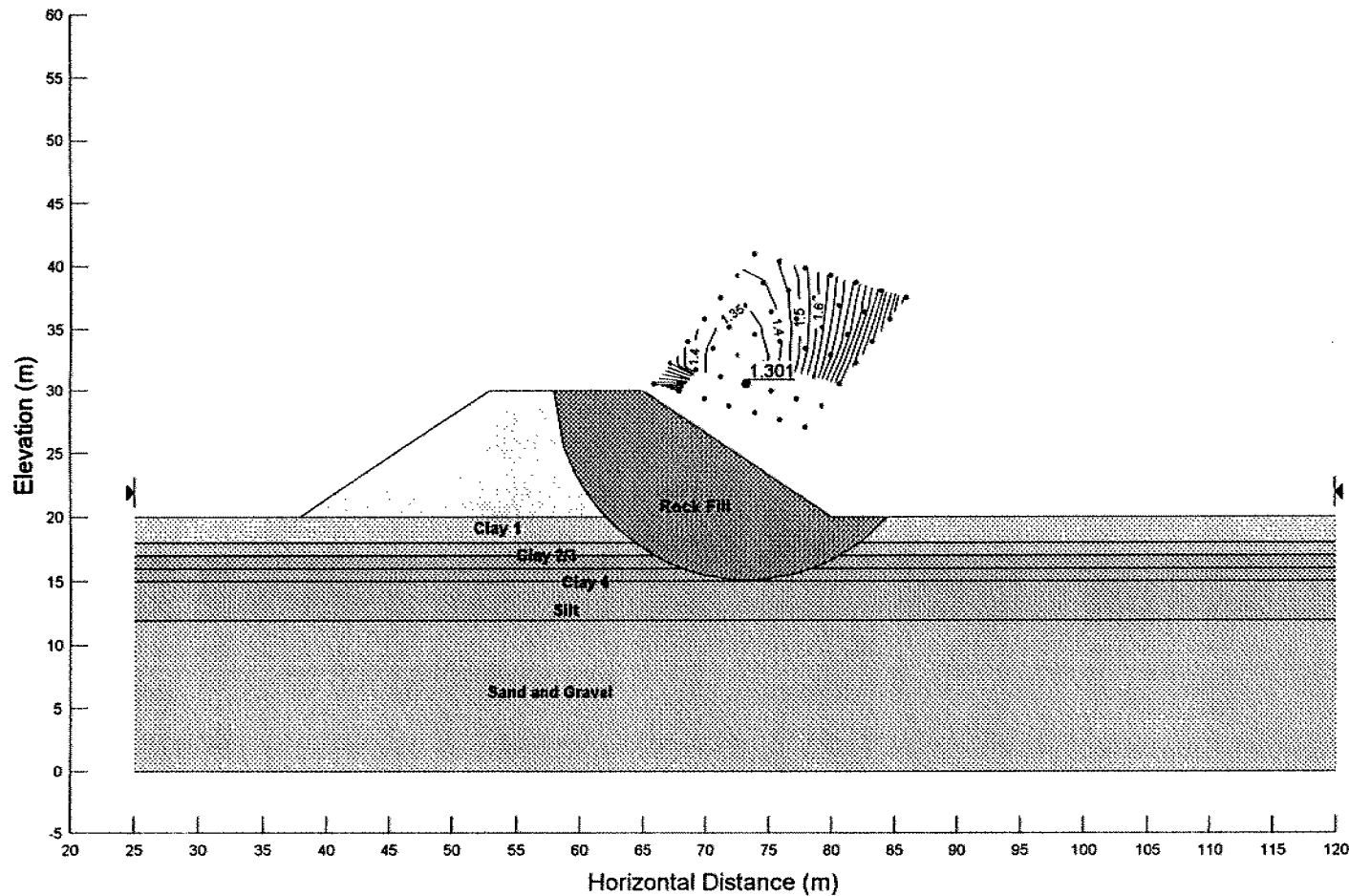
Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 7 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 8 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 10 metre embankment height, 1.5:1 overall side slope  
 5 metre clay foundation  
 H10T05CU.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

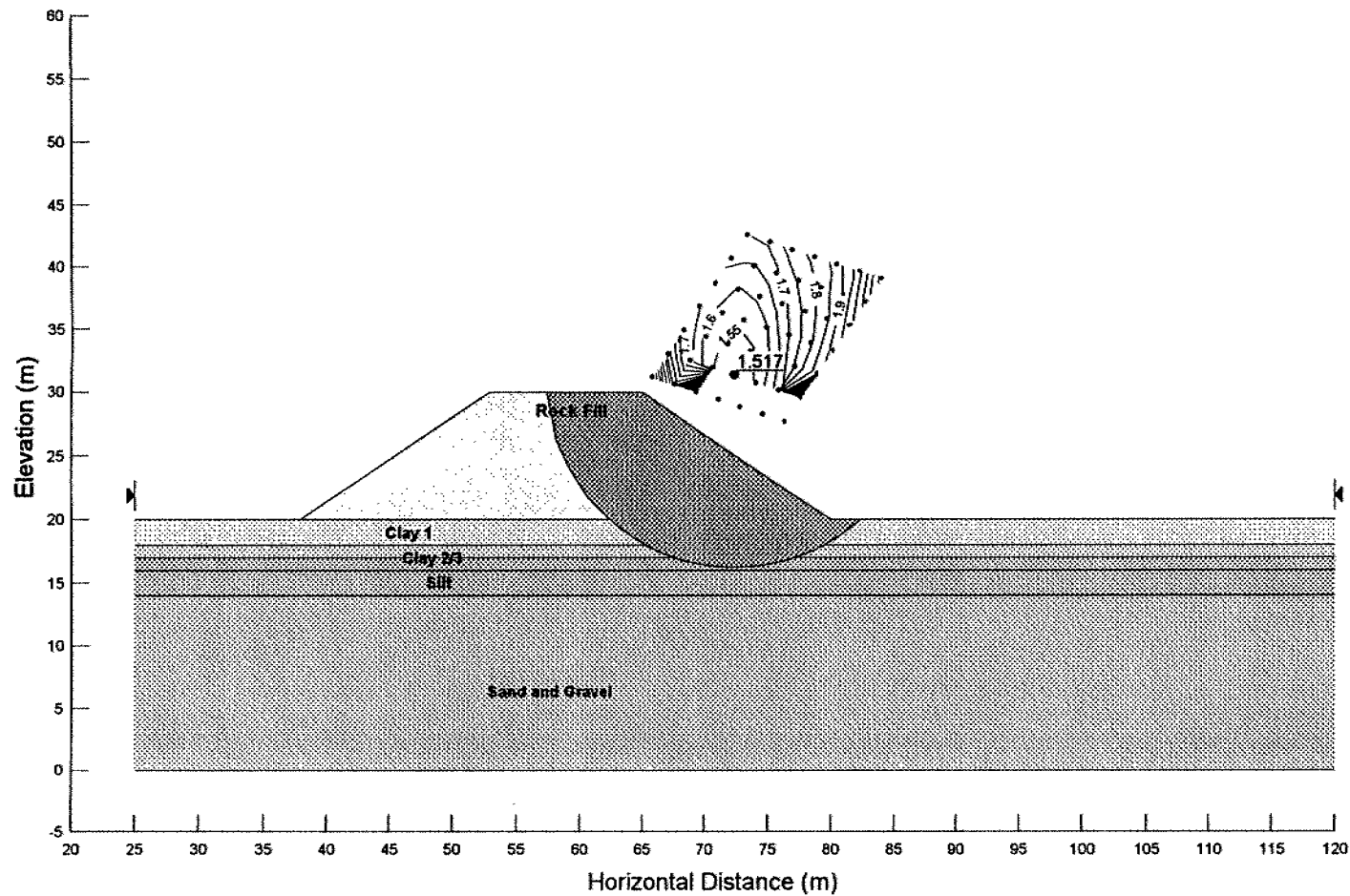
Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 7 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 8 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 10 metre embankment height, 1.5:1 overall side slope  
 4 metre clay foundation  
 H10T04CU.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

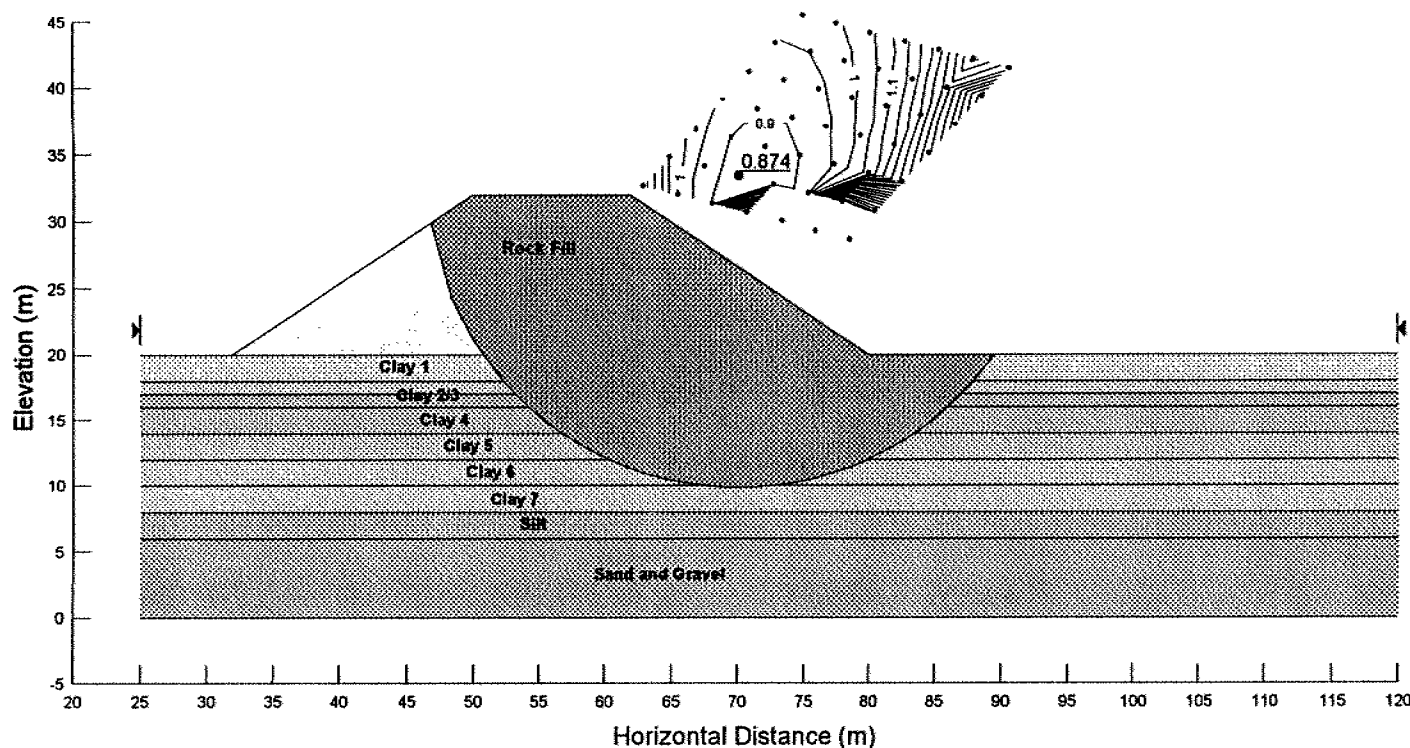
Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 6 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 7 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 12 metre embankment height, 1.5:1 overall side slope  
 12 metre clay foundation  
 H12T12CU.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Clay 5  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

Soil 7 - Clay 6  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 27.5  
 Rate of Increase 2.5  
 Cv - Maximum 32.5  
 Ch/Cv Ratio 1

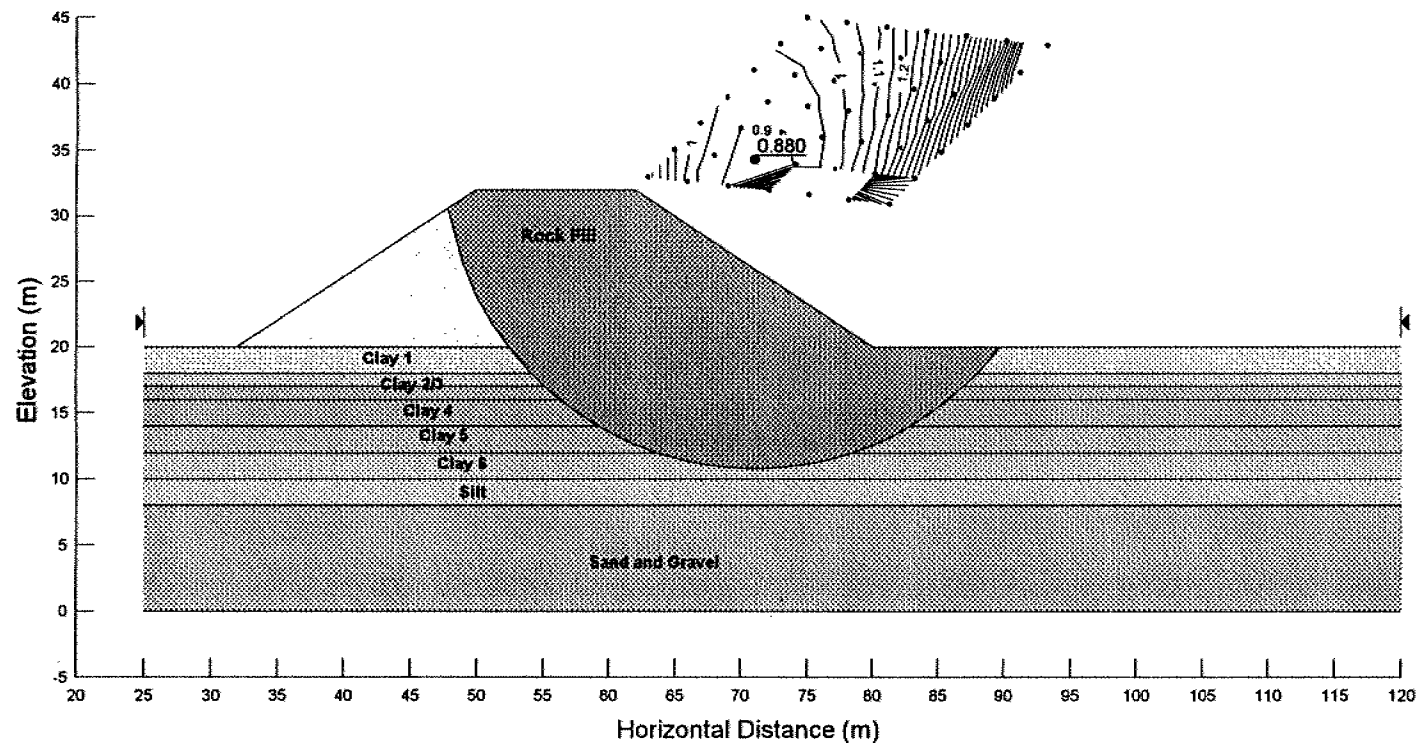
Soil 8 - Clay 7  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 32.5  
 Rate of Increase 2.5  
 Cv - Maximum 37.5  
 Ch/Cv Ratio 1

Soil 9 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 10 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 11 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 12 metre embankment height, 1.5:1 overall side slope  
 10 metre clay foundation  
 H12T10CU.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Clay 5  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

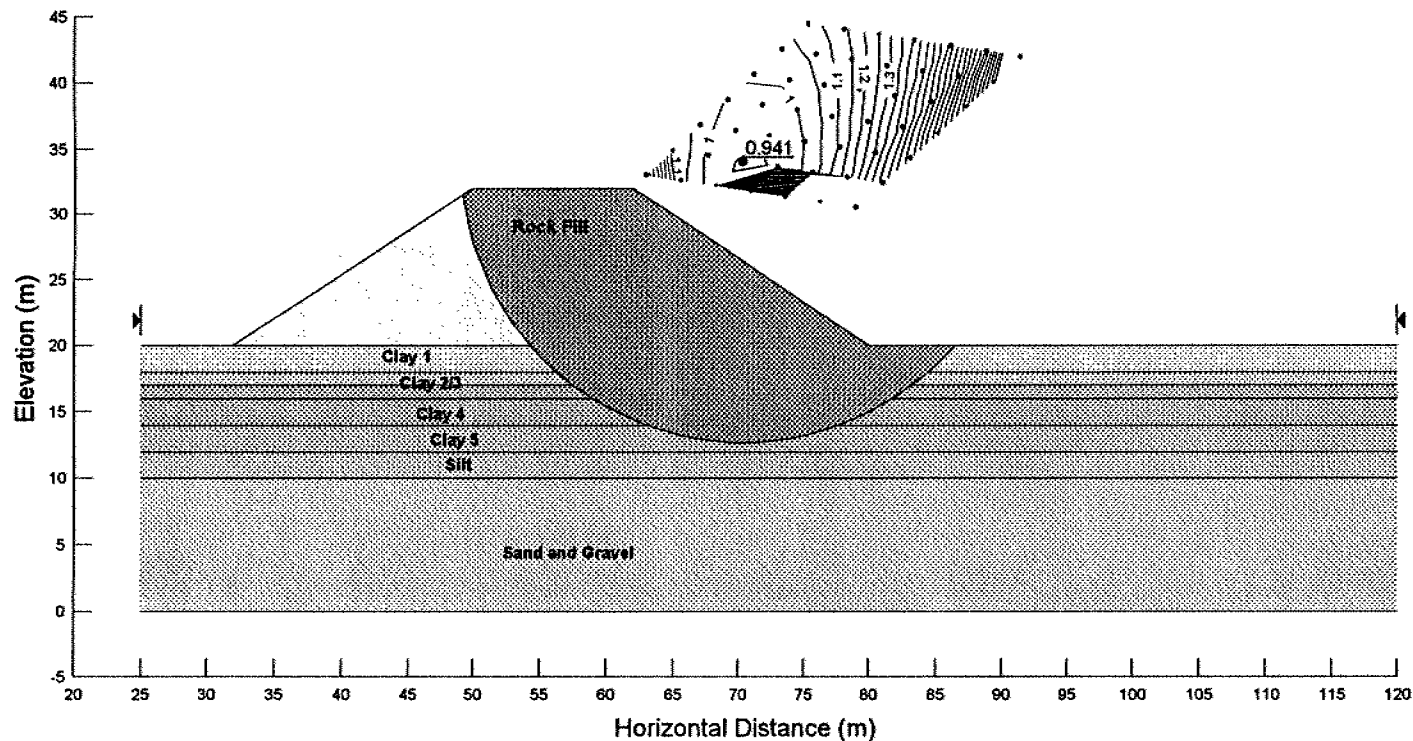
Soil 7 - Clay 6  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 27.5  
 Rate of Increase 2.5  
 Cv - Maximum 32.5  
 Ch/Cv Ratio 1

Soil 8 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 9 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 10 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 12 metre embankment height, 1.5:1 overall side slope  
 8 metre clay foundation  
 H12T08CU.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

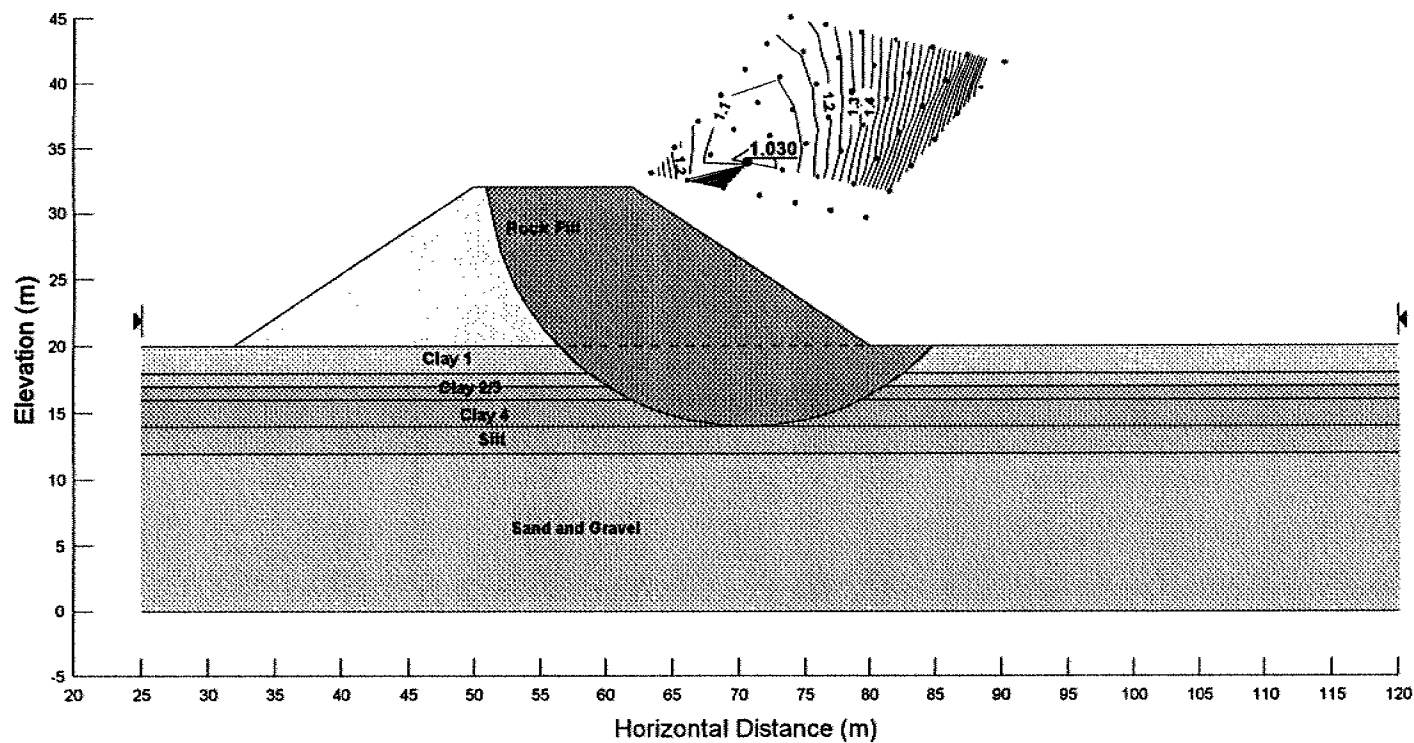
Soil 6 - Clay 5  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

Soil 7 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 8 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 9 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 12 metre embankment height, 1.5:1 overall side slope  
 6 metre clay foundation  
 H12T06CU.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

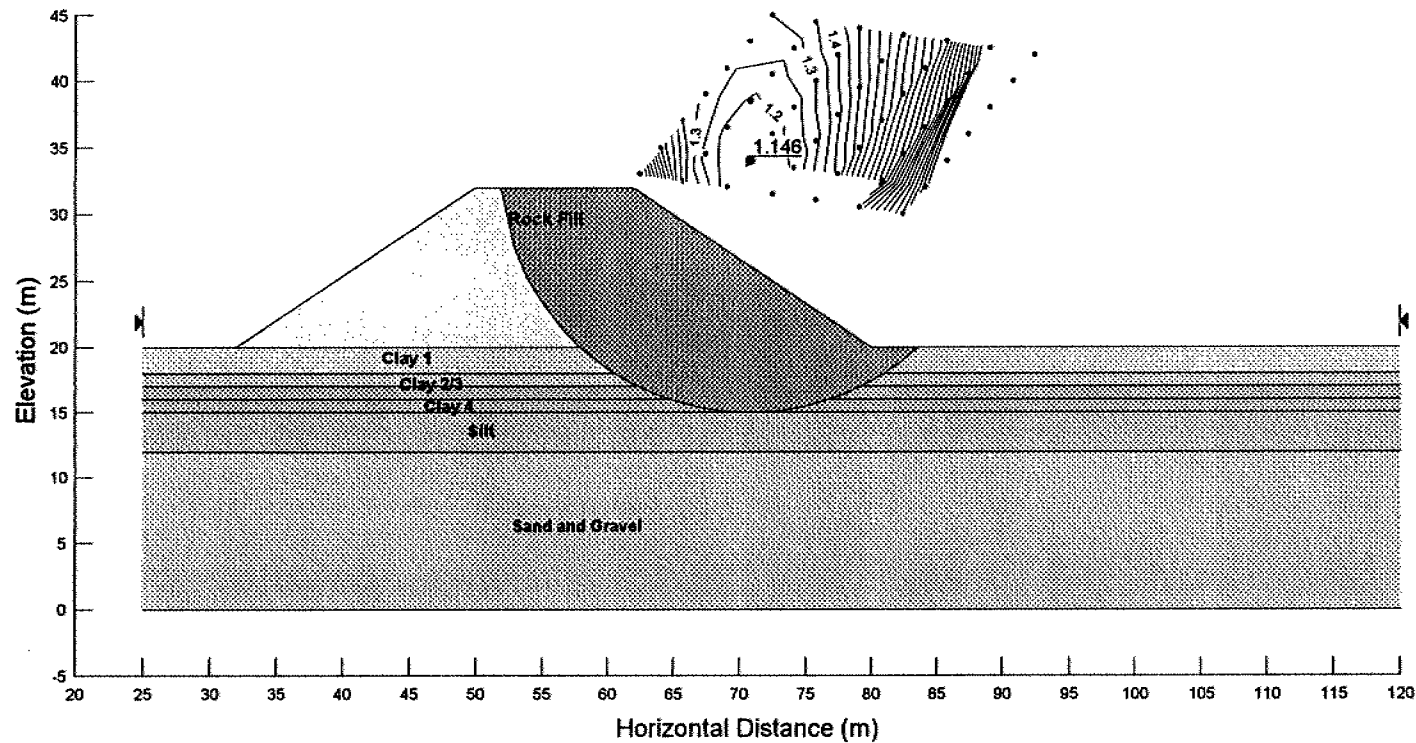
Soil 6 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 7 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 8 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1



Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 12 metre embankment height, 1.5:1 overall side slope  
 5 metre clay foundation  
 H12T05CU.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

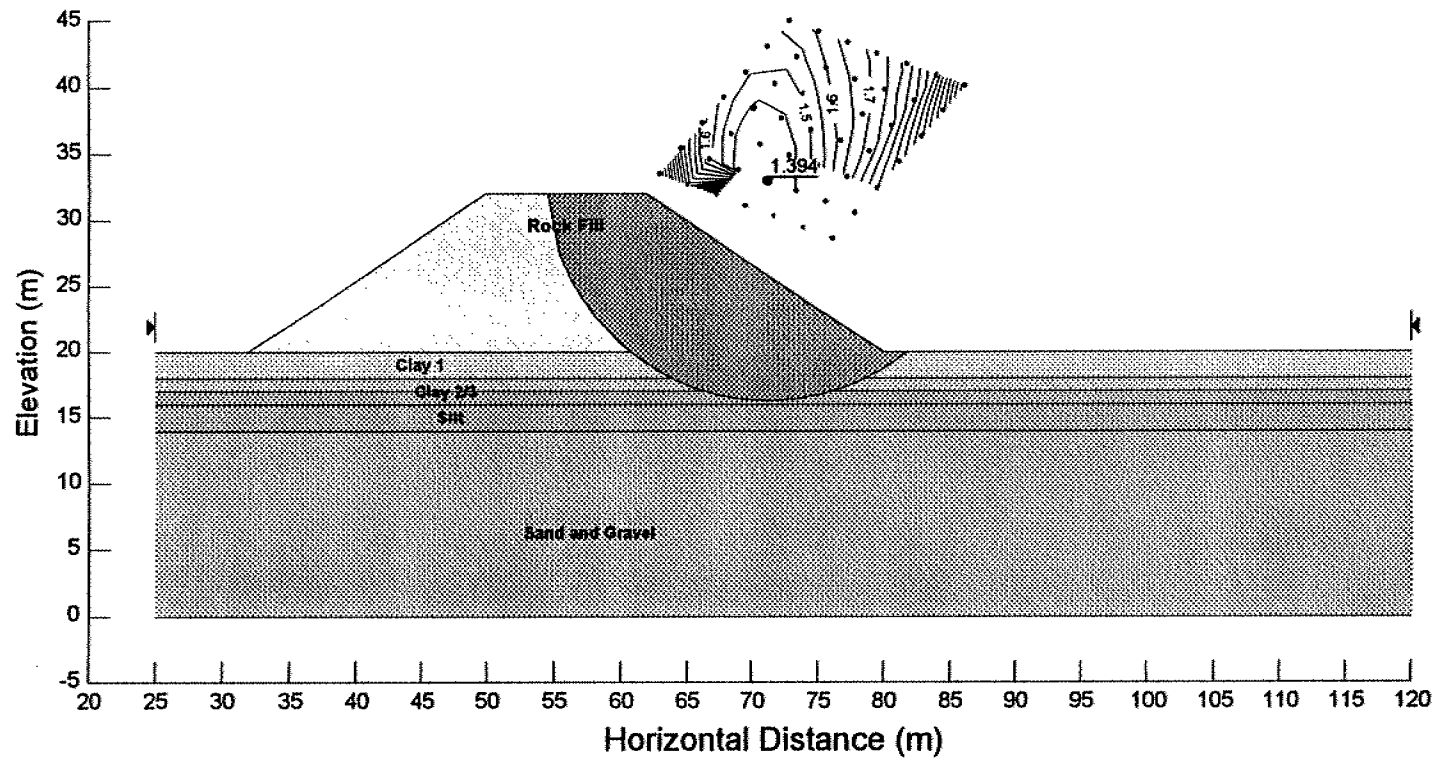
Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

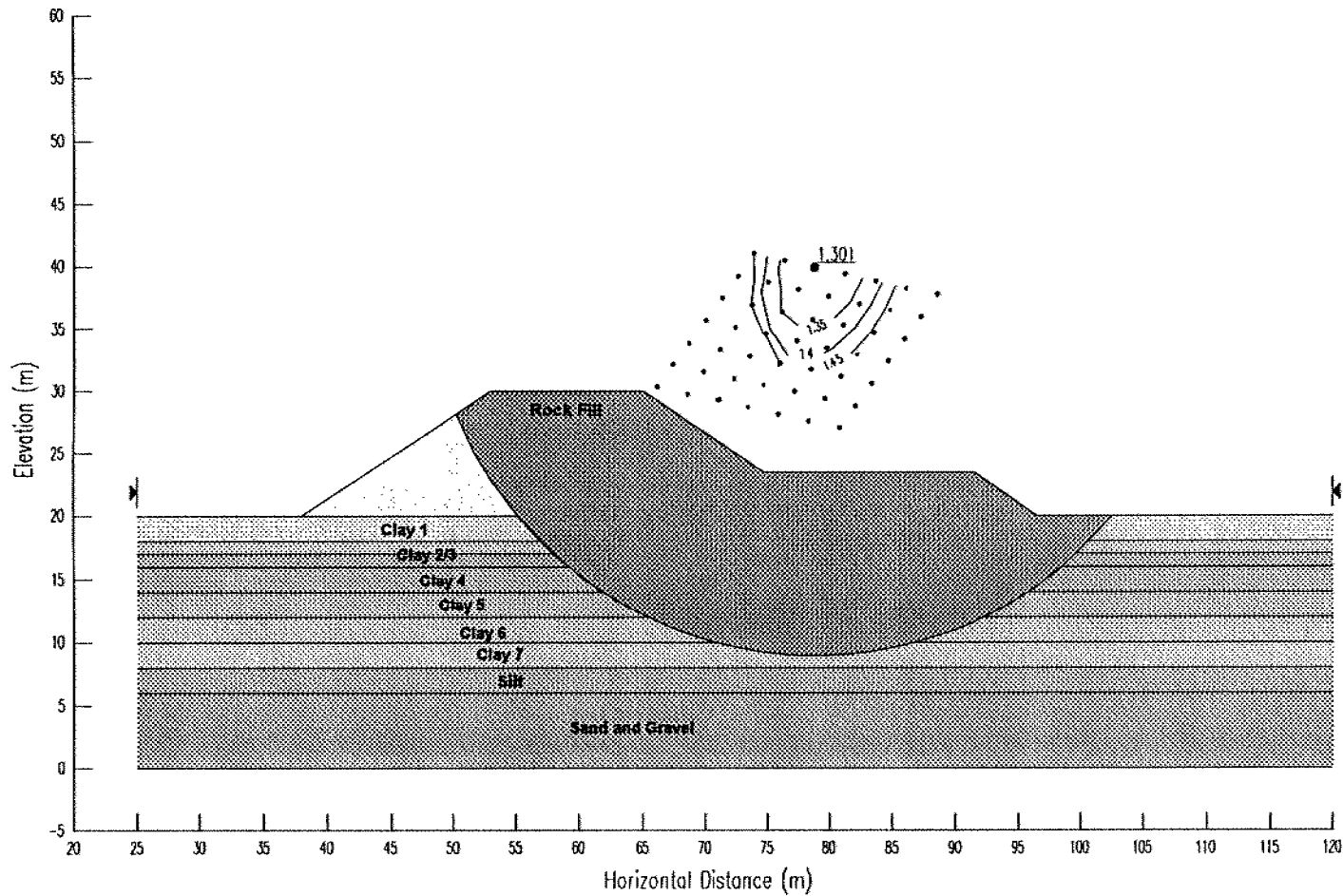
Soil 7 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 8 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 12 metre embankment height, 1.5:1 overall side slope  
 4 metre clay foundation  
 H12T04CU.SLP



Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 10 metre embankment height, 1.5:1 overall side slope  
 12 metre clay foundation  
 3.5 metre high, 16.75 metre long toe berm  
 B10T12CU.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Clay 5  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

Soil 7 - Clay 6  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 27.5  
 Rate of Increase 2.5  
 Cv - Maximum 32.5  
 Ch/Cv Ratio 1

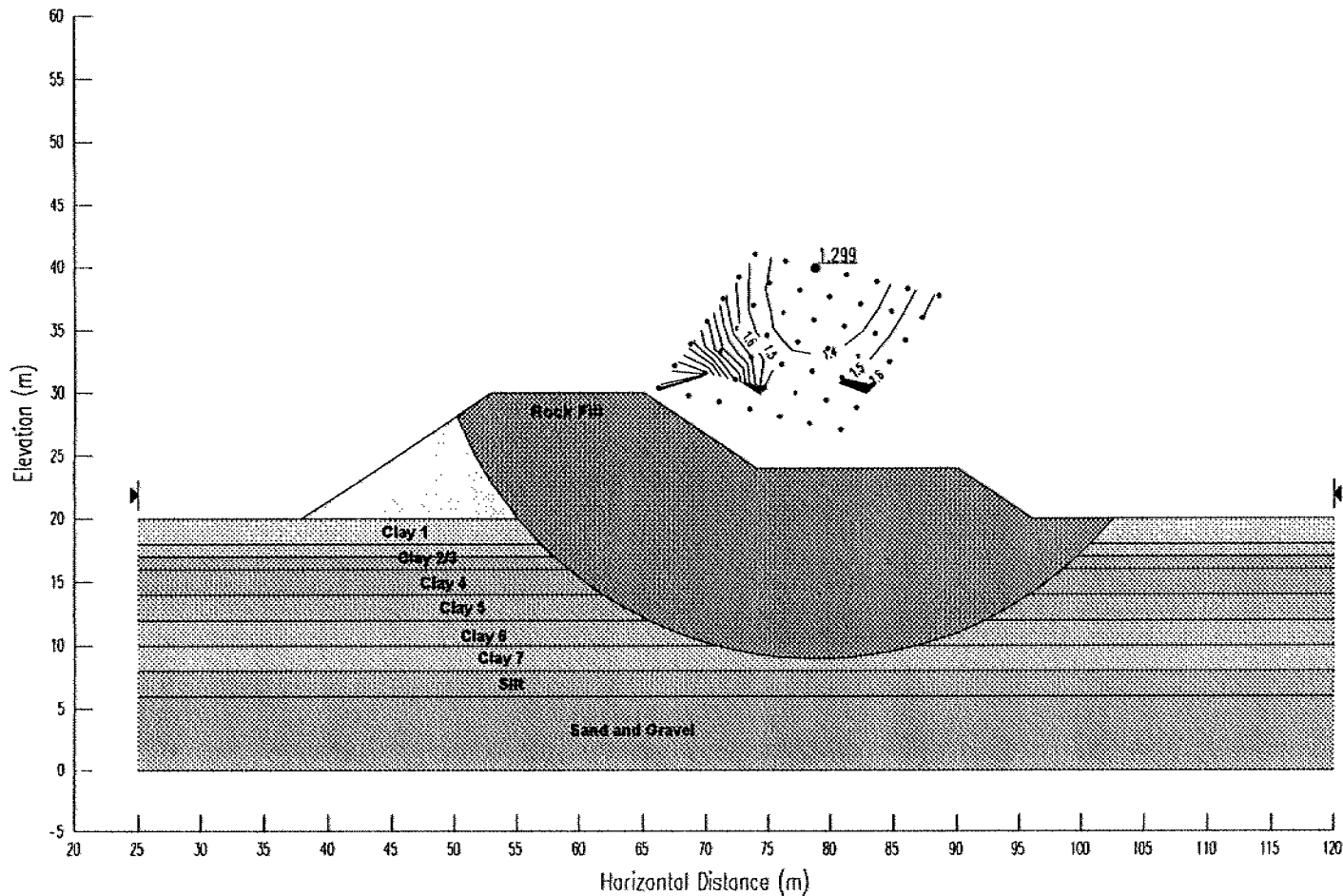
Soil 8 - Clay 7  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 32.5  
 Rate of Increase 2.5  
 Cv - Maximum 37.5  
 Ch/Cv Ratio 1

Soil 9 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 10 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 11 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 10 metre embankment height, 1.5:1 overall side slope  
 12 metre clay foundation  
 4 metre high, 16 metre long toe berm  
 B10T12C4.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Clay 5  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

Soil 7 - Clay 6  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 17.8  
 Cv 27.5  
 Rate of Increase 2.5  
 Cv - Maximum 32.5  
 Ch/Cv Ratio 1

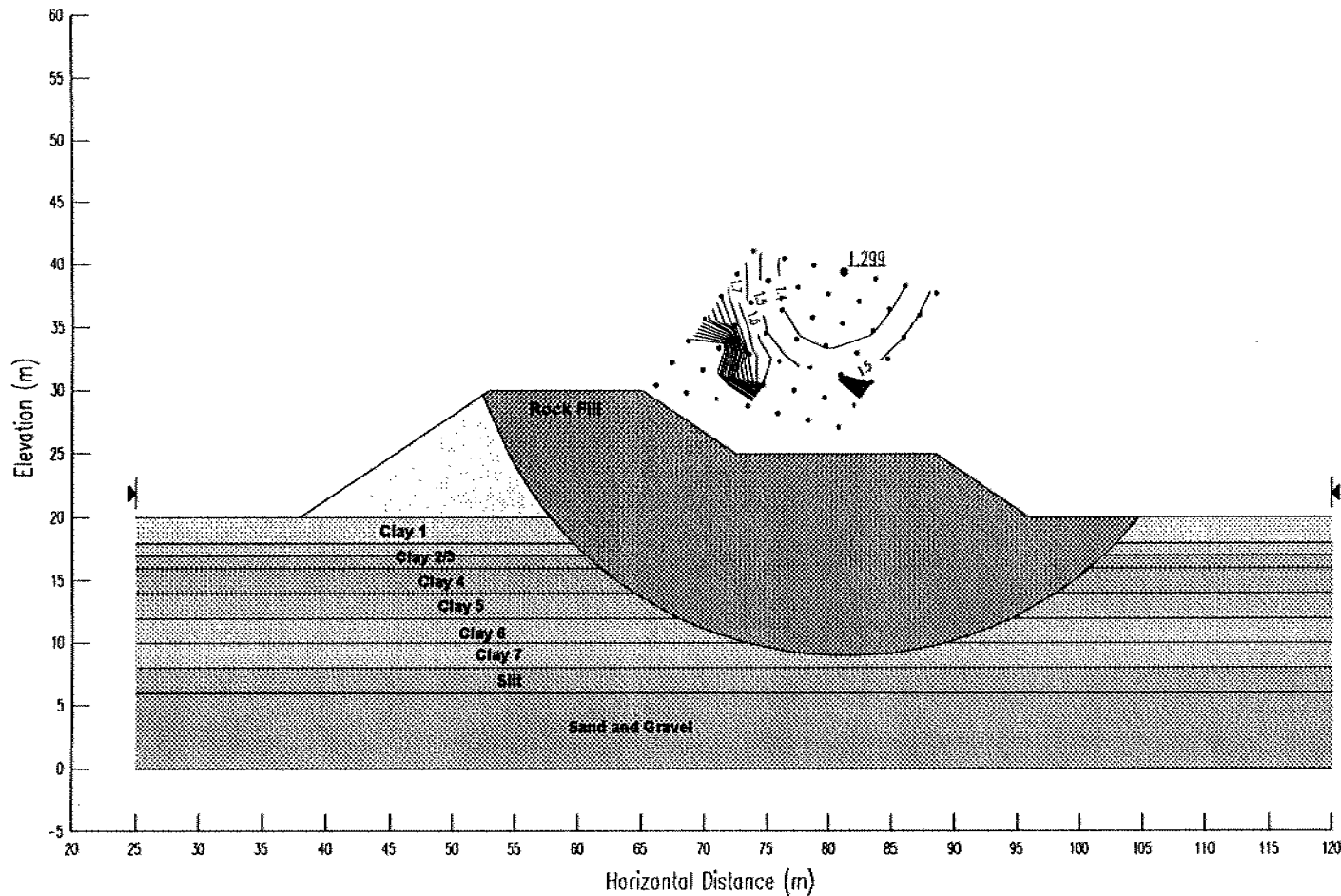
Soil 8 - Clay 7  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 18  
 Cv 32.5  
 Rate of Increase 2.5  
 Cv - Maximum 37.5  
 Ch/Cv Ratio 1

Soil 9 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 10 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 11 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 10 metre embankment height, 1.5:1 overall side slope  
 12 metre clay foundation  
 5 metre high, 16 metre long toe berm  
 B10T12C5.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Clay 5  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

Soil 7 - Clay 6  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 27.5  
 Rate of Increase 2.5  
 Cv - Maximum 32.5  
 Ch/Cv Ratio 1

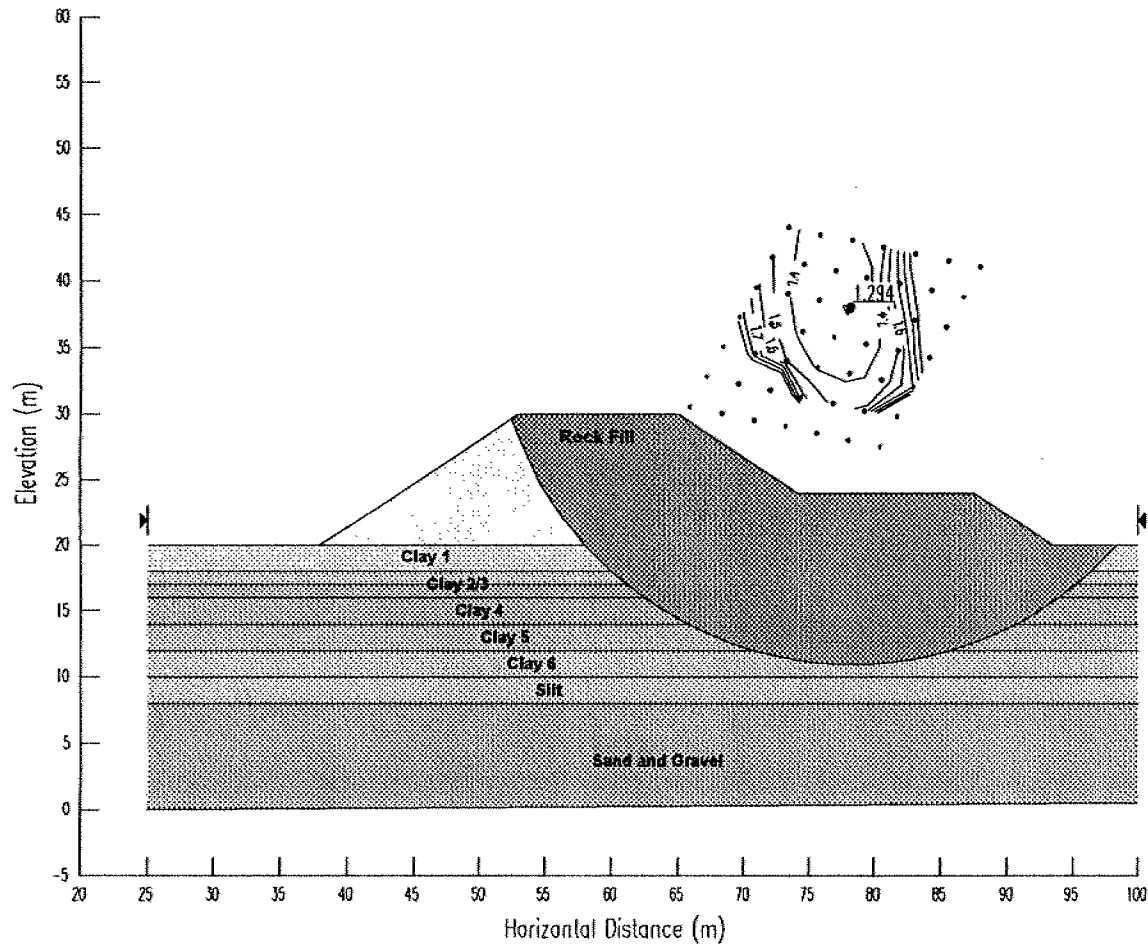
Soil 8 - Clay 7  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 32.5  
 Rate of Increase 2.5  
 Cv - Maximum 37.5  
 Ch/Cv Ratio 1

Soil 9 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 10 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 11 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 10 metre embankment height, 1.5:1 overall side slope  
 10 metre clay foundation  
 3.5 metre high, 14 metre long toe berm  
 B10T10CU.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Clay 5  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

Soil 7 - Clay 6  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 27.5  
 Rate of Increase 2.5  
 Cv - Maximum 32.5  
 Ch/Cv Ratio 1

Soil 8 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 9 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 10 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

# Trout Creek - South Interchange Highway Embankments (F98179)

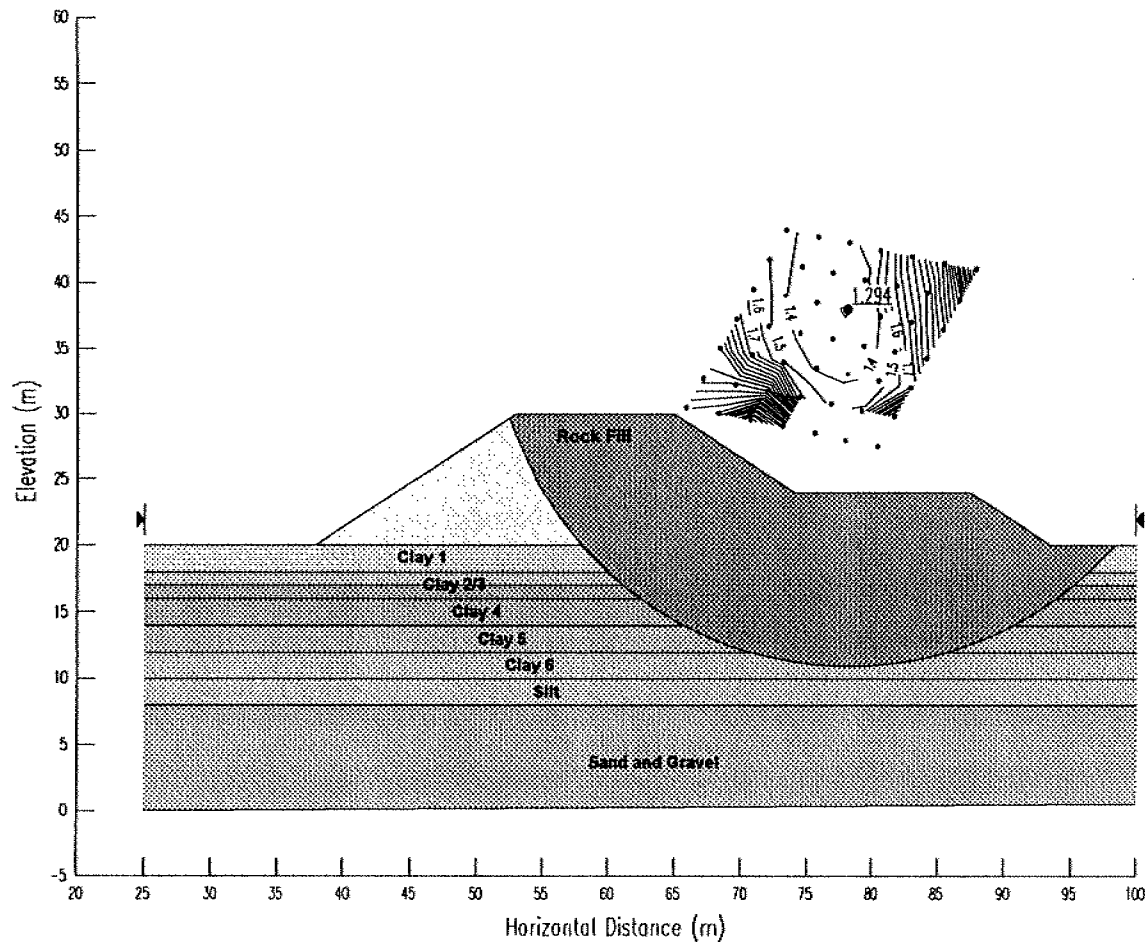
## Total Stress Analysis

10 metre embankment height, 1.5:1 overall side slope

10 metre clay foundation

4 metre high, 13.5 metre long toe berm

B10T10C4.SLP



Soil 1 - Rock Fill  
Soil Model Mohr-Coulomb  
Unit Weight 20  
Cohesion 0  
Phi 42

Soil 2 - Clay 1  
Soil Model S=f(depth)  
Unit Weight 20.5  
Cv 120  
Rate of Increase -35  
Cv - Minimum 50  
Ch/Cv Ratio 1

Soil 3 - Clay 2  
Soil Model S=f(depth)  
Unit Weight 18.3  
Cv 50  
Rate of Increase -20  
Cv - Minimum 30  
Ch/Cv Ratio 1

Soil 4 - Clay 3  
Soil Model S=f(depth)  
Unit Weight 18  
Cv 30  
Rate of Increase -5  
Cv - Minimum 25  
Ch/Cv Ratio 1

Soil 5 - Clay 4  
Soil Model Mohr-Coulomb  
Unit Weight 17.3  
Cohesion 25  
Phi 0

Soil 6 - Clay 5  
Soil Model S=f(depth)  
Unit Weight 17.8  
Cv 25  
Rate of Increase 1.25  
Cv - Maximum 27.5  
Ch/Cv Ratio 1

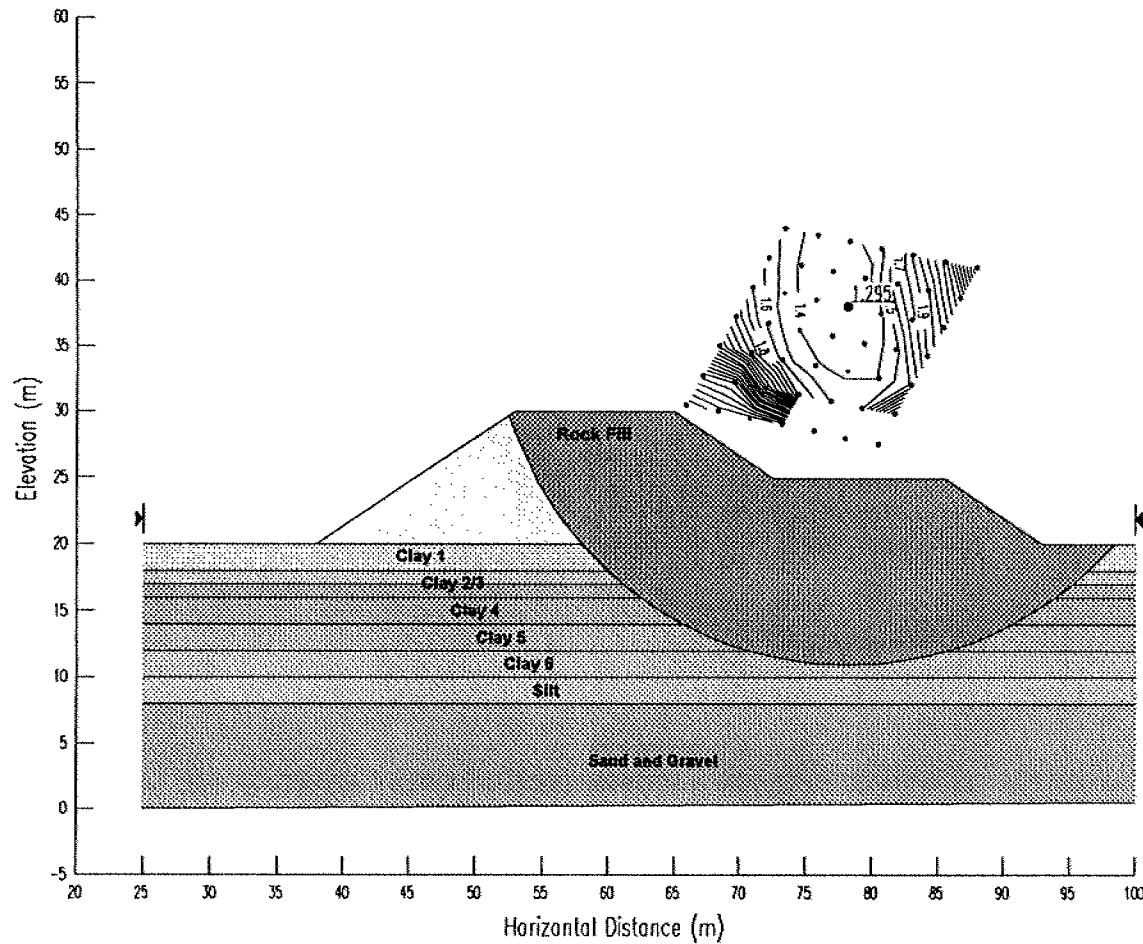
Soil 7 - Clay 6  
Soil Model S=f(depth)  
Unit Weight 17.8  
Cv 27.5  
Rate of Increase 2.5  
Cv - Maximum 32.5  
Ch/Cv Ratio 1

Soil 8 - Silt  
Soil Model Mohr-Coulomb  
Unit Weight 19.5  
Cohesion 0  
Phi 31

Soil 9 - Sand/Gravel  
Soil Model Mohr-Coulomb  
Unit Weight 21.5  
Cohesion 0  
Phi 35

Soil 10 - Bedrock  
Soil Model Bedrock  
Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 10 metre embankment height, 1.5:1 overall side slope  
 10 metre clay foundation  
 5 metre high, 13 metre long toe berm  
 B10T10C5.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Clay 5  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

Soil 7 - Clay 6  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 27.5  
 Rate of Increase 2.5  
 Cv - Maximum 32.5  
 Ch/Cv Ratio 1

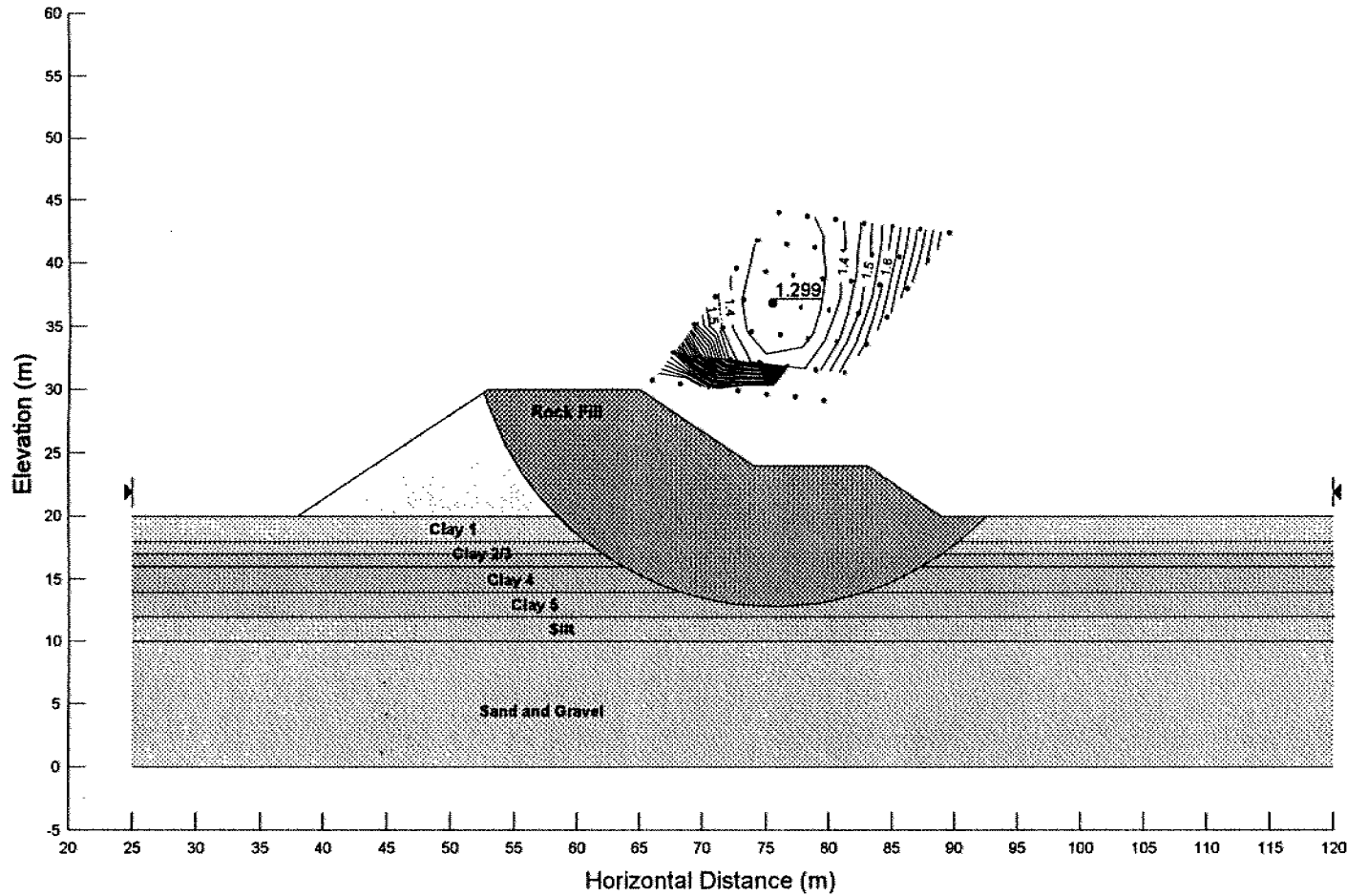
Soil 8 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 9 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 10 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1



Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 10 metre embankment height, 1.5:1 overall side slope  
 8 metre clay foundation  
 4 metre high, 9 metre long toe berm  
 B10T08C4.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

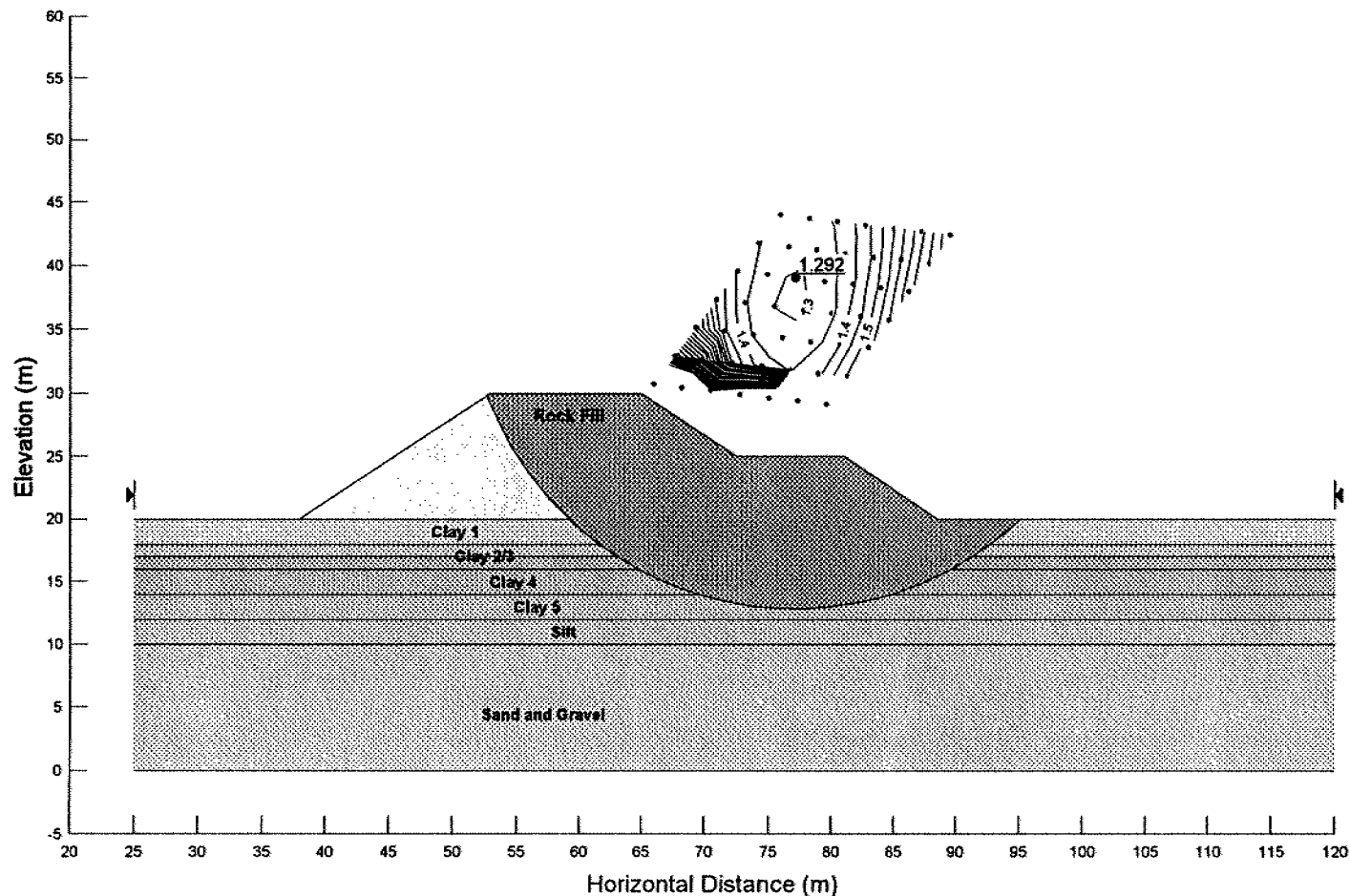
Soil 6 - Clay 5  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

Soil 7 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 8 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 9 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 10 metre embankment height, 1.5:1 overall side slope  
 8 metre clay foundation  
 5 metre high, 8.5 metre long toe berm  
 B10T08C5.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

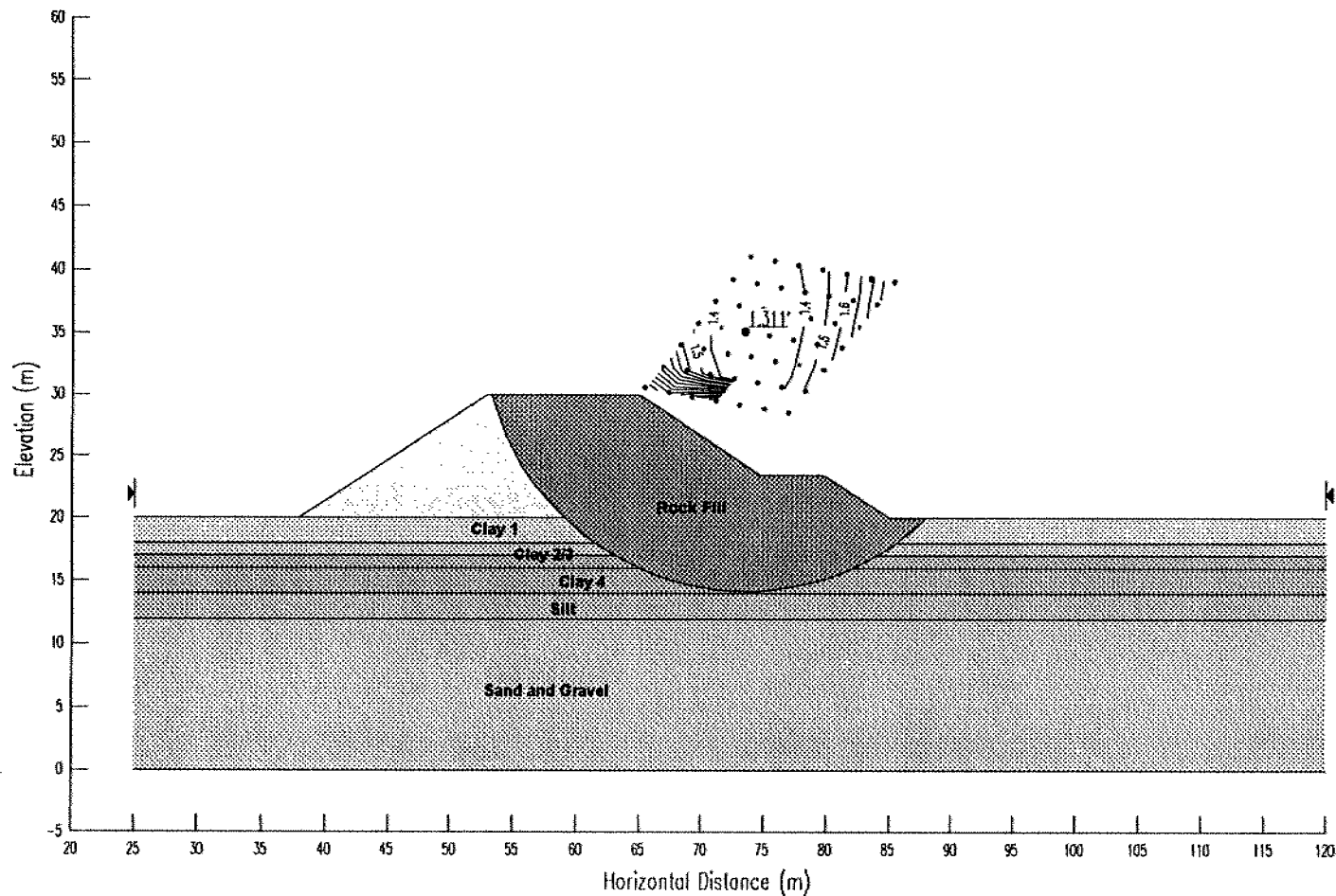
Soil 6 - Clay 5  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

Soil 7 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 8 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 9 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 10 metre embankment height, 1.5:1 overall side slope  
 6 metre clay foundation  
 3.5 metre high, 5 metre long toe berm  
 B10T06CU.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

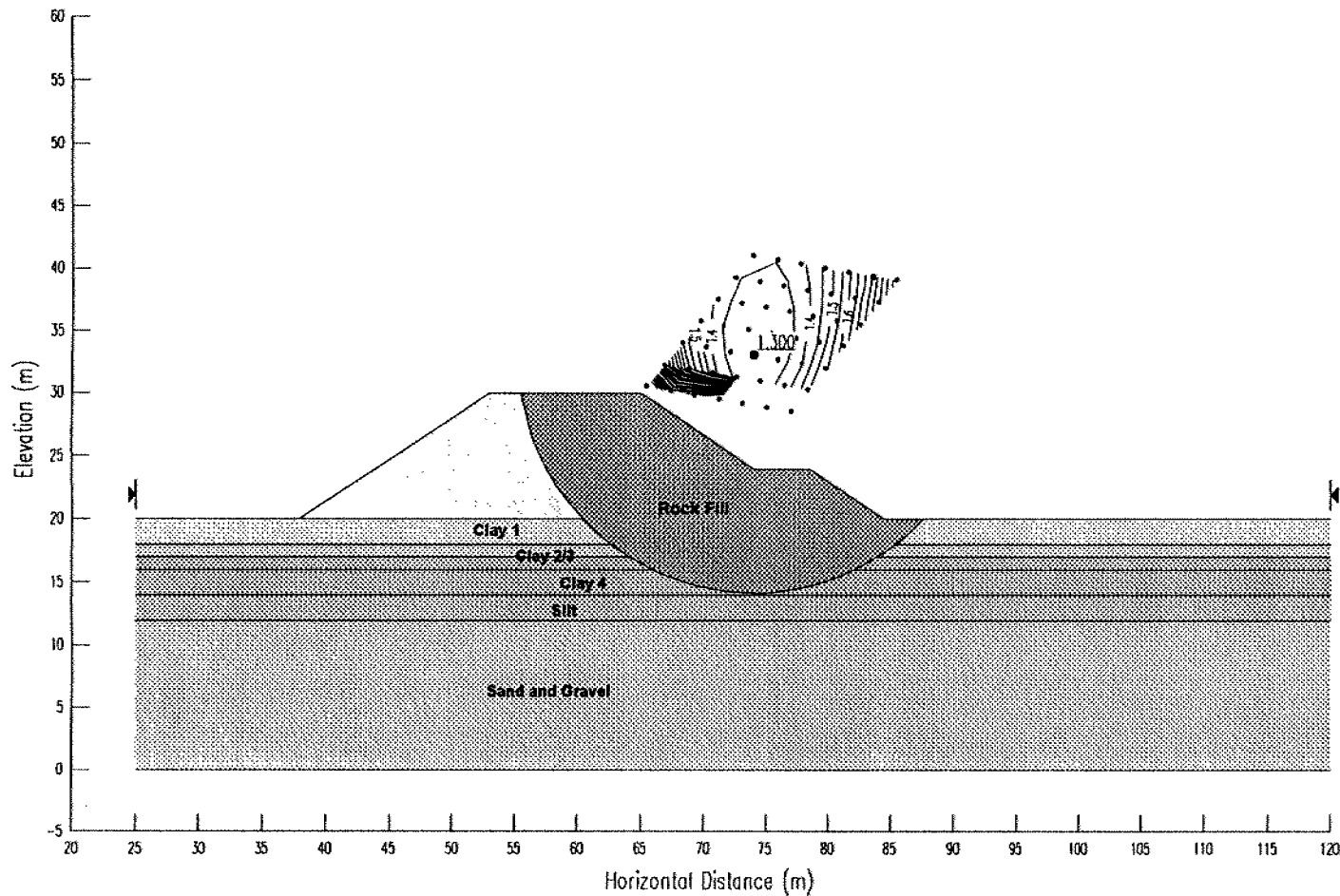
Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 7 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 8 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 10 metre embankment height, 1.5:1 overall side slope  
 6 metre clay foundation  
 4 metre high, 4.5 metre long toe berm  
 B10T06C4.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

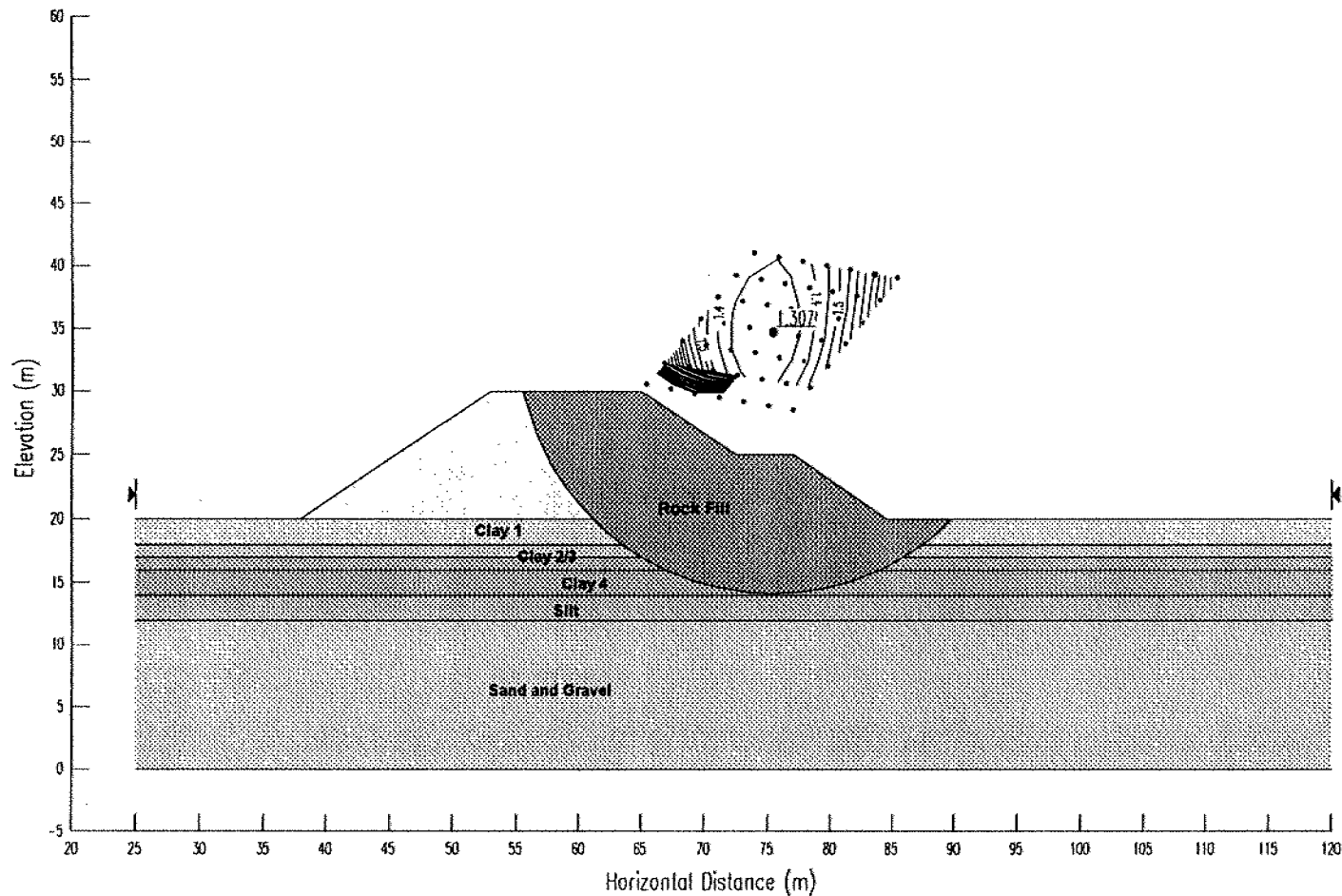
Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 7 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 8 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 10 metre embankment height, 1.5:1 overall side slope  
 6 metre clay foundation  
 5 metre high, 4.5 metre long toe berm  
 B10T06C5.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

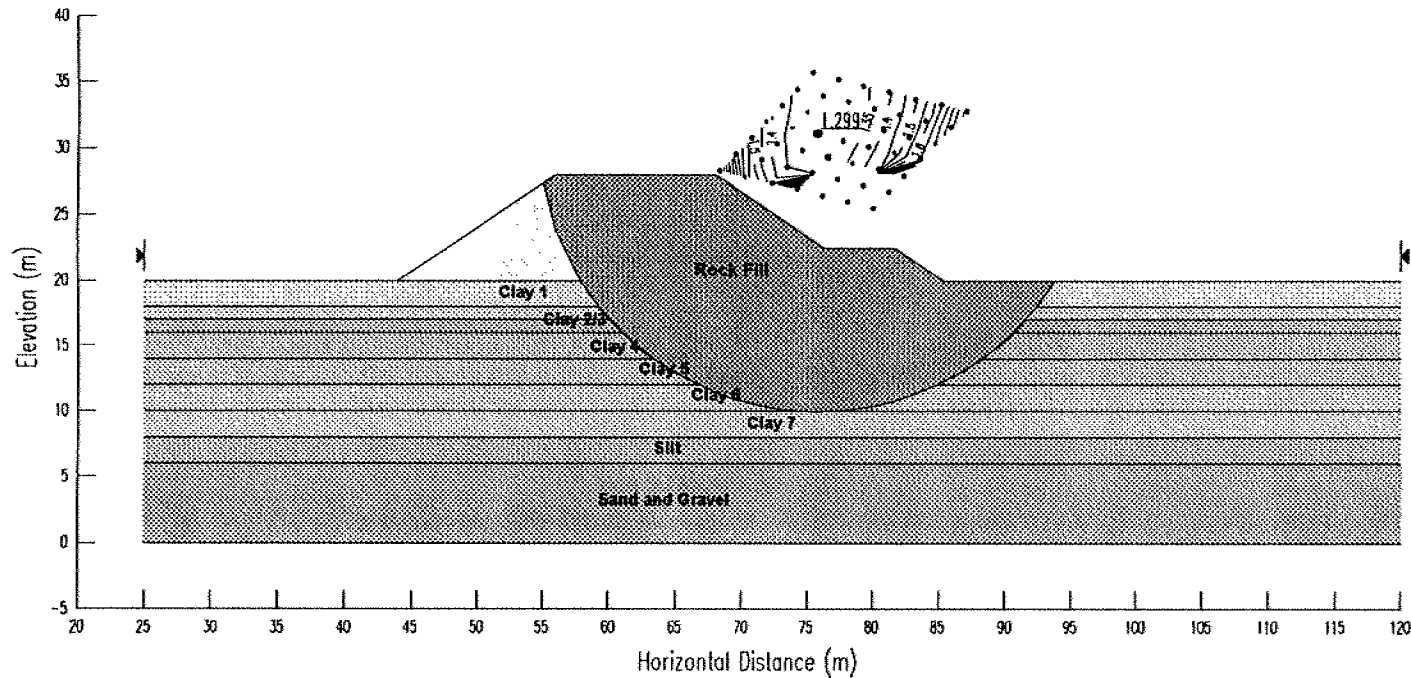
Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 7 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 8 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 8 metre embankment height, 1.5:1 overall side slope  
 12 metre clay foundation  
 2.5 metre high, 5.5 metre long toe berm  
 B08T12CU.SLP



Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

Soil 7 - Clay 6  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 27.5  
 Rate of Increase 2.5  
 Cv - Maximum 32.5  
 Ch/Cv Ratio 1

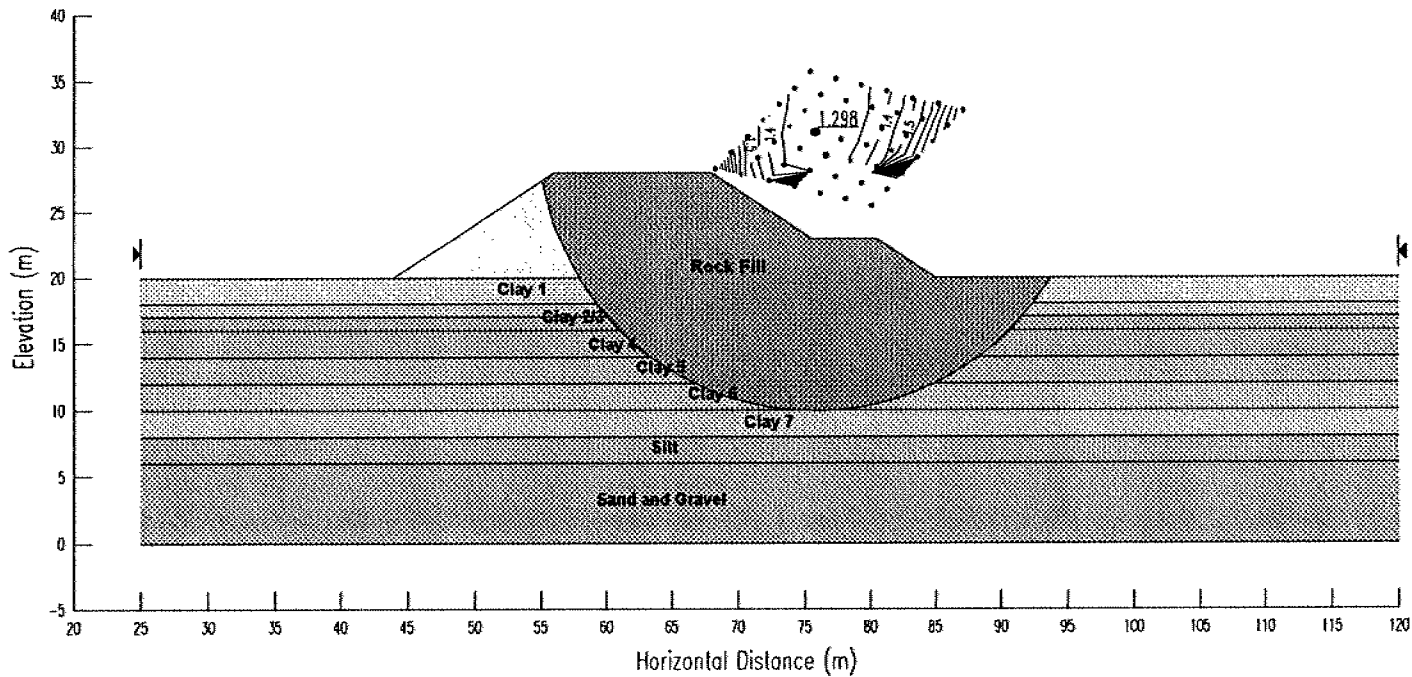
Soil 8 - Clay 7  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 32.5  
 Rate of Increase 2.5  
 Cv - Maximum 37.5  
 Ch/Cv Ratio 1

Soil 9 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 10 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 11 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 8 metre embankment height, 1.5:1 overall side slope  
 12 metre clay foundation  
 3 metre high, 5 metre long toe berm  
 B08T12C3.SLP



Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

Soil 7 - Clay 6  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 27.5  
 Rate of Increase 2.5  
 Cv - Maximum 32.5  
 Ch/Cv Ratio 1

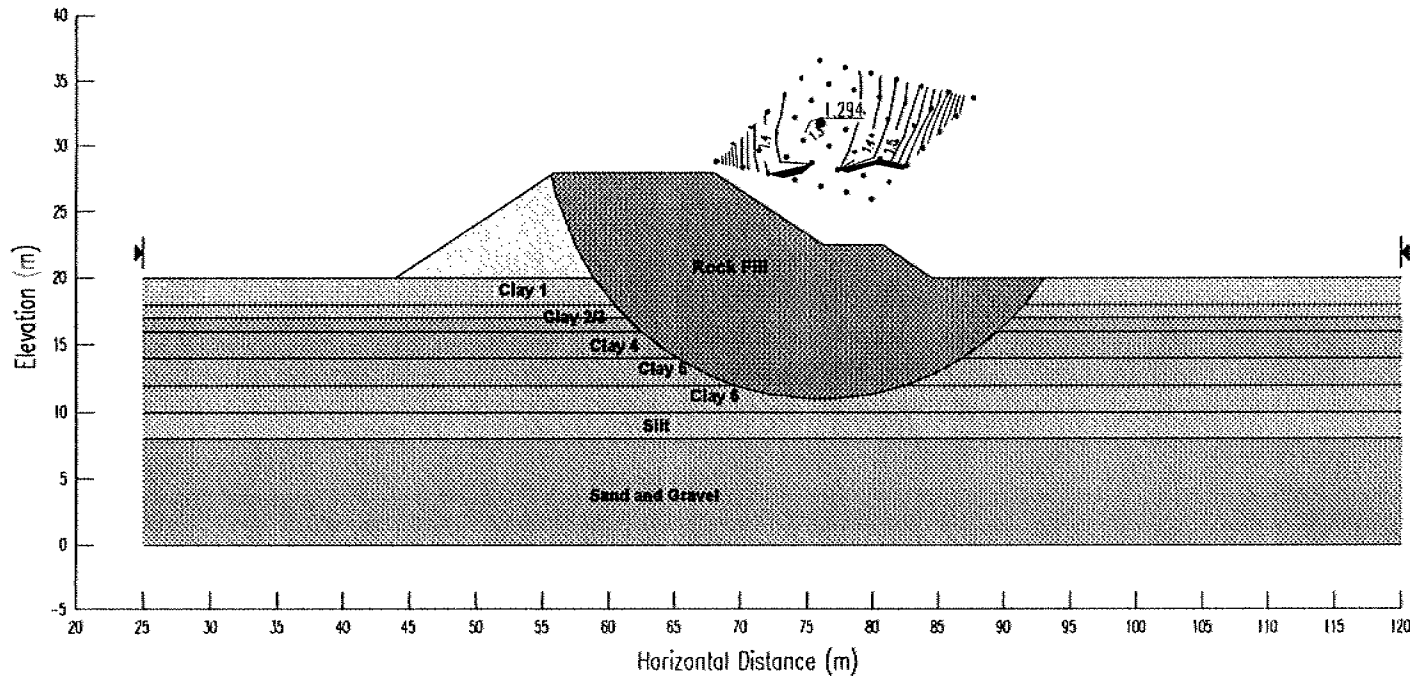
Soil 8 - Clay 7  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 32.5  
 Rate of Increase 2.5  
 Cv - Maximum 37.5  
 Ch/Cv Ratio 1

Soil 9 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 10 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 11 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 8 metre embankment height, 1.5:1 overall side slope  
 10 metre clay foundation  
 2.5 metre high, 4.5 metre long toe berm  
 B8T10CU.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Clay 5  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

Soil 7 - Clay 6  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 27.5  
 Rate of Increase 2.5  
 Cv - Maximum 32.5  
 Ch/Cv Ratio 1

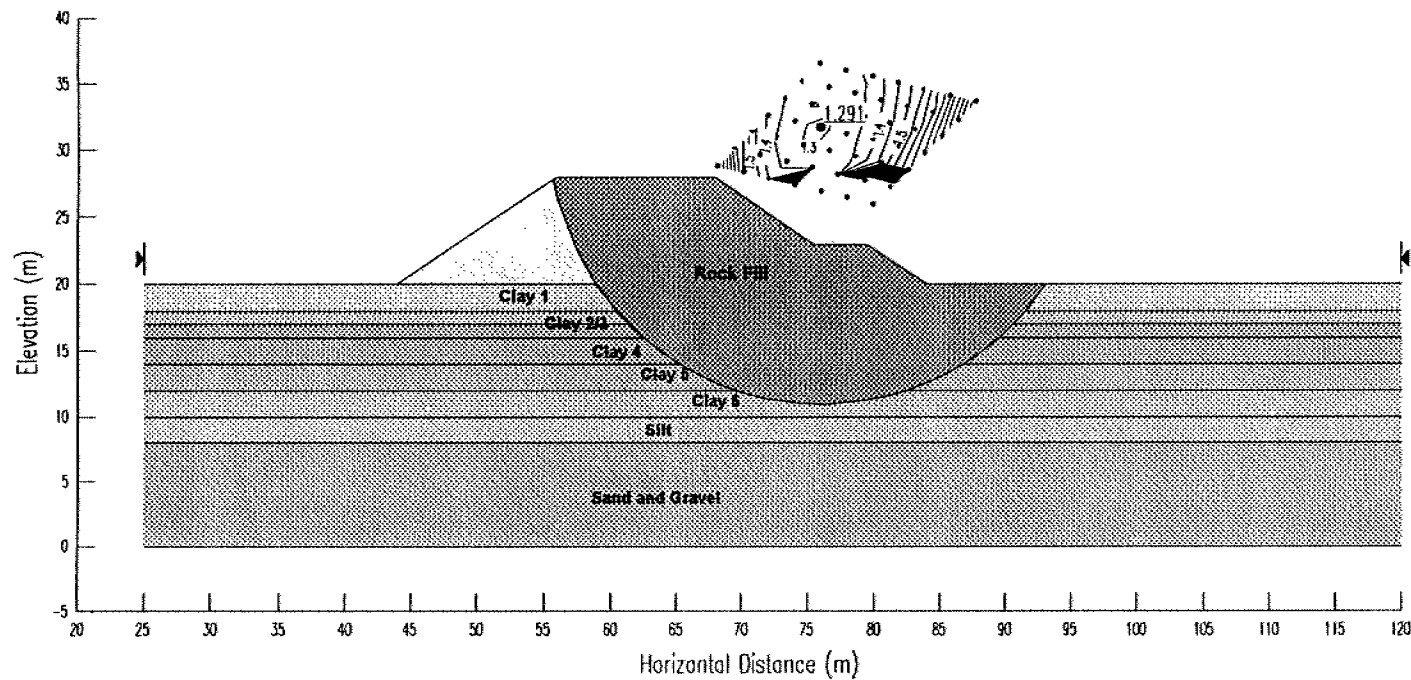
Soil 8 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 9 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 10 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1



Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 8 metre embankment height, 1.5:1 overall side slope  
 10 metre clay foundation  
 3 metre high, 4 metre long toe berm  
 B8T10C3.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Clay 5  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

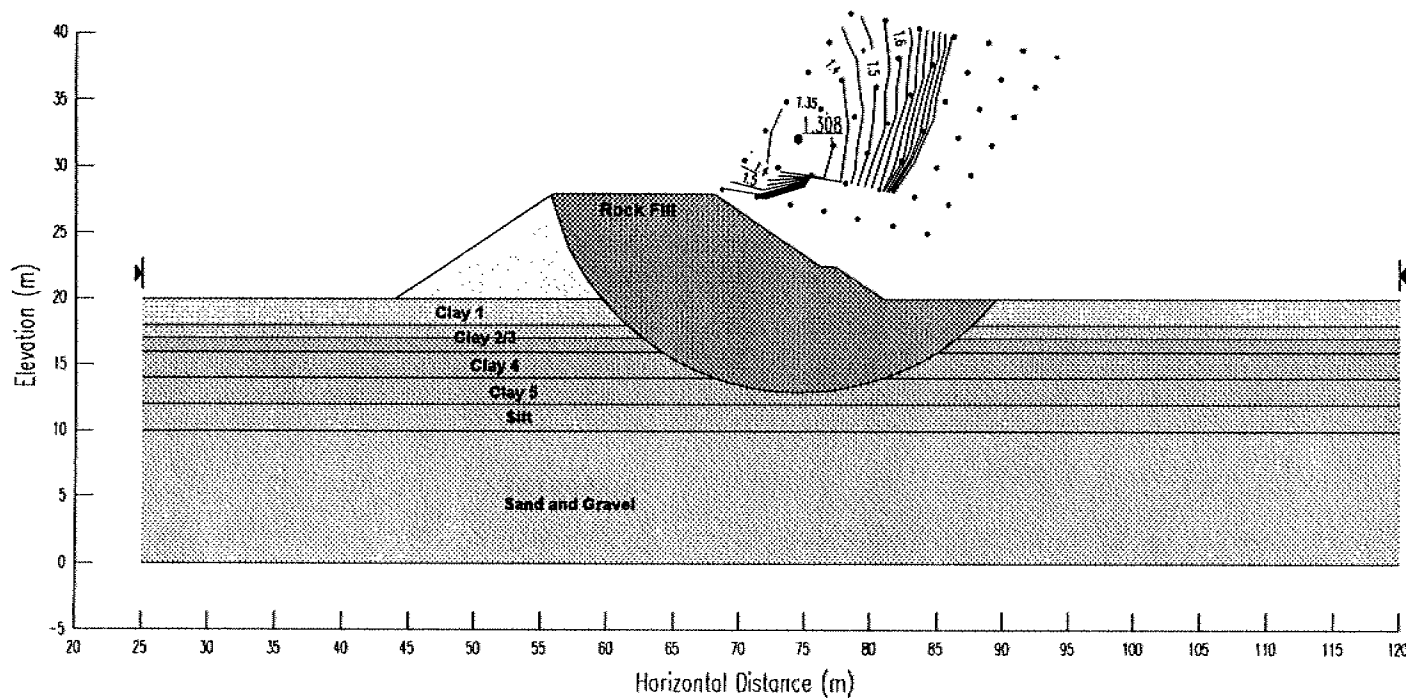
Soil 7 - Clay 6  
 Soil Model S=f(depth)  
 Unit Weight 17.6  
 Cv 27.5  
 Rate of Increase 2.5  
 Cv - Maximum 32.5  
 Ch/Cv Ratio 1

Soil 8 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 9 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 10 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 8 metre embankment height, 1.5:1 overall side slope  
 8 metre clay foundation  
 2.5 metre high, 1 metre long toe berm  
 B08T08CU.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

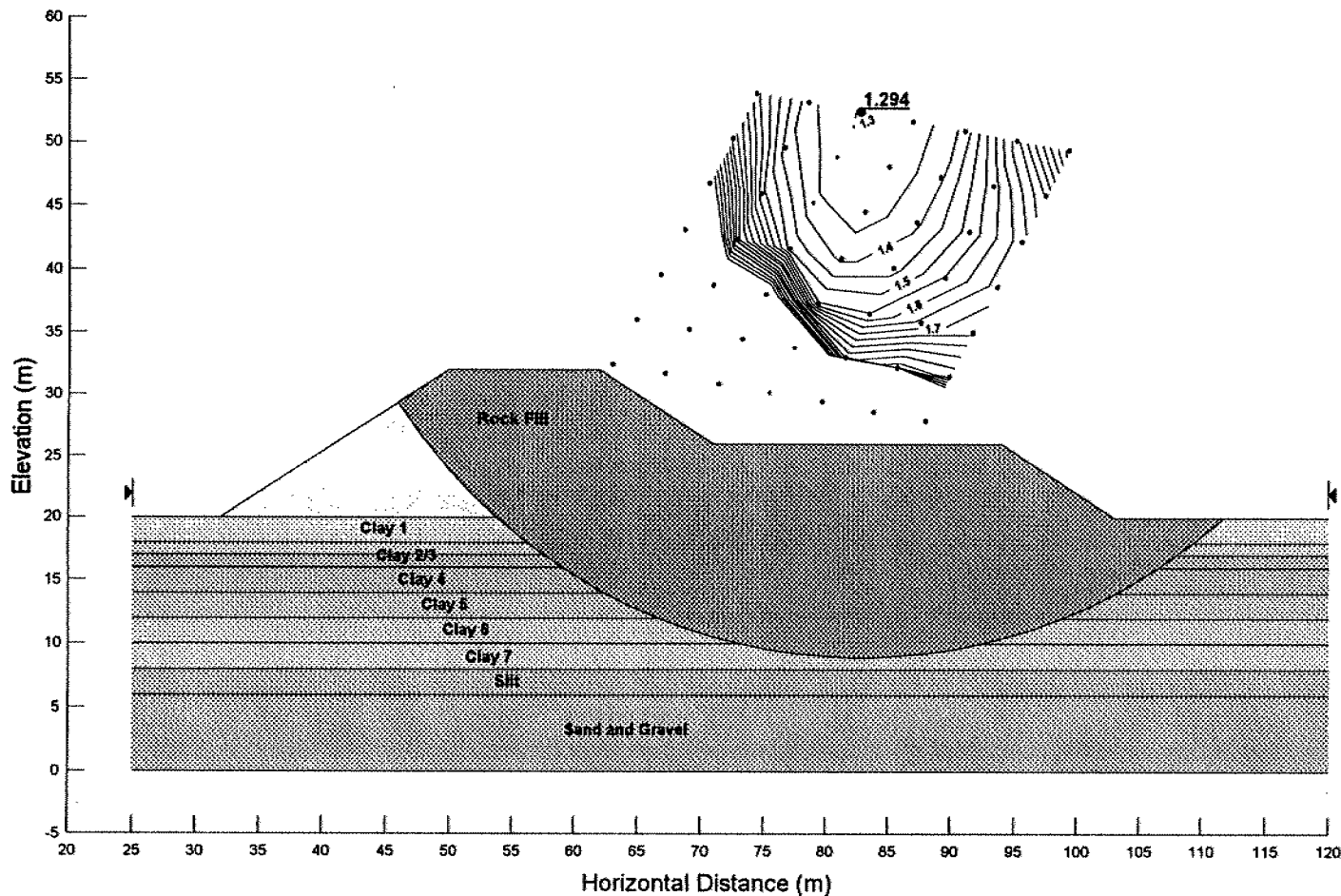
Soil 6 - Clay 5  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

Soil 7 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 8 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 9 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 12 metre embankment height, 1.5:1 overall side slope  
 12 metre clay foundation  
 6 metre high, 23 metre long toe berm  
 B12T12CU.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Clay 5  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

Soil 7 - Clay 6  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 27.5  
 Rate of Increase 2.5  
 Cv - Maximum 32.5  
 Ch/Cv Ratio 1

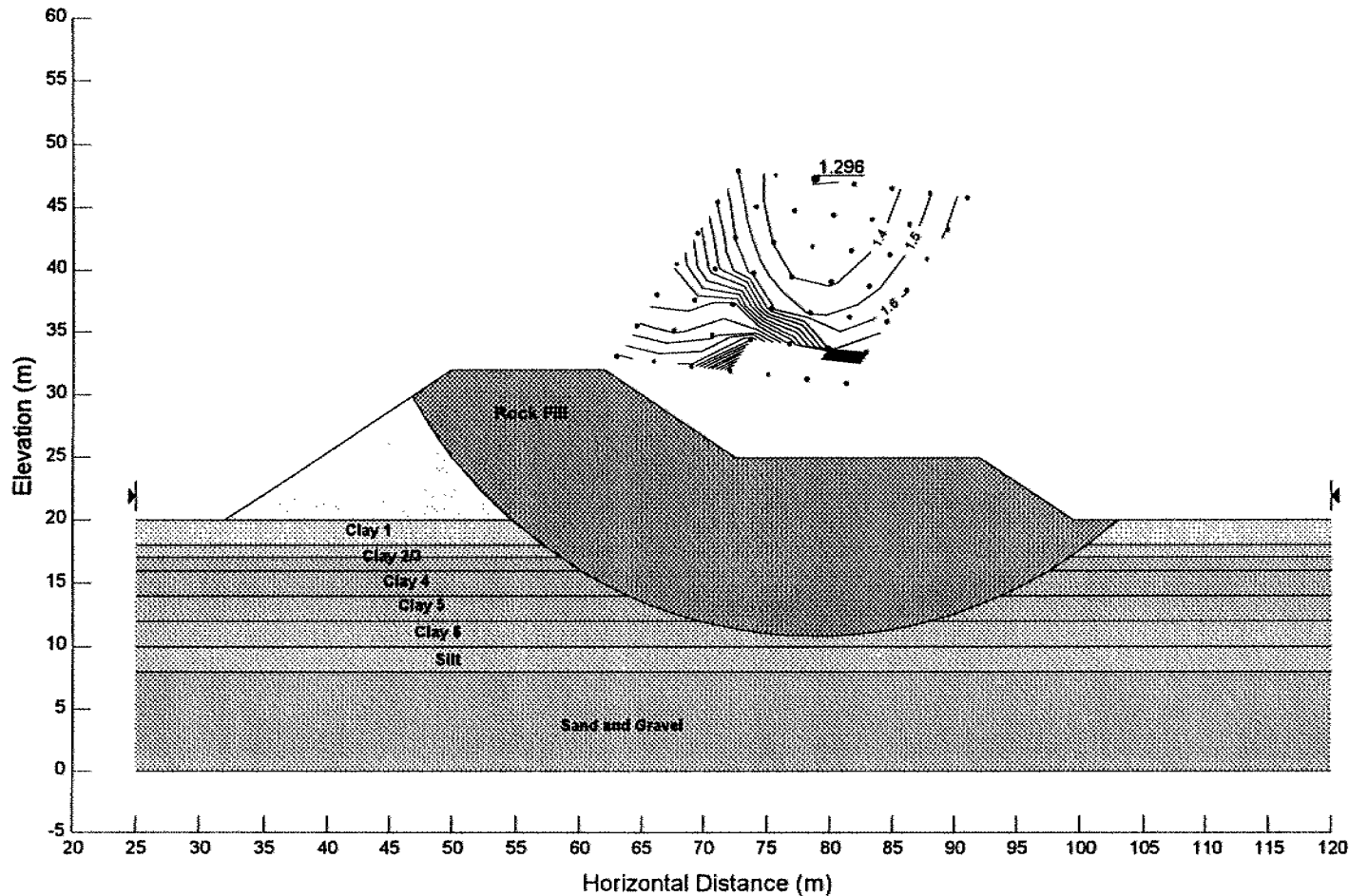
Soil 8 - Clay 7  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 32.5  
 Rate of Increase 2.5  
 Cv - Maximum 37.5  
 Ch/Cv Ratio 1

Soil 9 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 10 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 11 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 12 metre embankment height, 1.5:1 overall side slope  
 10 metre clay foundation  
 5 metre high, 19.5 metre long toe berm  
 B12T10C5.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Clay 5  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

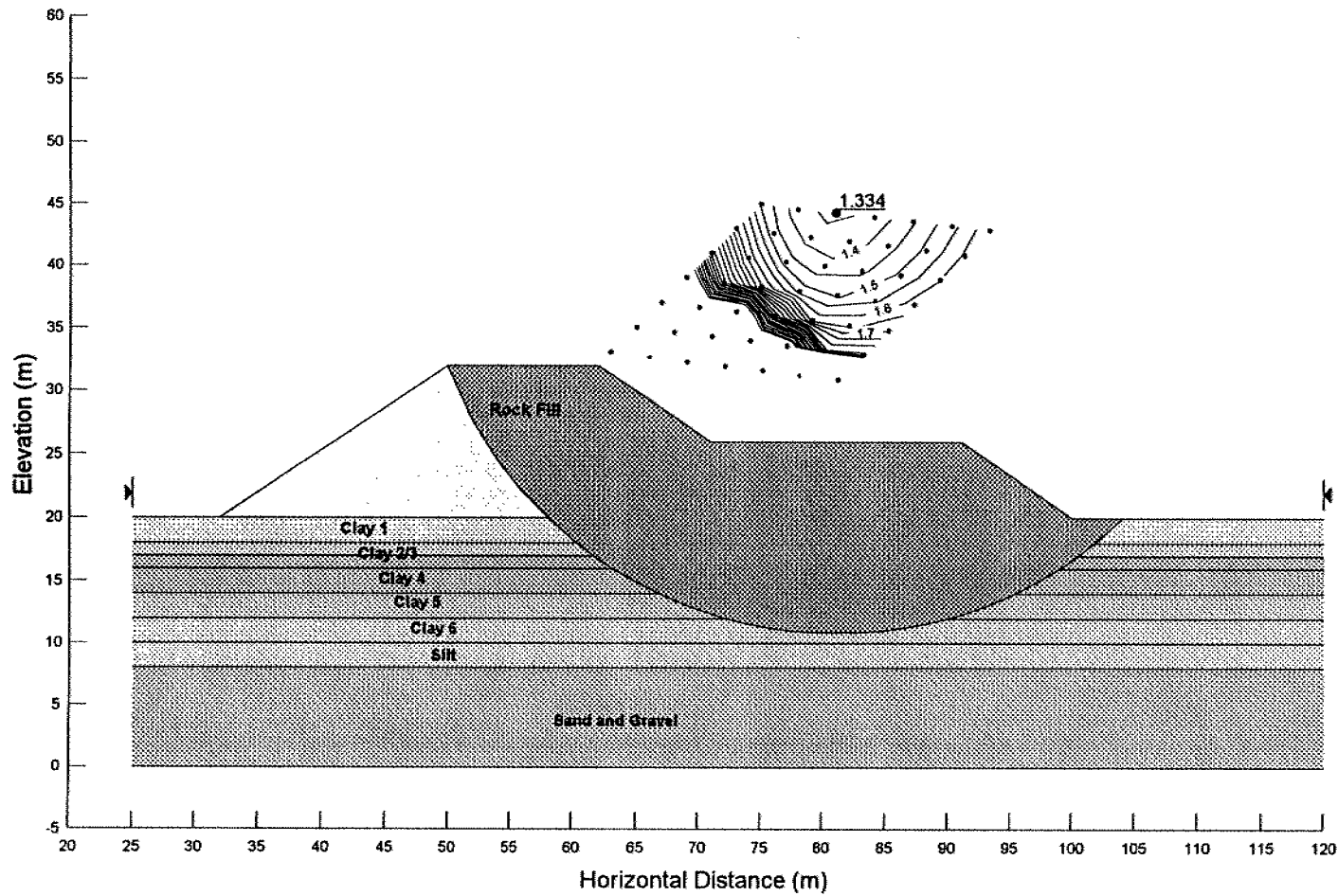
Soil 7 - Clay 6  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 27.5  
 Rate of Increase 2.5  
 Cv - Maximum 32.5  
 Ch/Cv Ratio 1

Soil 8 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 9 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

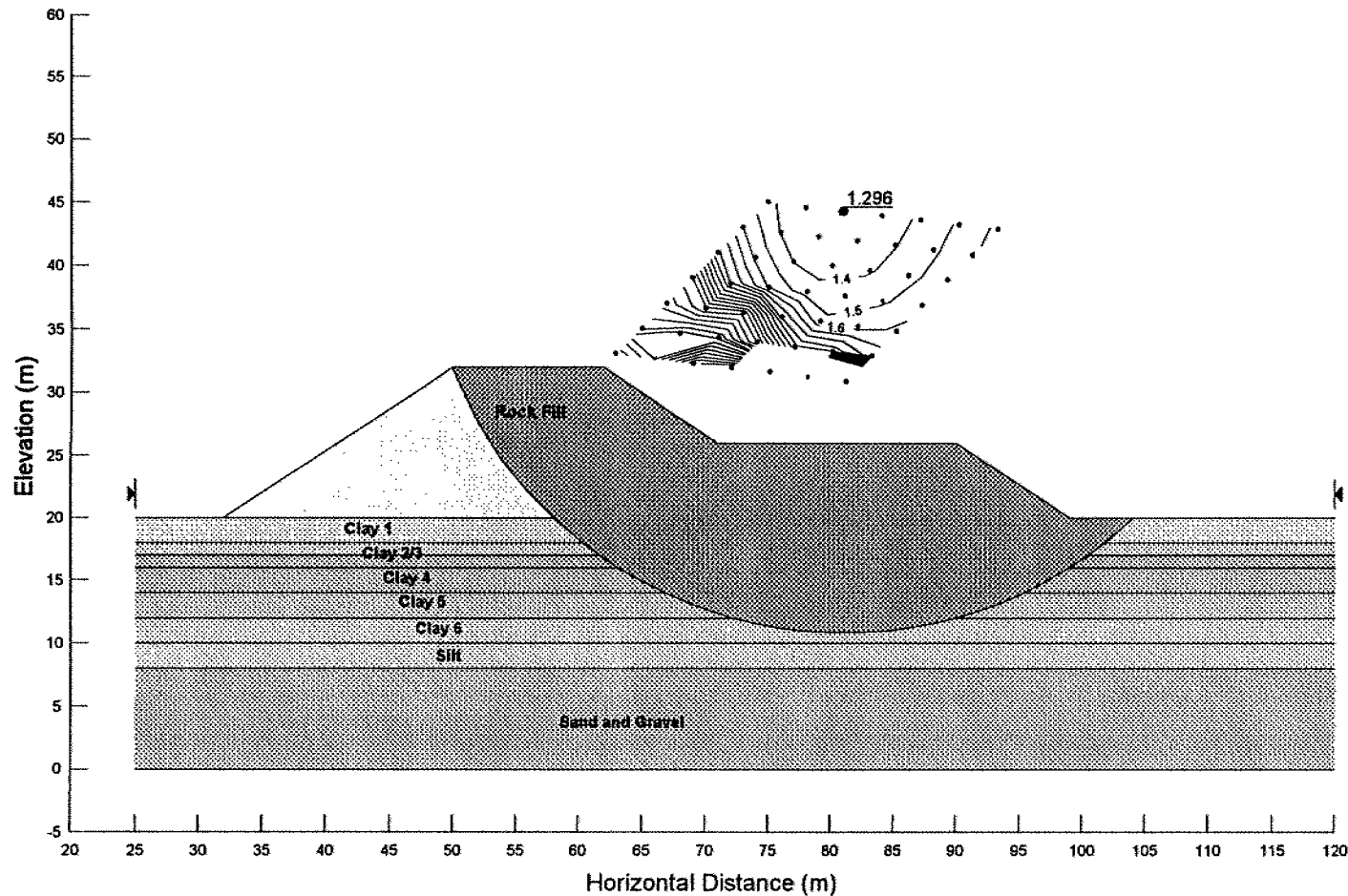
Soil 10 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 12 metre embankment height, 1.5:1 overall side slope  
 10 metre clay foundation  
 5 metre high, 20 metre long toe berm  
 B12T10CU.SLP



Soil 1 - Rock Fill Soil Model Mohr-Coulomb Unit Weight 20 Cohesion 0 Phi 42	Soil 6 - Clay 5 Soil Model S=f(depth) Unit Weight 17.8 Cv 25 Rate of Increase 1.25 Cv - Maximum 27.5 Ch/Cv Ratio 1
Soil 2 - Clay 1 Soil Model S=f(depth) Unit Weight 20.5 Cv 120 Rate of Increase -35 Cv - Minimum 50 Ch/Cv Ratio 1	Soil 7 - Clay 6 Soil Model S=f(depth) Unit Weight 17.8 Cv 27.5 Rate of Increase 2.5 Cv - Maximum 32.5 Ch/Cv Ratio 1
Soil 3 - Clay 2 Soil Model S=f(depth) Unit Weight 18.3 Cv 50 Rate of Increase -20 Cv - Minimum 30 Ch/Cv Ratio 1	Soil 8 - Silt Soil Model Mohr-Coulomb Unit Weight 19.5 Cohesion 0 Phi 31
Soil 4 - Clay 3 Soil Model S=f(depth) Unit Weight 18 Cv 30 Rate of Increase -5 Cv - Minimum 25 Ch/Cv Ratio 1	Soil 9 - Sand/Gravel Soil Model Mohr-Coulomb Unit Weight 21.5 Cohesion 0 Phi 35
Soil 5 - Clay 4 Soil Model Mohr-Coulomb Unit Weight 17.3 Cohesion 25 Phi 0	Soil 10 - Bedrock Soil Model Bedrock Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 12 metre embankment height, 1.5:1 overall side slope  
 10 metre clay foundation  
 6 metre high, 19 metre long toe berm  
 B12T10C6.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Clay 5  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

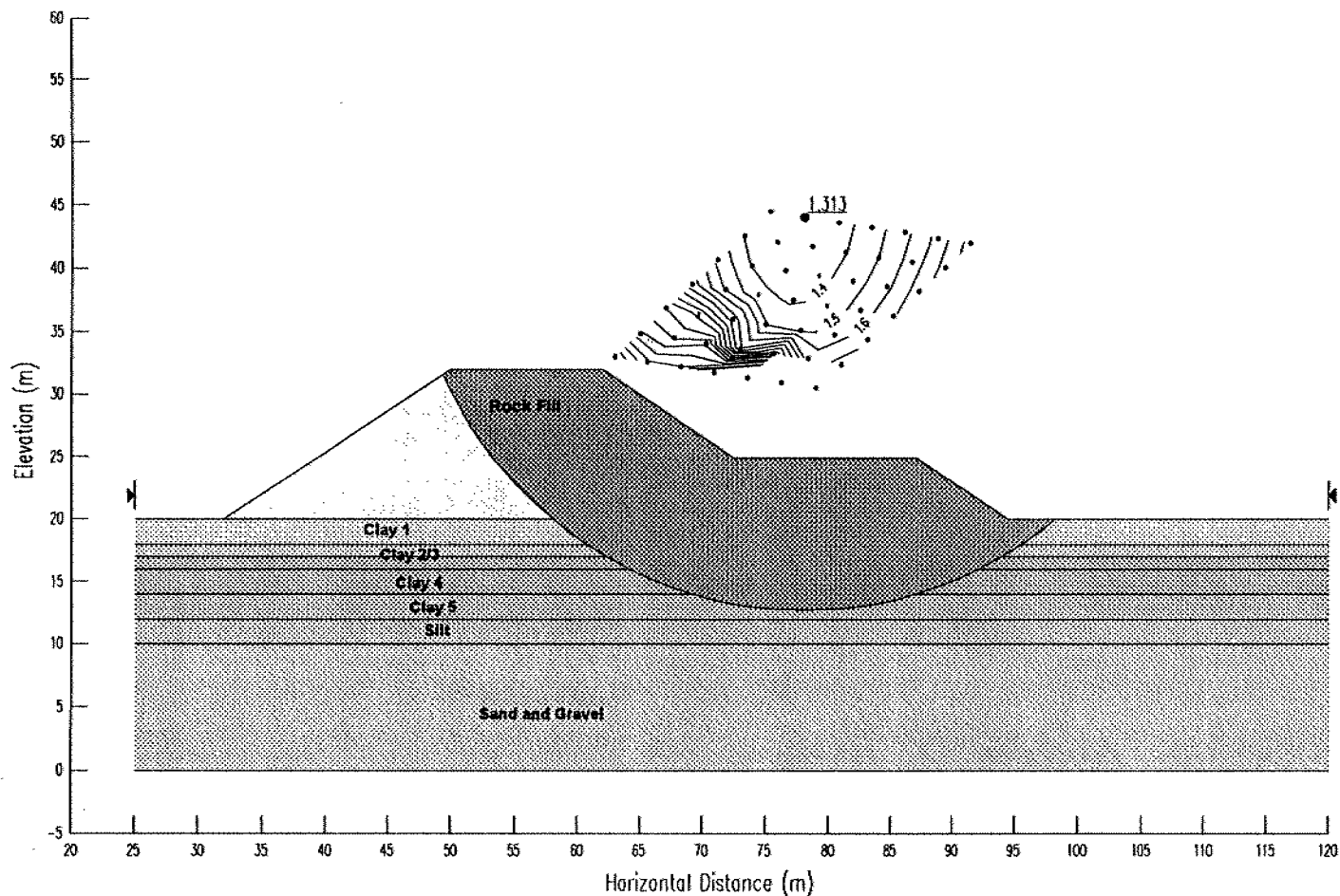
Soil 7 - Clay 6  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 17.8  
 Cv 27.5  
 Rate of Increase 2.5  
 Cv - Maximum 32.5  
 Ch/Cv Ratio 1

Soil 8 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 9 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 10 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 12 metre embankment height, 1.5:1 overall side slope  
 8 metre clay foundation  
 4 metre high, 15 metre long toe berm  
 B12T08C4.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

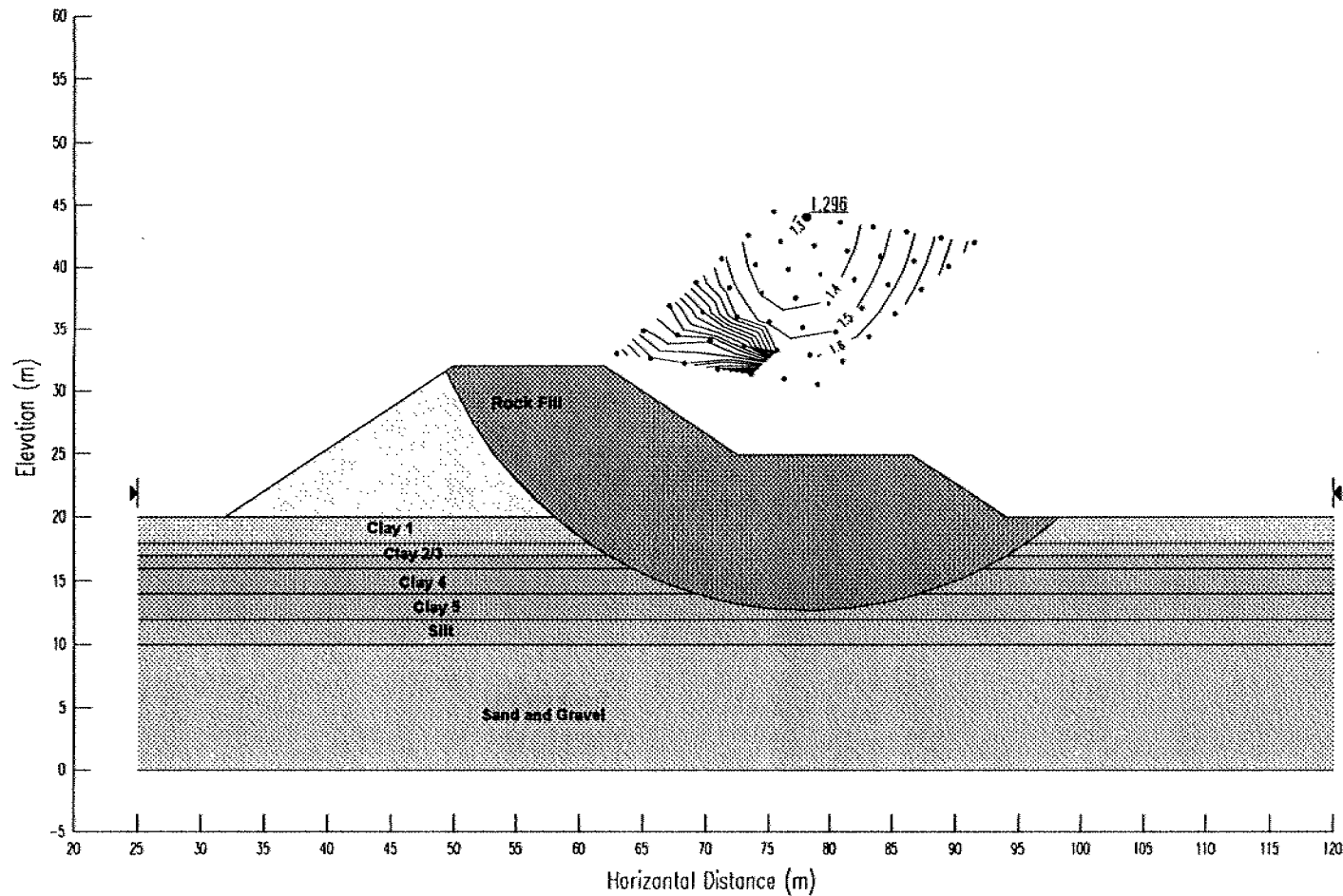
Soil 6 - Clay 5  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

Soil 7 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 8 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 9 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 12 metre embankment height, 1.5:1 overall side slope  
 8 metre clay foundation  
 5 metre high, 14 metre long toe berm  
 B12T08C5.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Clay 5  
 Soil Model S=f(depth)  
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

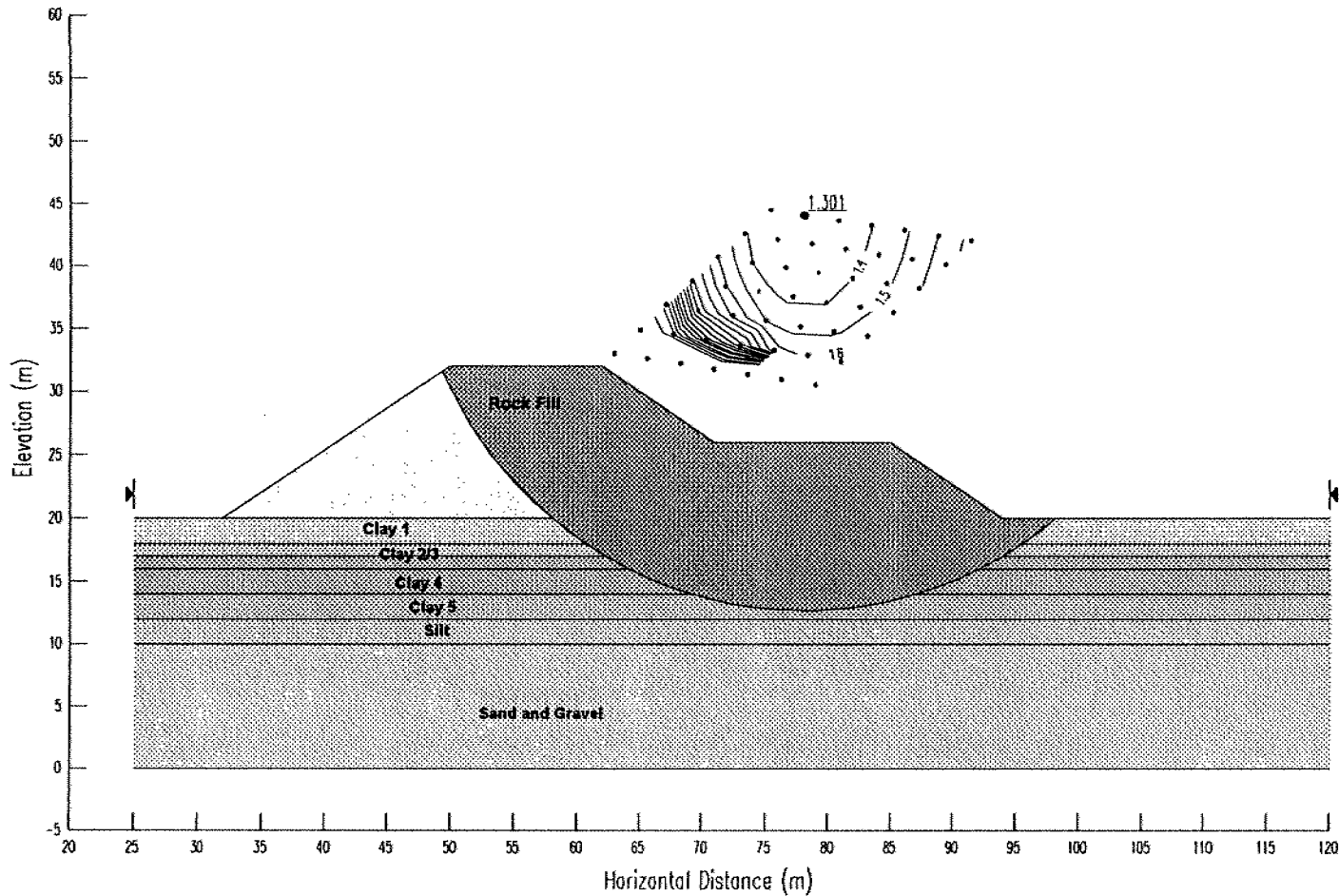
Soil 7 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 8 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 9 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1



Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 12 metre embankment height, 1.5:1 overall side slope  
 8 metre clay foundation  
 6 metre high, 14 metre long toe berm  
 B12T08C6.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

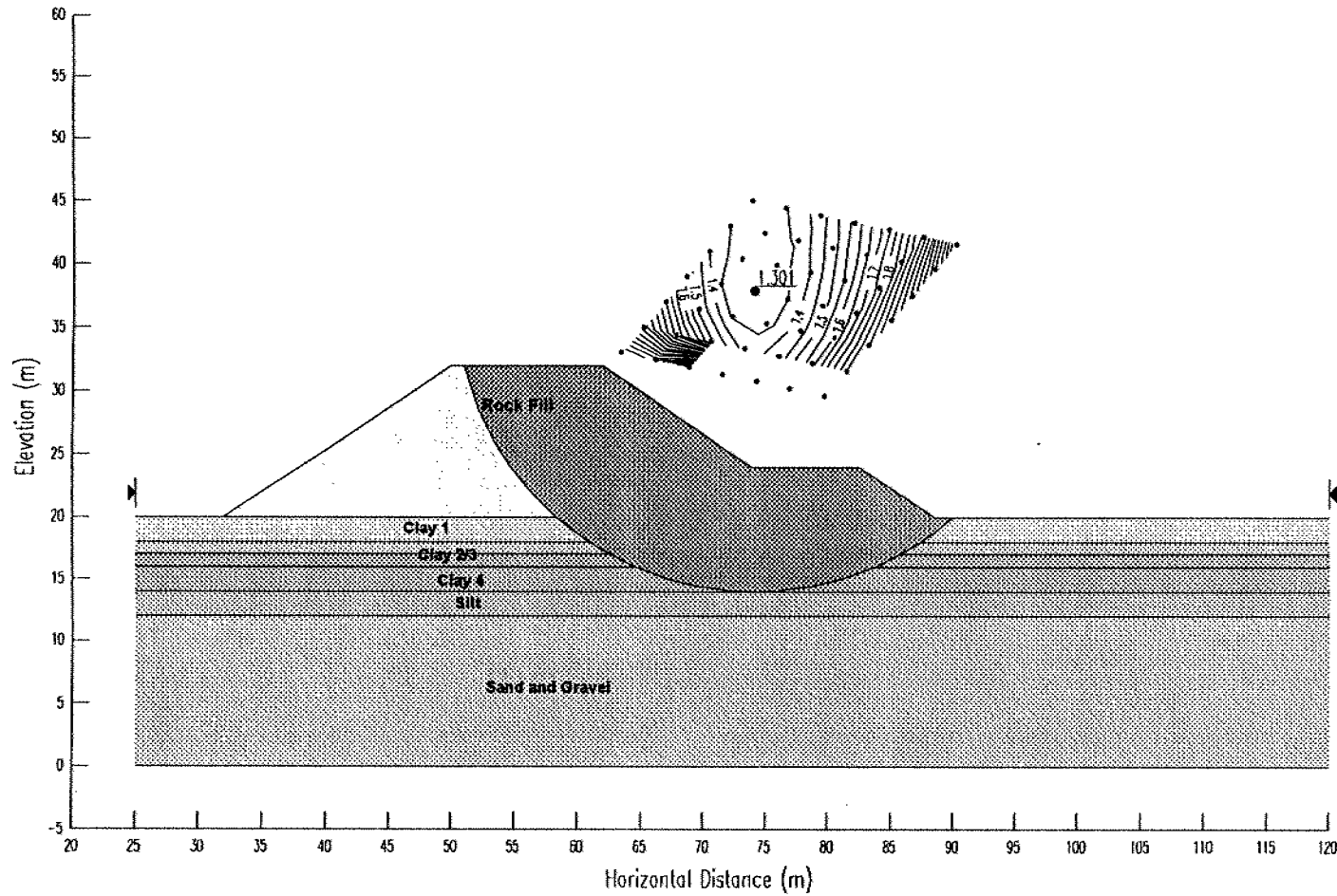
Soil 6 - Clay 5  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 17.8  
 Cv 25  
 Rate of Increase 1.25  
 Cv - Maximum 27.5  
 Ch/Cv Ratio 1

Soil 7 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 8 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 9 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 12 metre embankment height, 1.5:1 overall side slope  
 6 metre clay foundation  
 4 metre high, 8.5 metre long toe berm  
 B12T06CU.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

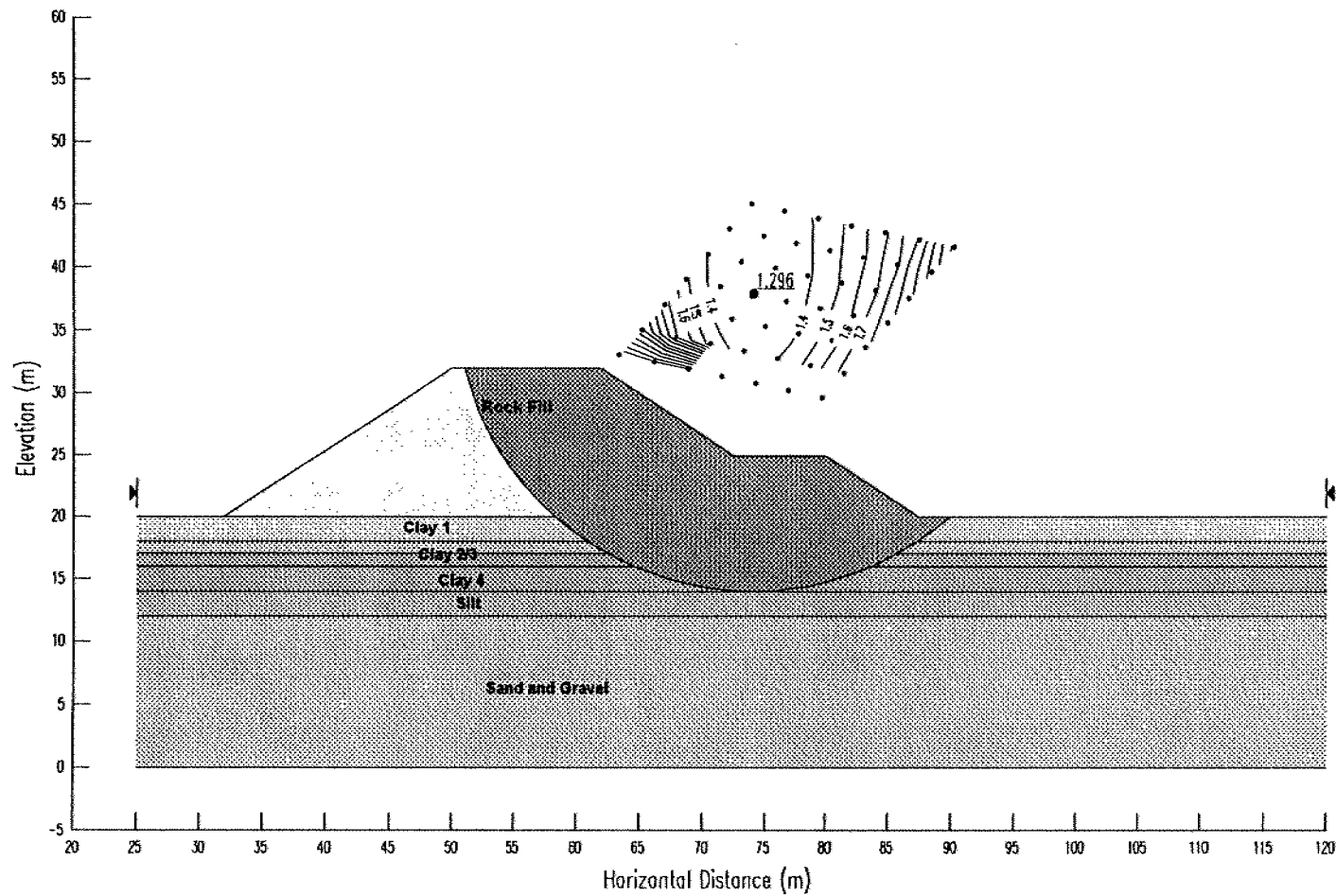
Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 7 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 8 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 12 metre embankment height, 1.5:1 overall side slope  
 6 metre clay foundation  
 5 metre high, 7.5 metre long toe berm  
 B12T06C5.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

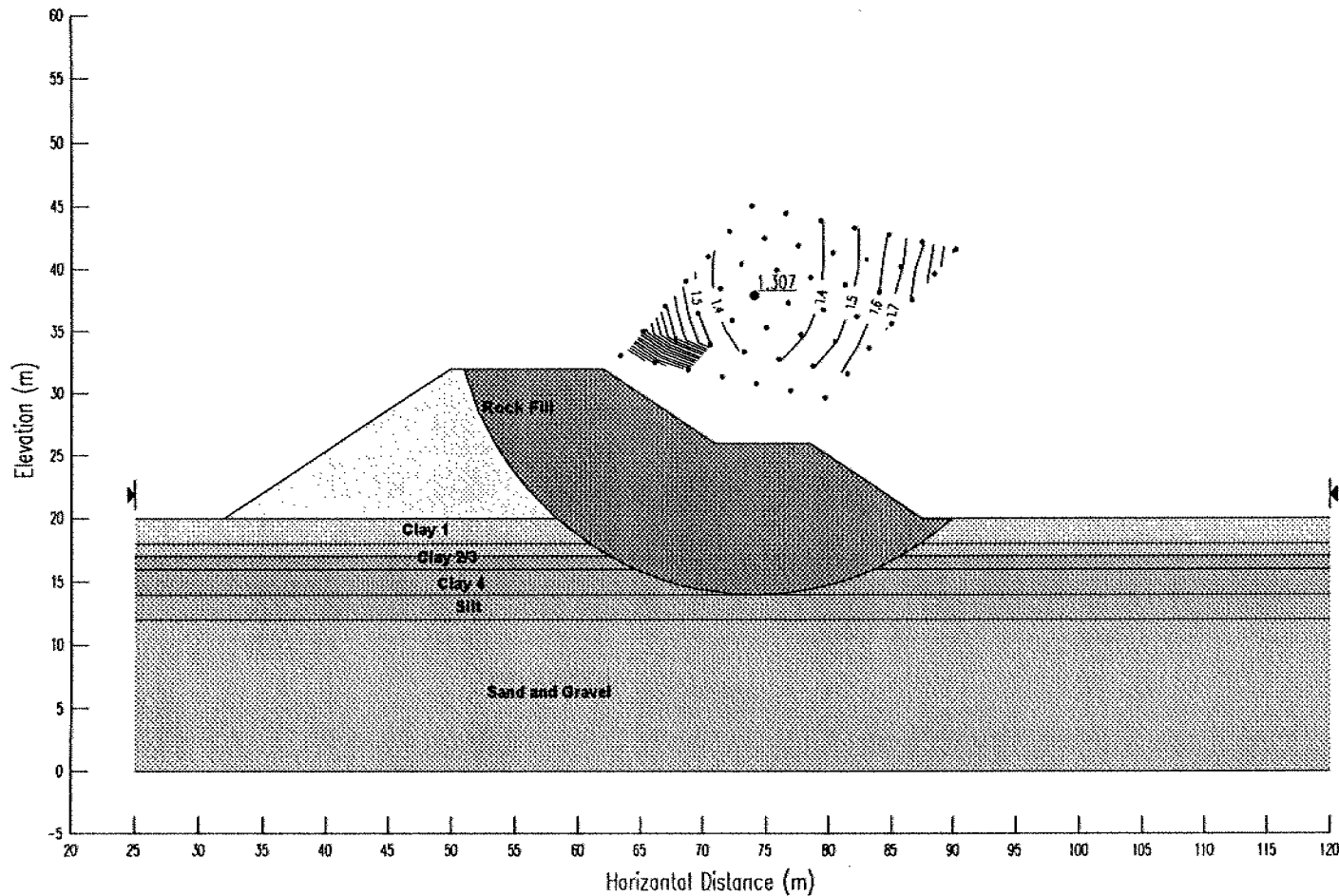
Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 7 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 8 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 12 metre embankment height, 1.5:1 overall side slope  
 6 metre clay foundation  
 6 metre high, 7.5 metre long toe berm  
 B12T06C6.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model S=f(depth)  
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model S=f(depth)  
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model S=f(depth)  
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

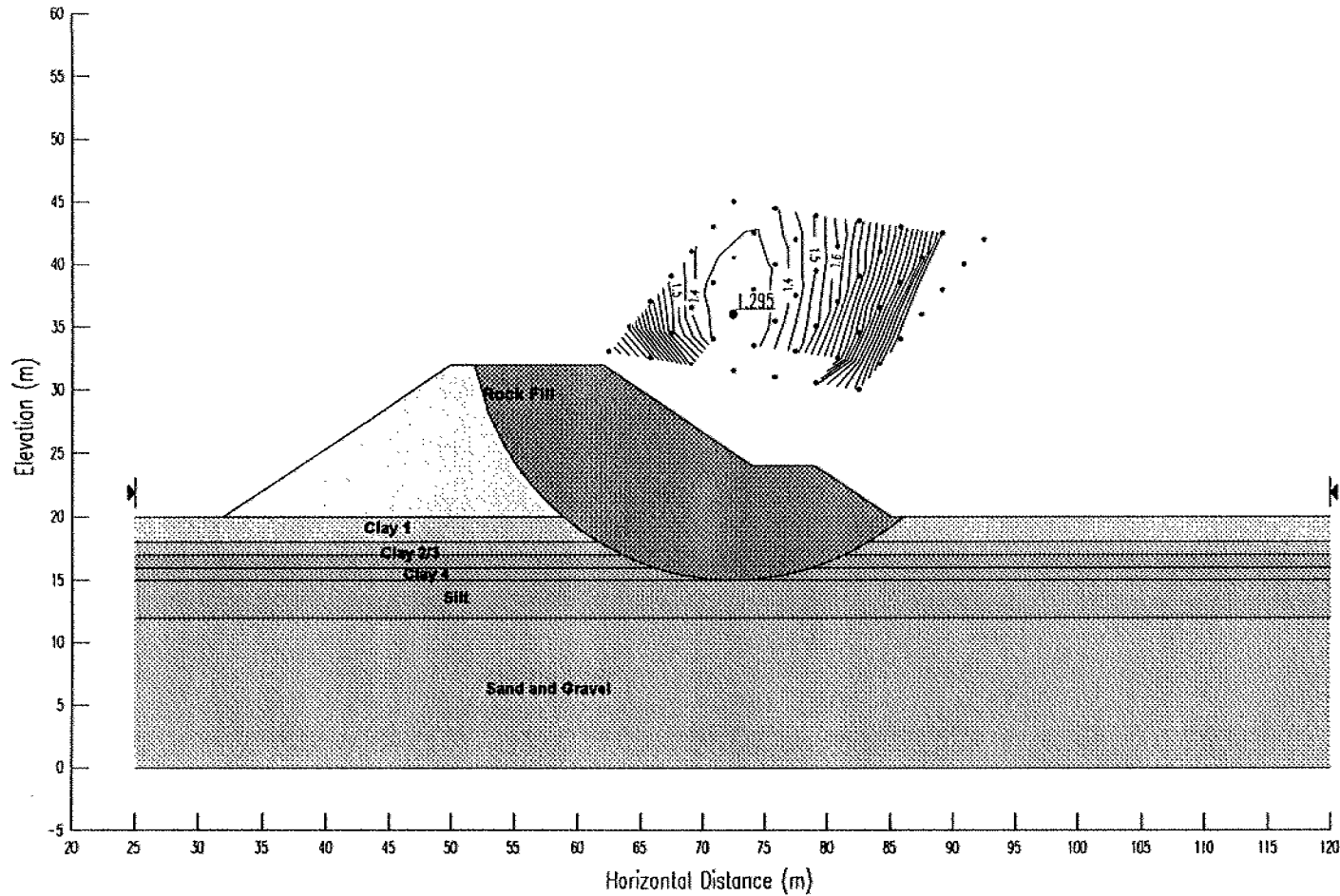
Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 7 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 8 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Total Stress Analysis  
 12 metre embankment height, 1.5:1 overall side slope  
 5 metre clay foundation  
 4 metre high, 5 metre long toe berm  
 B12T05CU.SLP



Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 20.5  
 Cv 120  
 Rate of Increase -35  
 Cv - Minimum 50  
 Ch/Cv Ratio 1

Soil 3 - Clay 2  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 18.3  
 Cv 50  
 Rate of Increase -20  
 Cv - Minimum 30  
 Ch/Cv Ratio 1

Soil 4 - Clay 3  
 Soil Model  $S=f(\text{depth})$   
 Unit Weight 18  
 Cv 30  
 Rate of Increase -5  
 Cv - Minimum 25  
 Ch/Cv Ratio 1

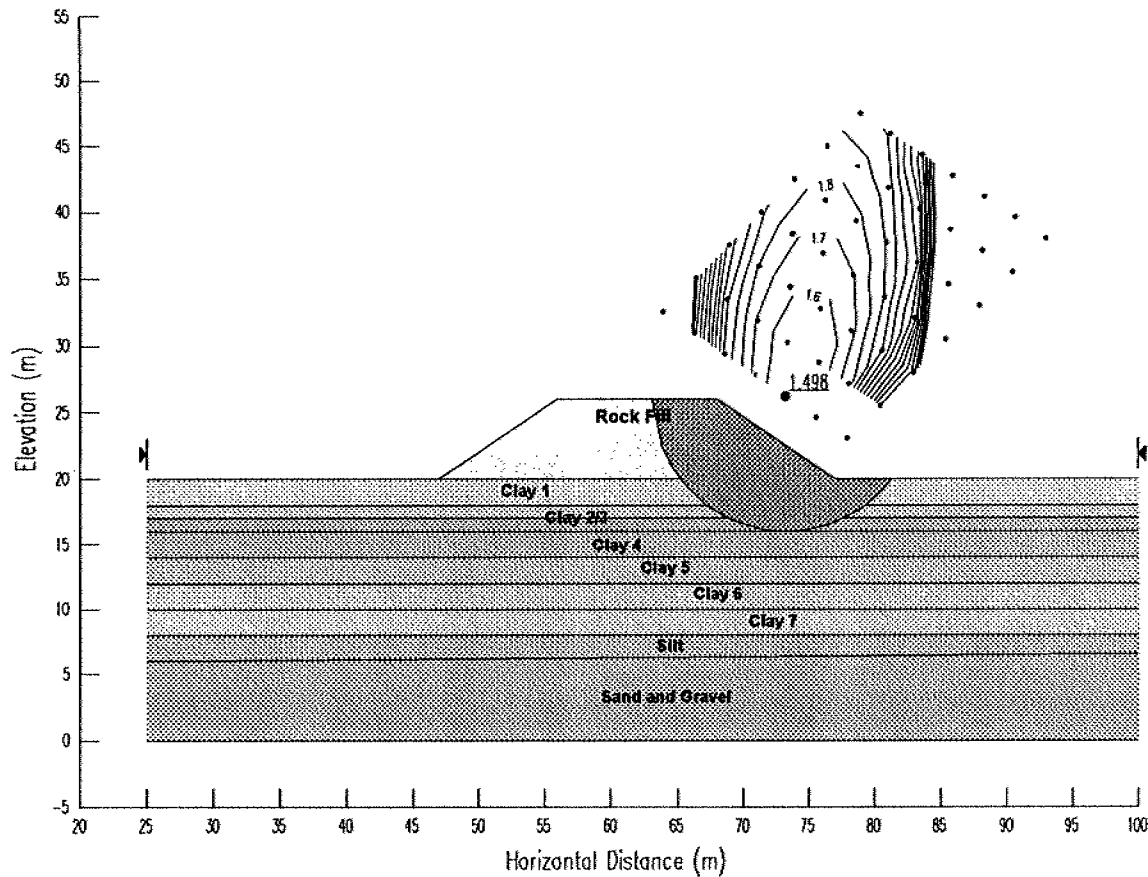
Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 25  
 Phi 0

Soil 6 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 7 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 8 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 6 metre embankment height, 1.5:1 overall side slope  
 12 metre clay foundation  
 H06T12E3.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 7 - Clay 6  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

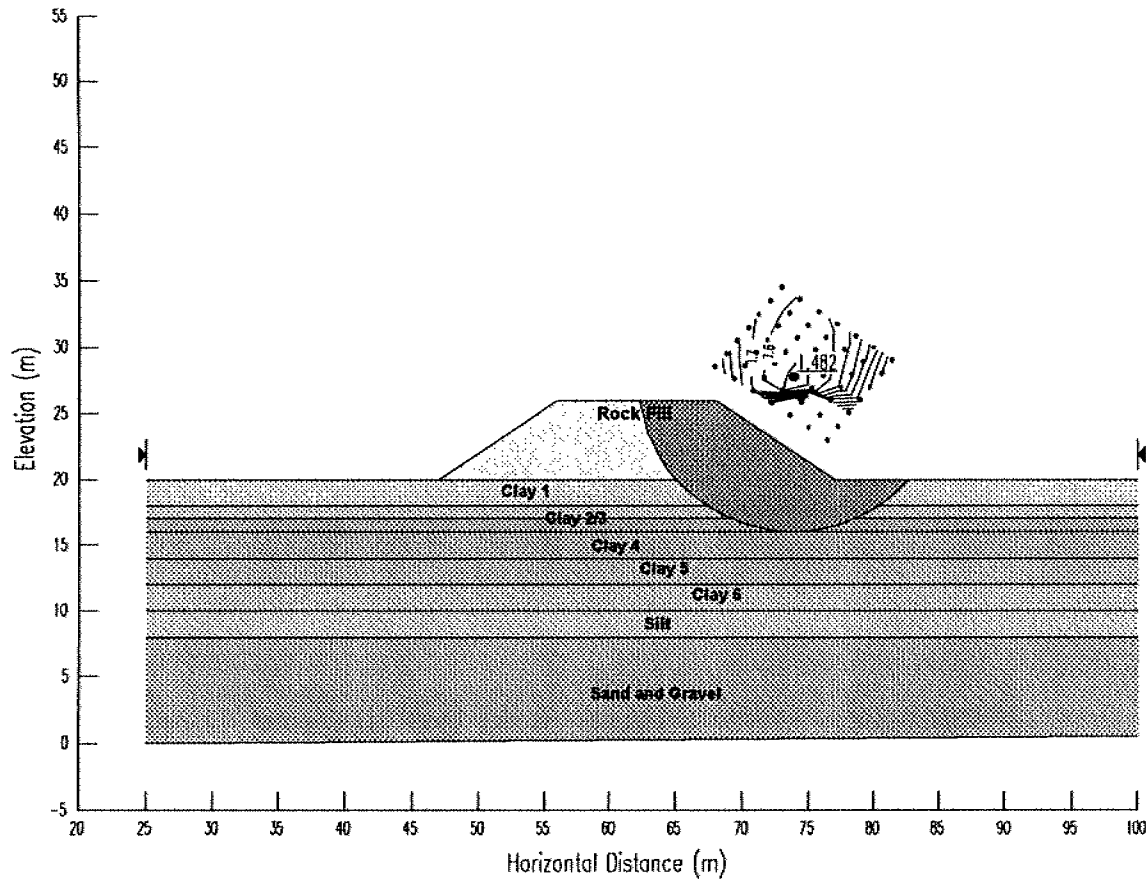
Soil 8 - Clay 7  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 9 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 10 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 11 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 6 metre embankment height, 1.5:1 overall side slope  
 10 metre clay foundation  
 H06T10E3.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

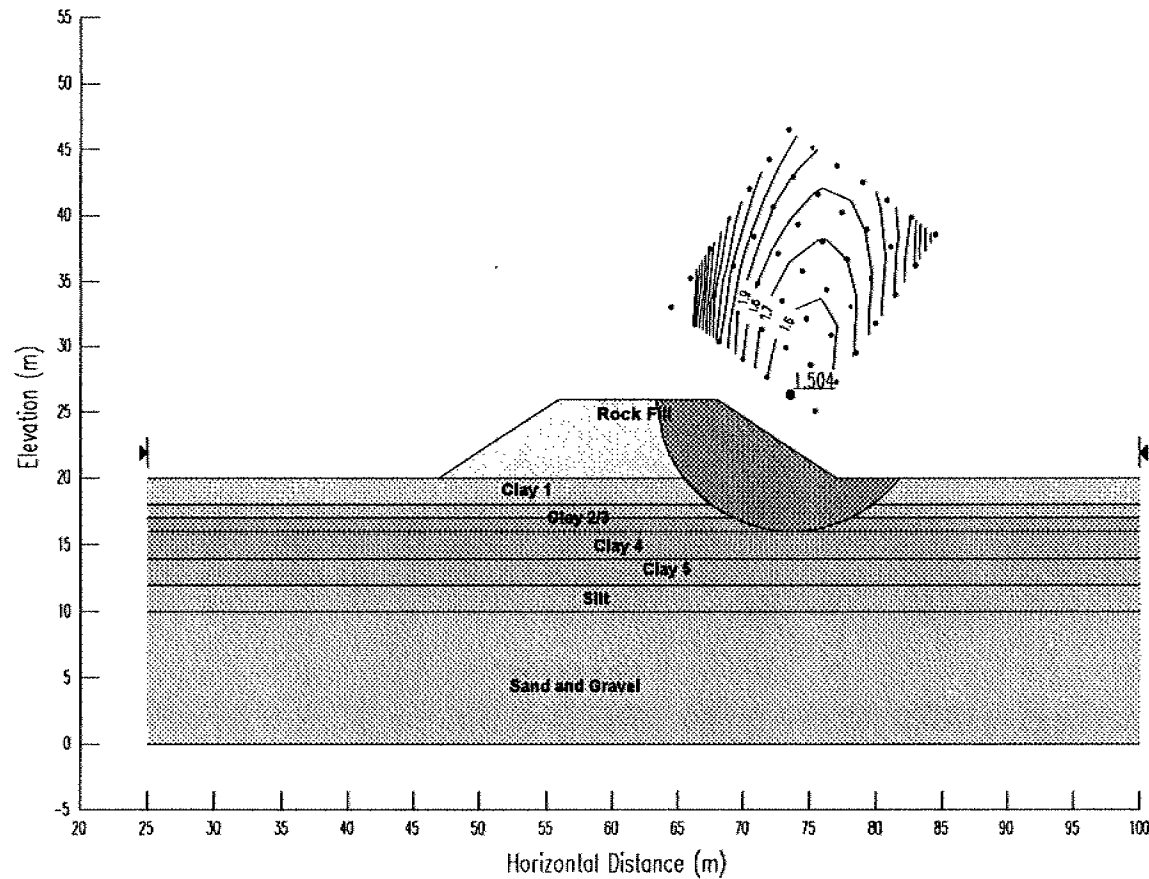
Soil 7 - Clay 6  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 8 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 9 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 10 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 6 metre embankment height, 1.5:1 overall side slope  
 8 metre clay foundation  
 H06T08E3.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

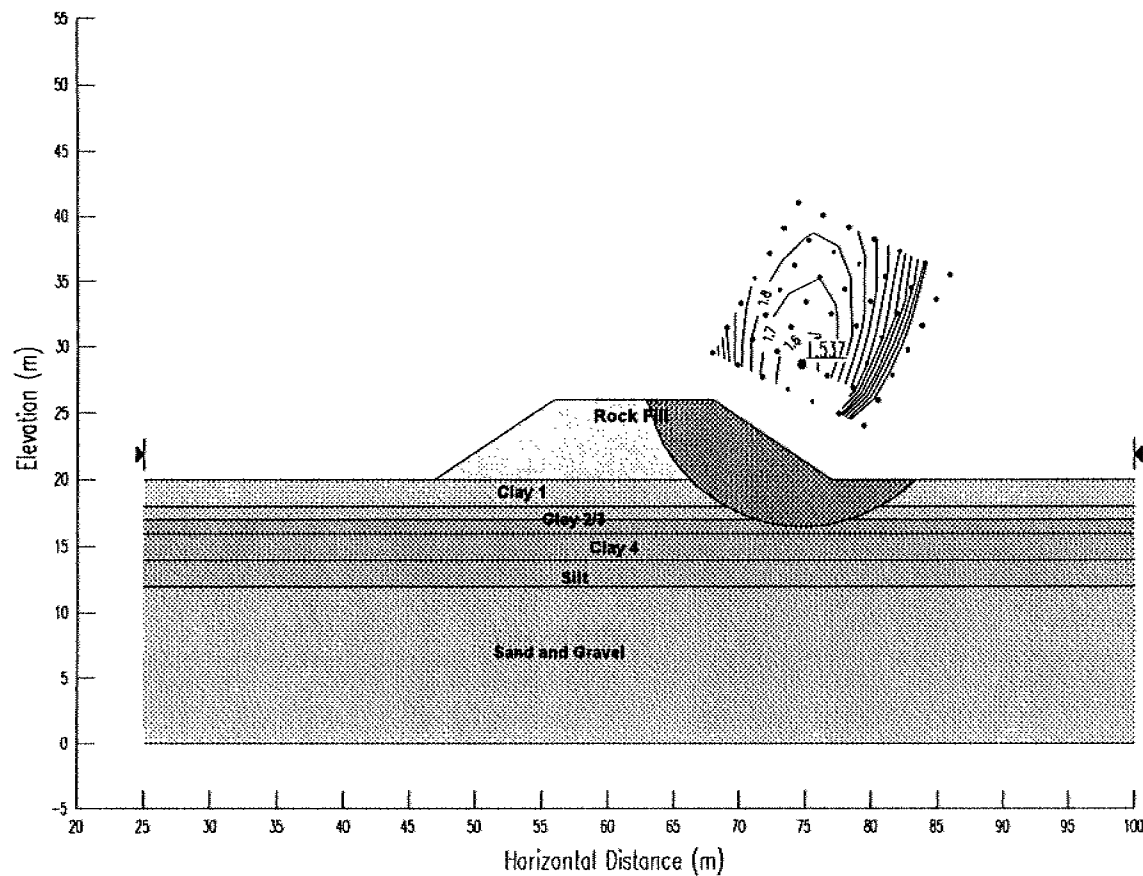
Soil 7 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 8 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 9 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1



Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 6 metre embankment height, 1.5:1 overall side slope  
 6 metre clay foundation  
 H06T06E3.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

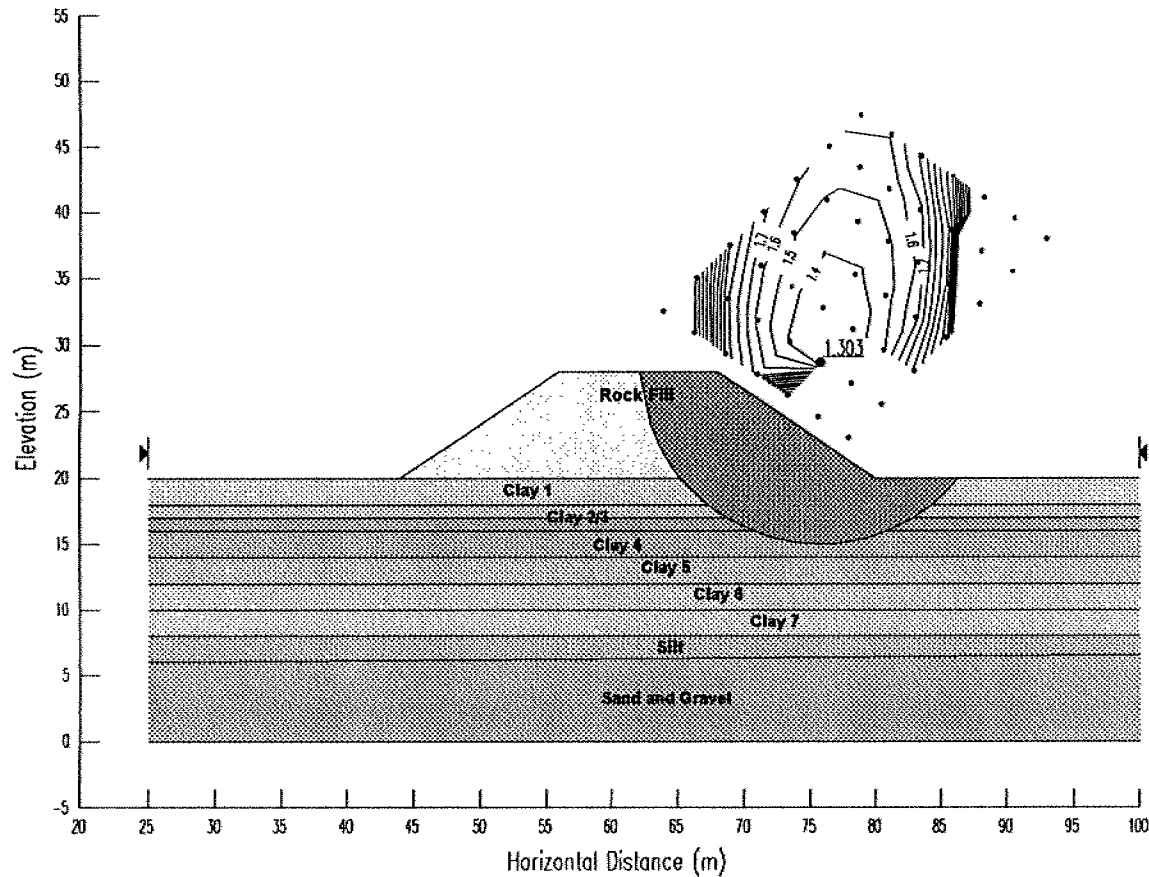
Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 7 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 8 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 8 metre embankment height, 1.5:1 overall side slope  
 12 metre clay foundation  
 H08T12E3.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 7 - Clay 6  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

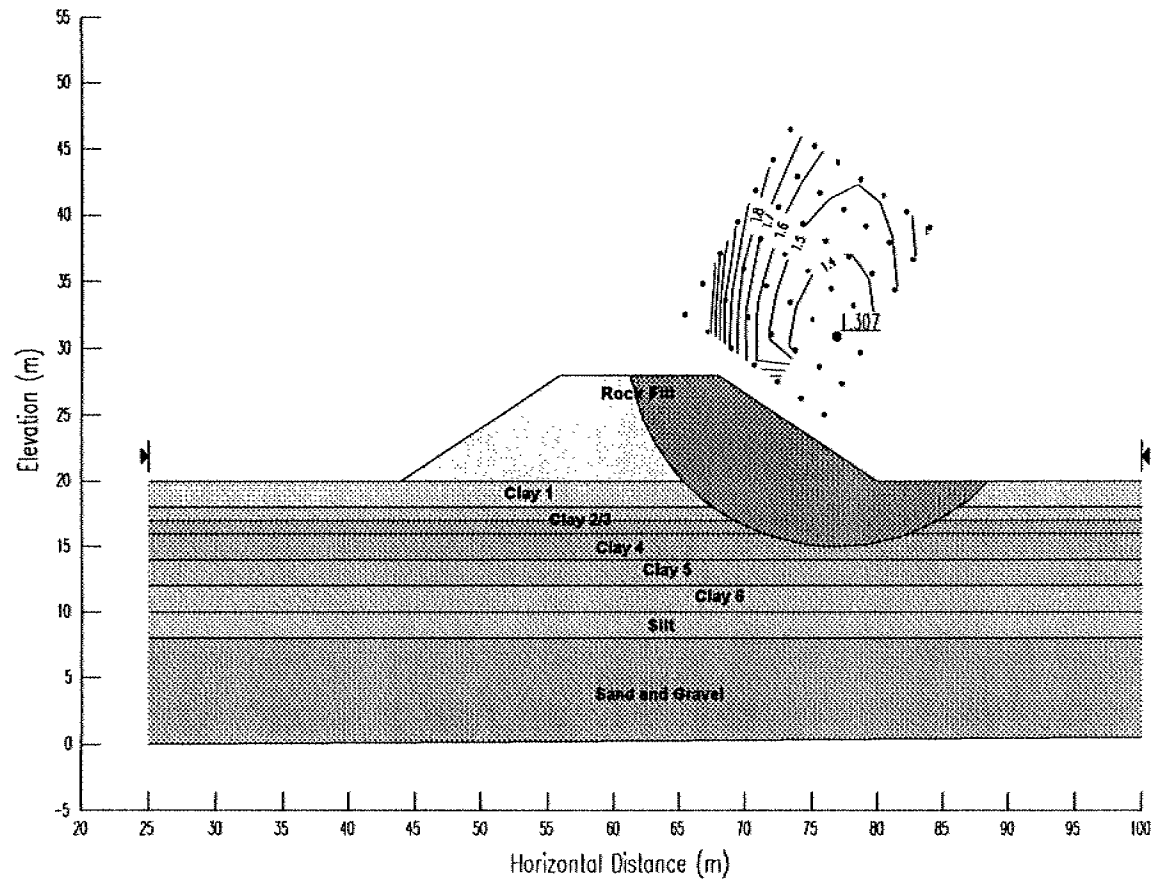
Soil 8 - Clay 7  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 9 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 10 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 11 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 8 metre embankment height, 1.5:1 overall side slope  
 10 metre clay foundation  
 H08T10E3.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

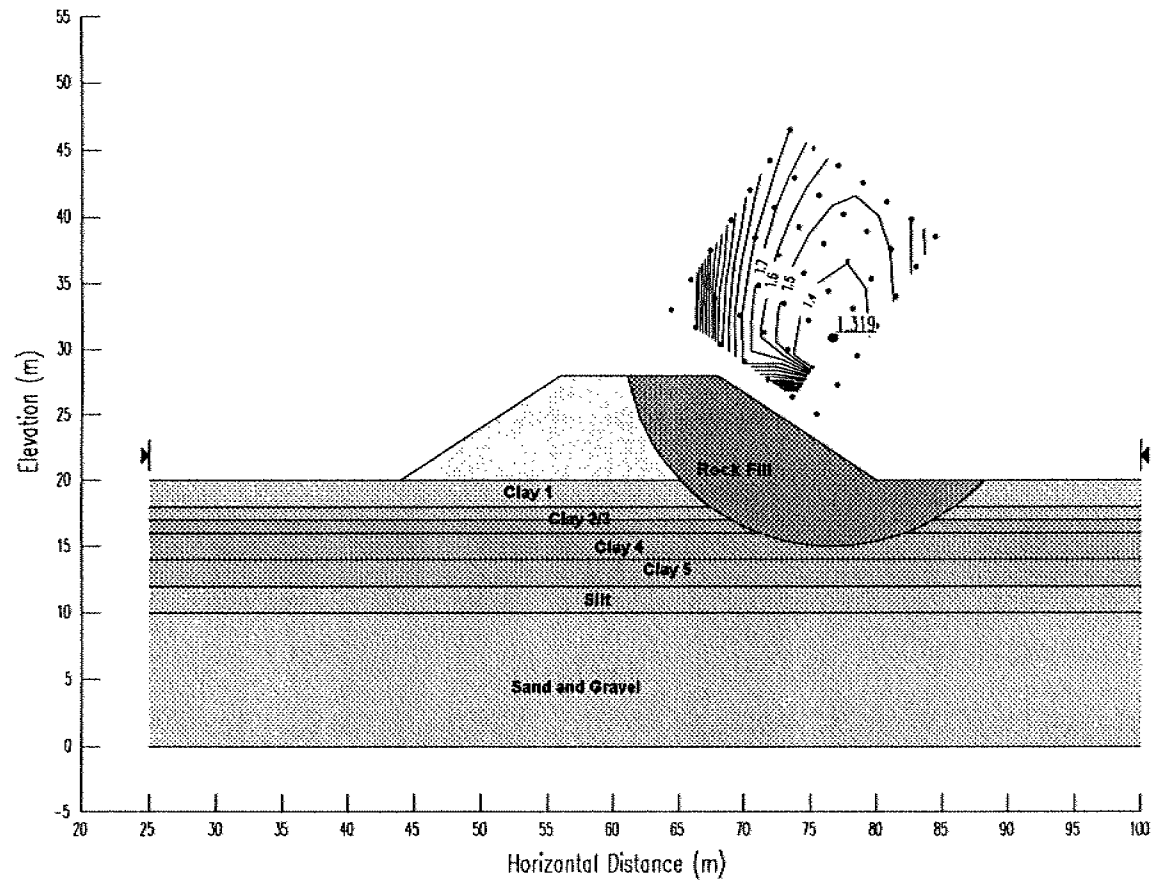
Soil 7 - Clay 6  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 8 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 9 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 10 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 8 metre embankment height, 1.5:1 overall side slope  
 8 metre clay foundation  
 H08T08E3.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

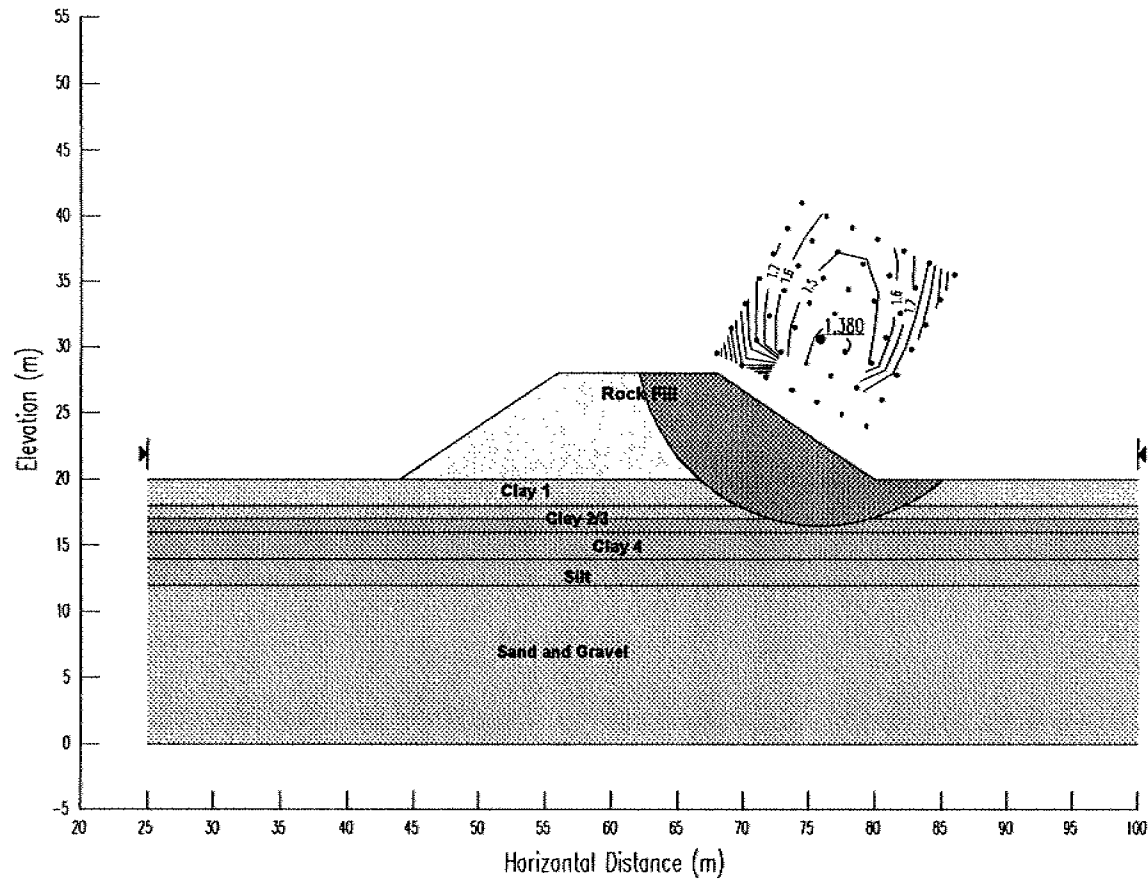
Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 7 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 8 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 9 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 8 metre embankment height, 1.5:1 overall side slope  
 6 metre clay foundation  
 H08T06E3.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

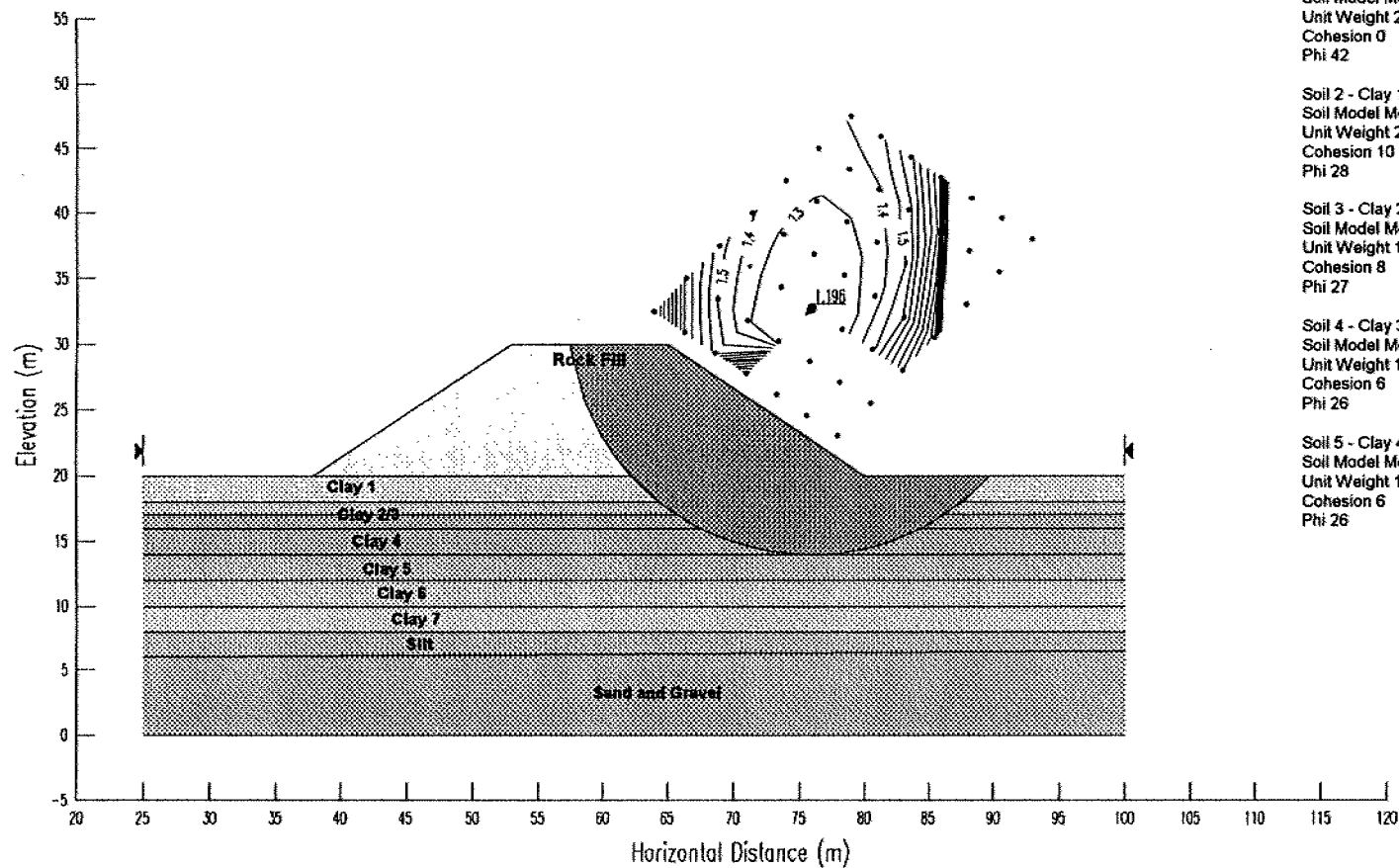
Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 7 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 8 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 10 metre embankment height, 1.5:1 overall side slope  
 12 metre clay foundation  
 H10T12E3.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 7 - Clay 6  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

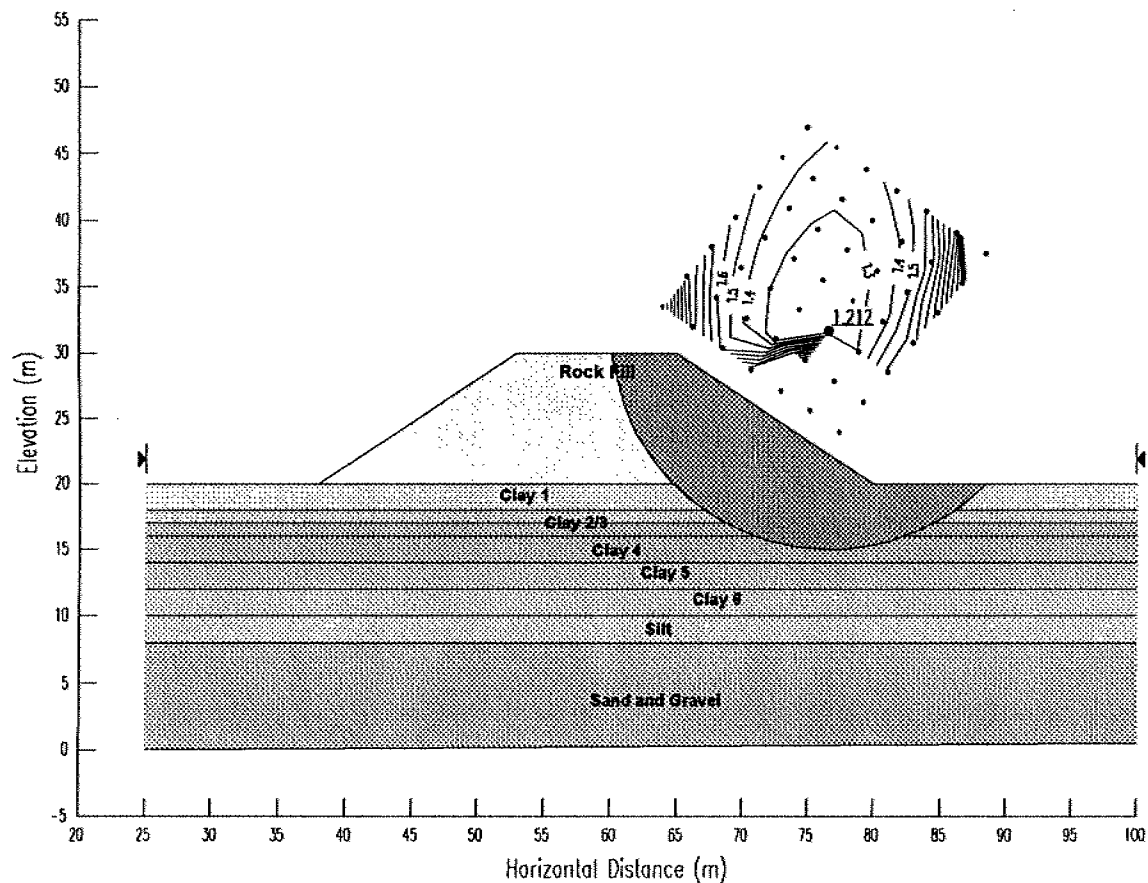
Soil 8 - Clay 7  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 9 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 10 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 11 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 10 metre embankment height, 1.5:1 overall side slope  
 10 metre clay foundation  
 H10T10E3.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

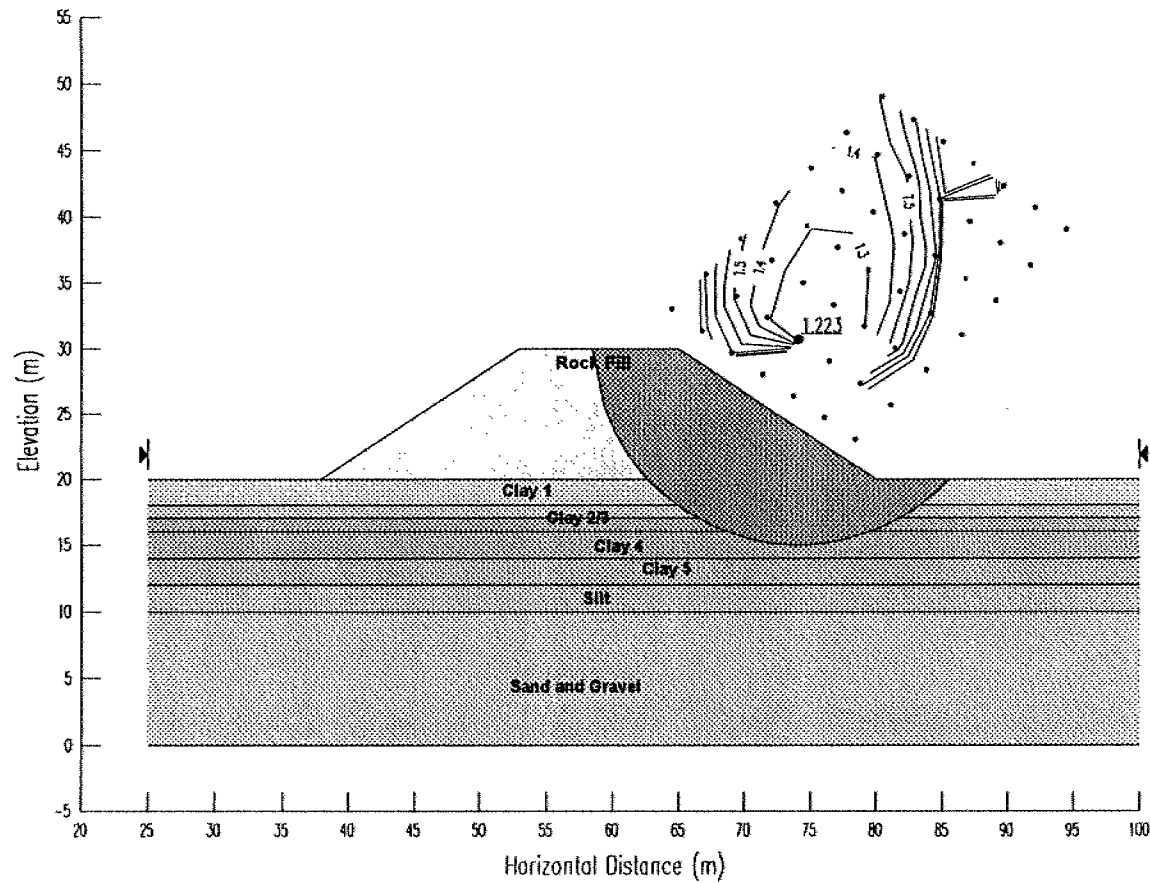
Soil 7 - Clay 6  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 8 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 9 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 10 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 10 metre embankment height, 1.5:1 overall side slope  
 8 metre clay foundation  
 H10T08E3.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

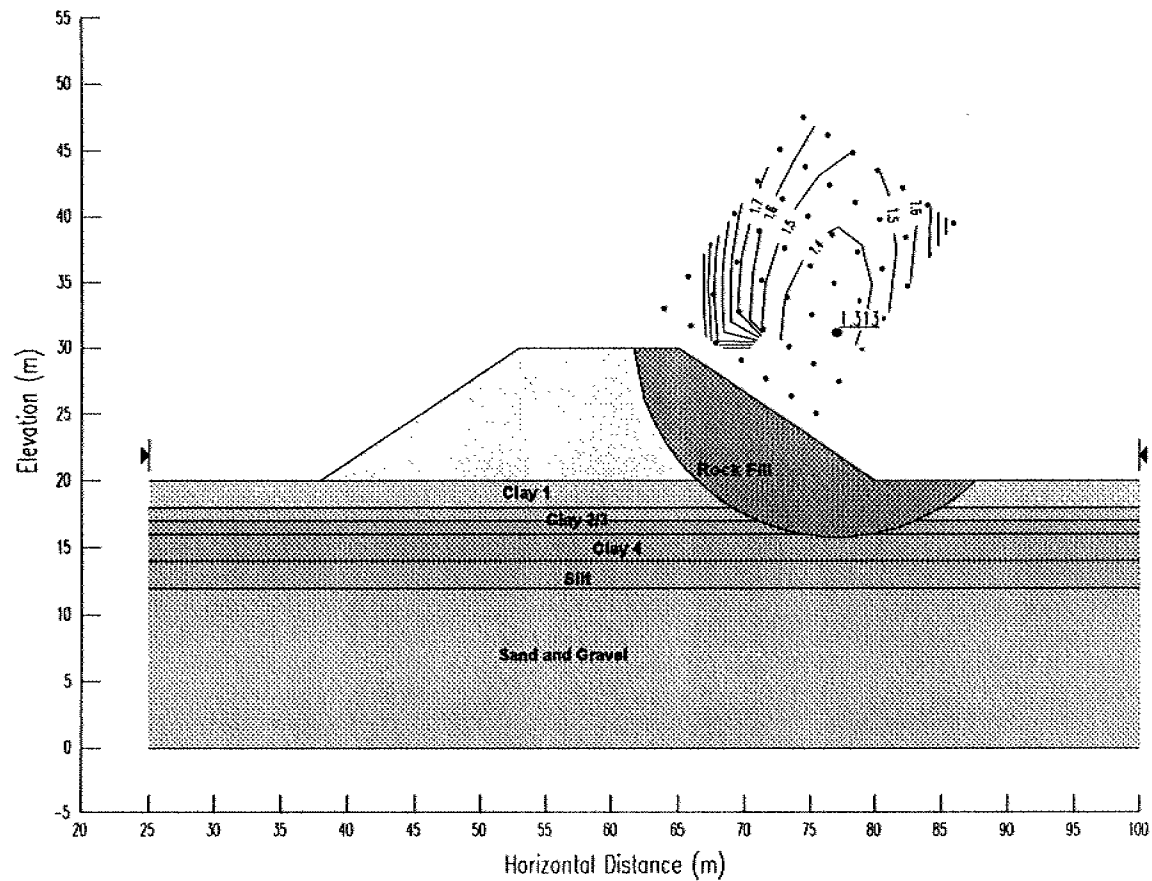
Soil 7 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 8 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 9 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1



Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 10 metre embankment height, 1.5:1 overall side slope  
 6 metre clay foundation  
 H10T06E3.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

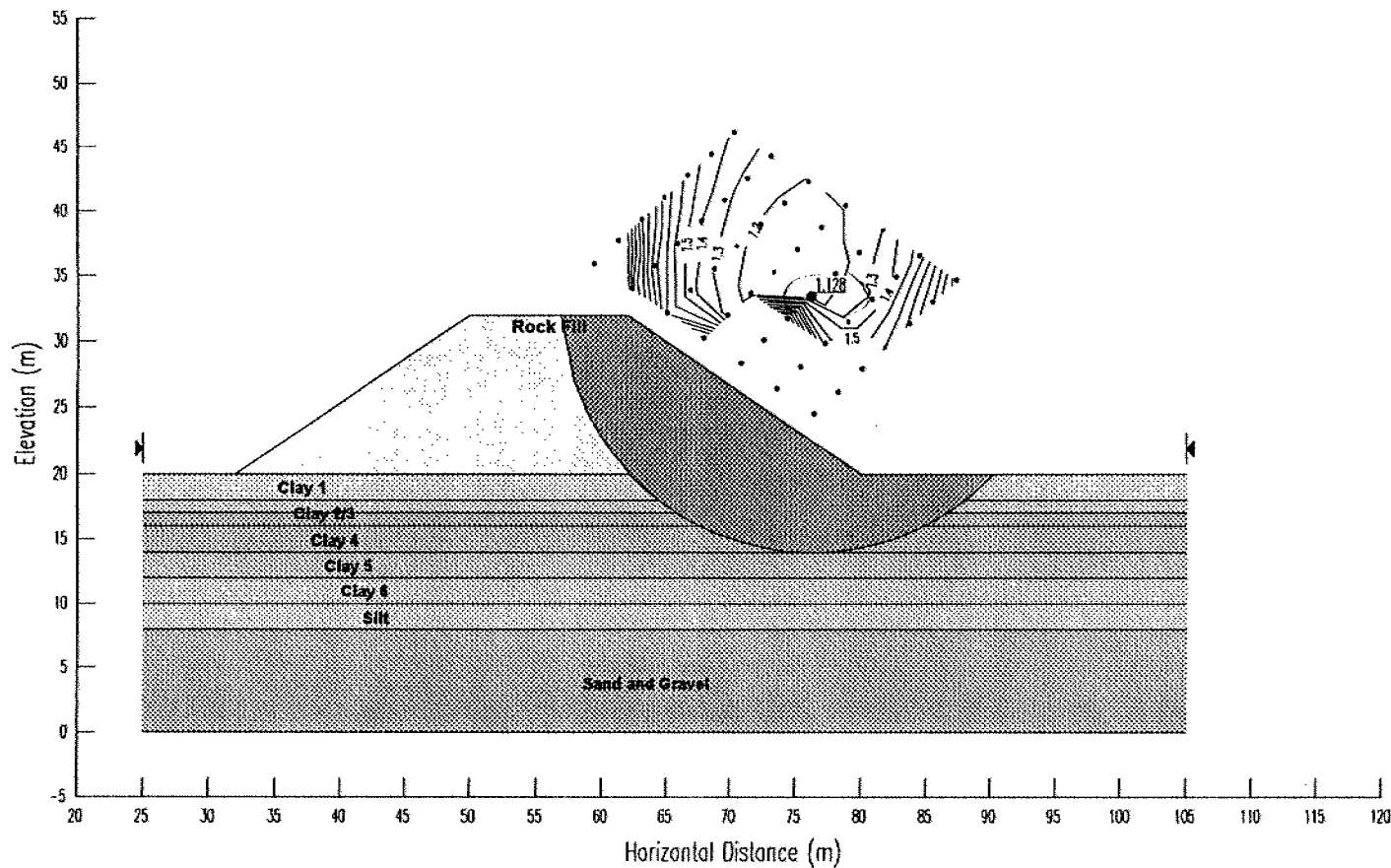
Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 7 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 8 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 12 metre embankment height, 1.5:1 overall side slope  
 10 metre clay foundation  
 H12T10E3.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

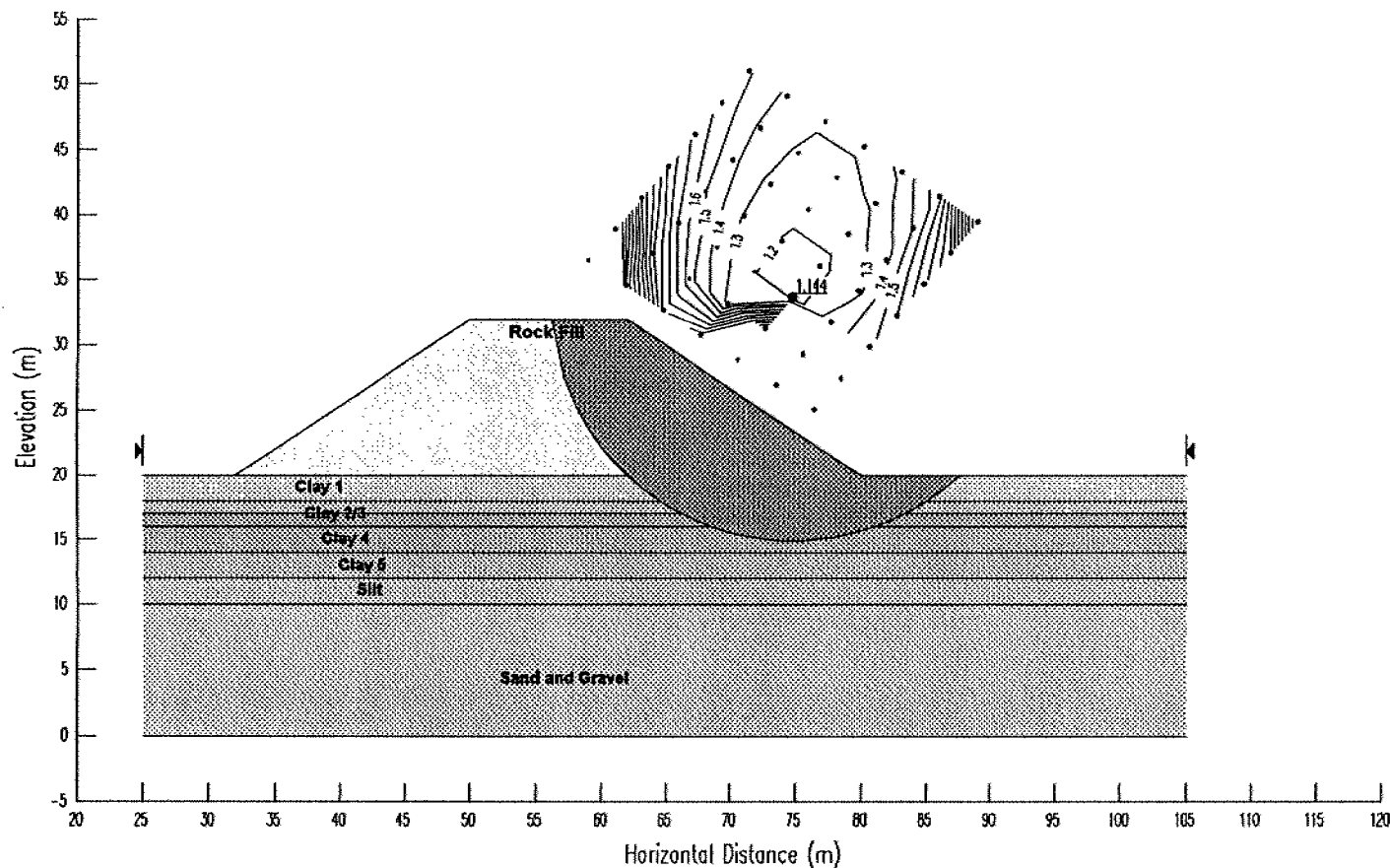
Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 7 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 8 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 9 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 12 metre embankment height, 1.5:1 overall side slope  
 8 metre clay foundation  
 H12T08E3.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 7 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 8 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 9 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 12 metre embankment height, 1.5:1 overall side slope  
 6 metre clay foundation  
 H12T06E3.SLP

Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

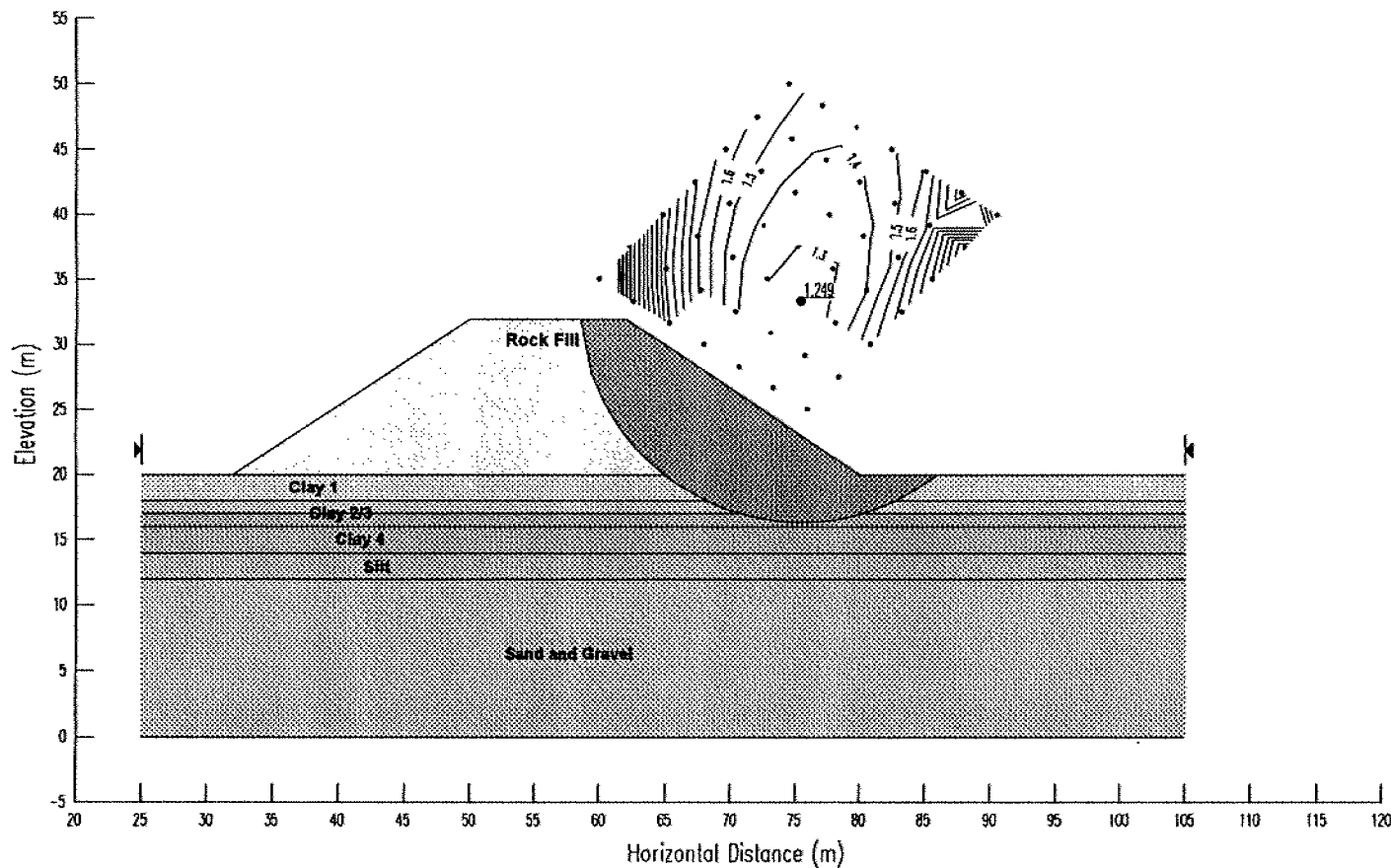
Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

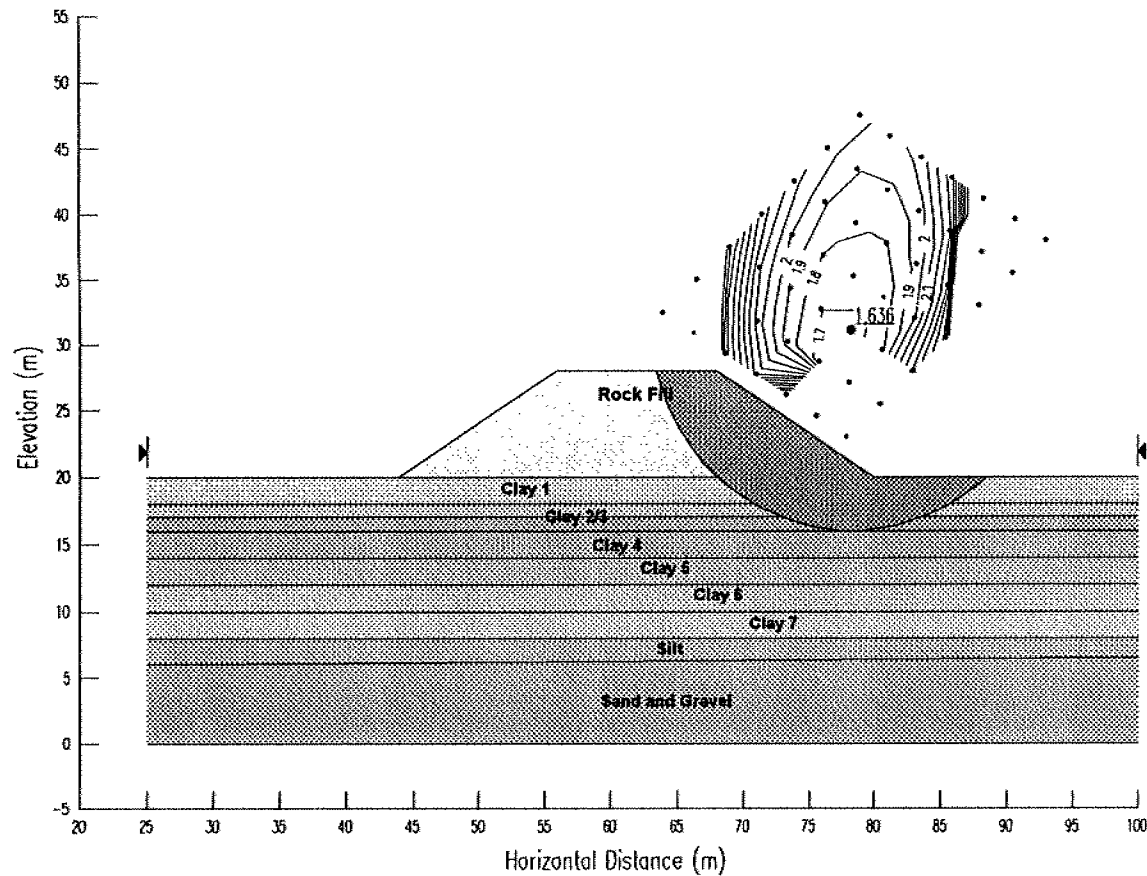
Soil 6 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 7 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 8 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1



Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 with 3m spaced Wick Drains  
 8 metre embankment height, 1.5:1 overall side slope  
 12 metre clay foundation  
 H08T1233.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 7 - Clay 6  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

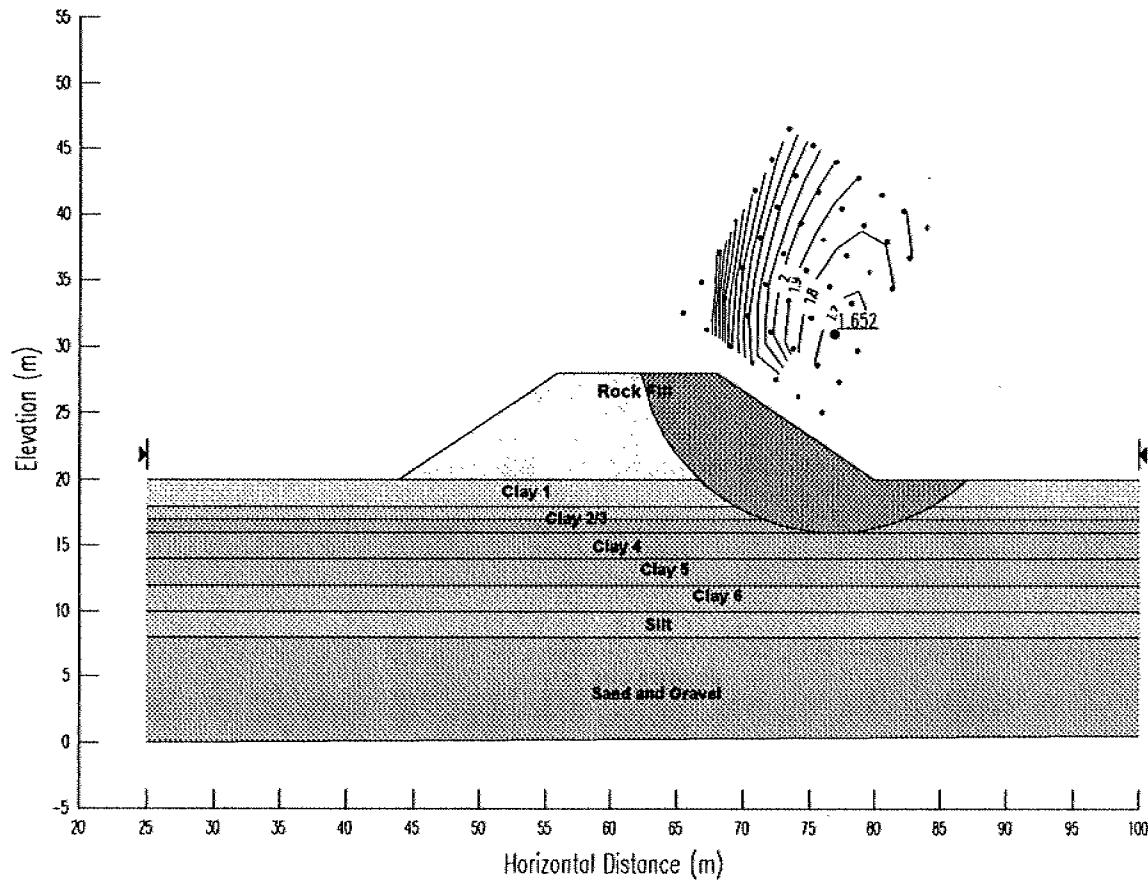
Soil 8 - Clay 7  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 9 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 10 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 11 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179) .  
 Effective Stress Analysis - End of Construction at 3 months  
 with 3m spaced Wick Drains  
 8 metre embankment height, 1.5:1 overall side slope  
 10 metre clay foundation  
 H08T1033.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

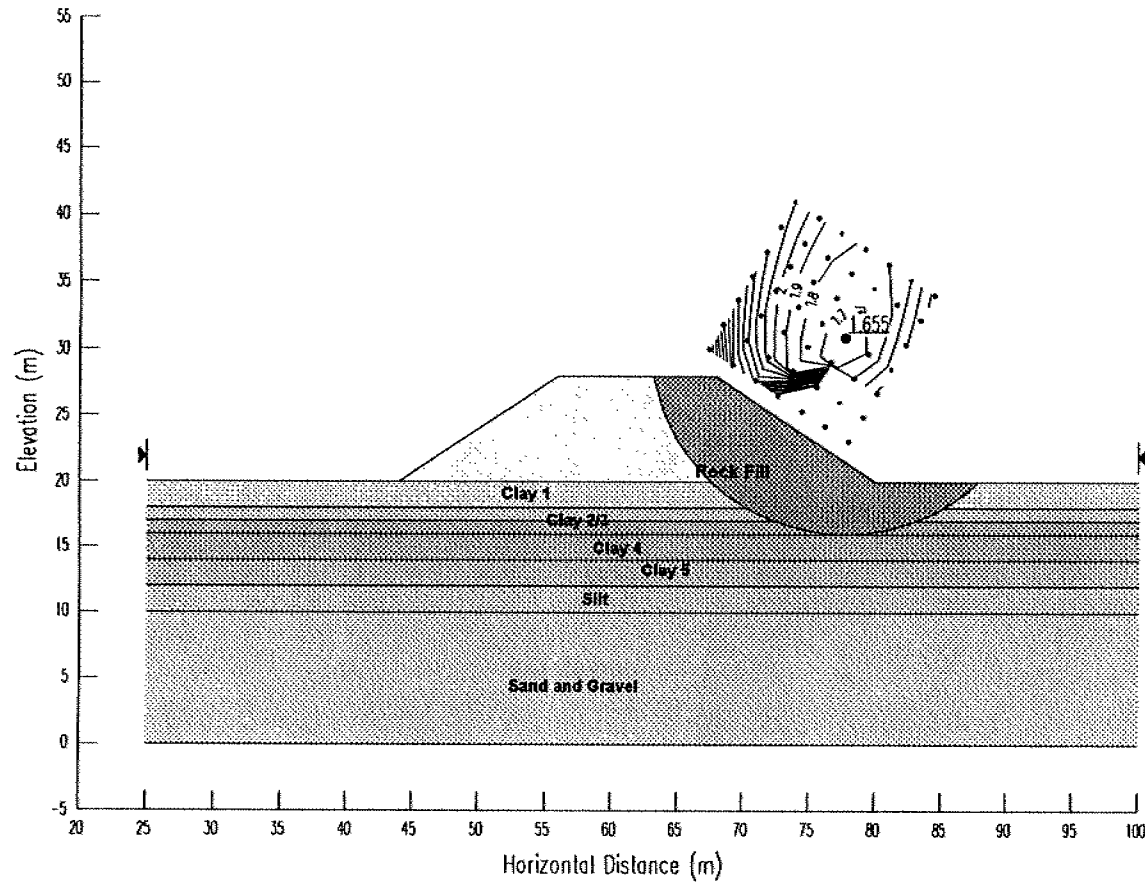
Soil 7 - Clay 6  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 8 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 9 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 10 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 with 3m spaced Wick Drains  
 8 metre embankment height, 1.5:1 overall side slope  
 8 metre clay foundation  
 H08T0833.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

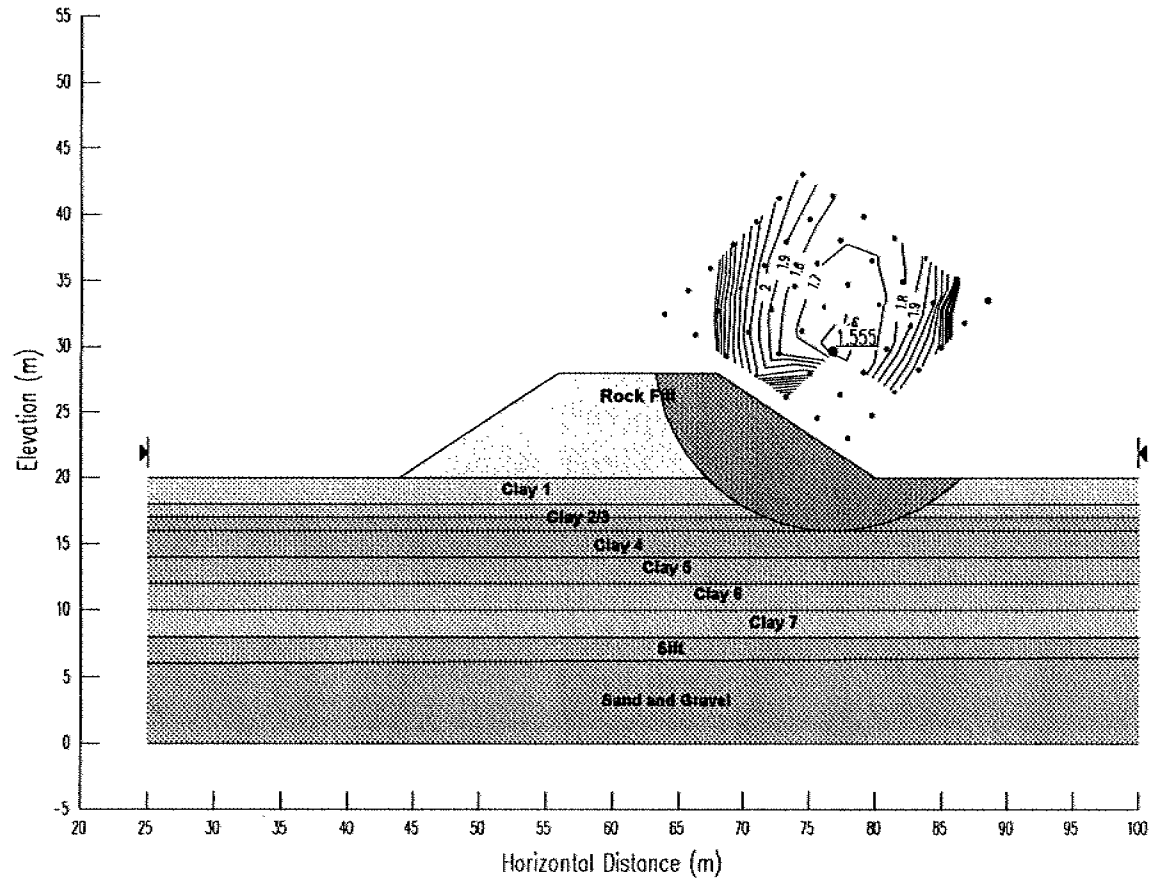
Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 7 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 8 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 9 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 with 4m spaced Wick Drains  
 8 metre embankment height, 1.5:1 overall side slope  
 12 metre clay foundation  
 H08T1243.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.6  
 Cohesion 6  
 Phi 26

Soil 7 - Clay 6  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 8 - Clay 7  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

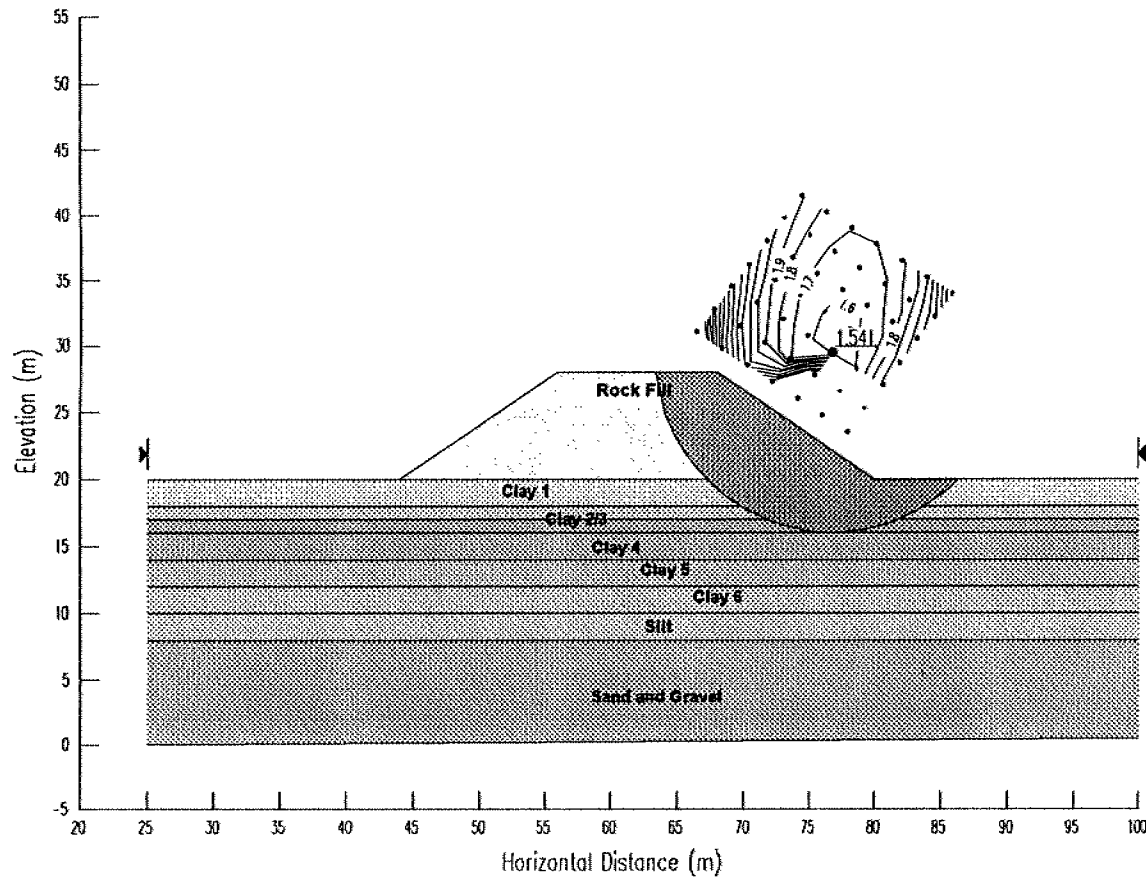
Soil 9 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 10 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 11 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1



Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 with 4m spaced Wick Drains  
 8 metre embankment height, 1.5:1 overall side slope  
 10 metre clay foundation  
 H08T1043.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

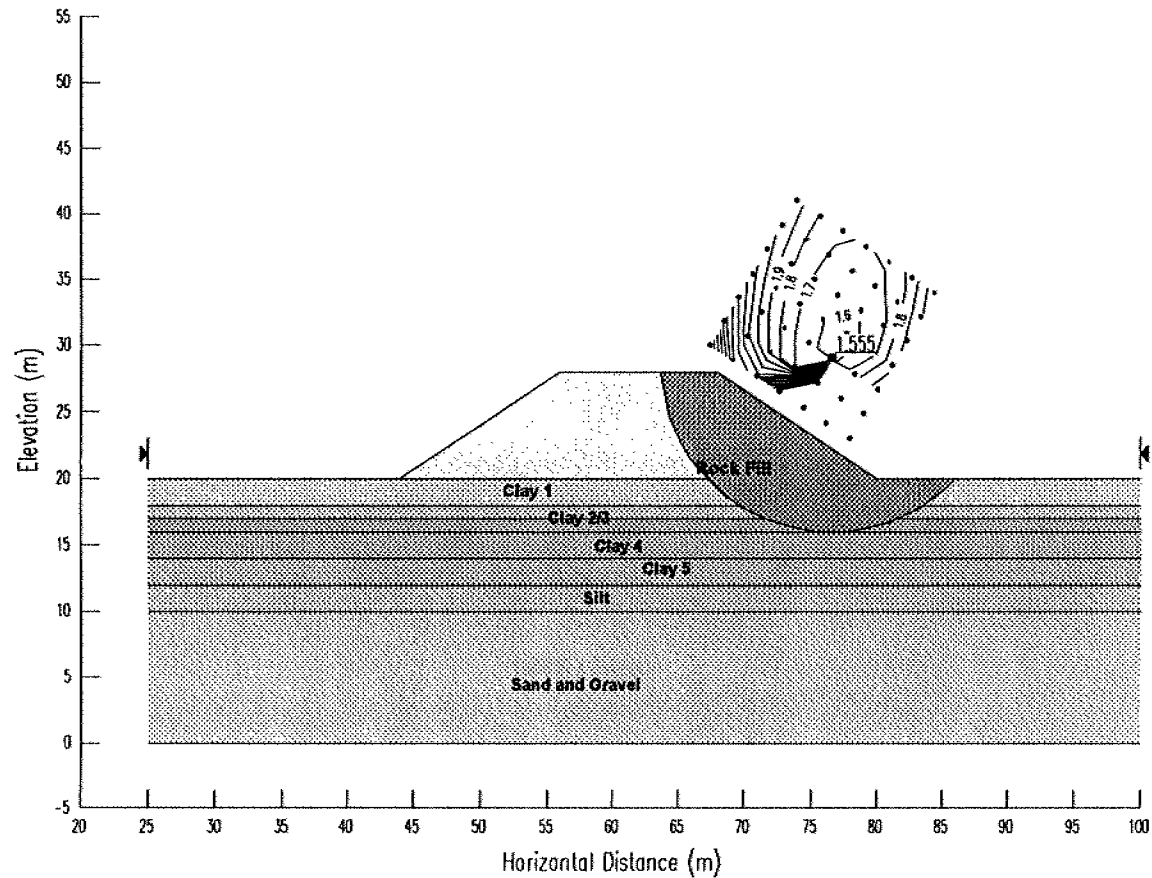
Soil 7 - Clay 6  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 8 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 9 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 10 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 with 4m spaced Wick Drains  
 8 metre embankment height, 1.5:1 overall side slope  
 8 metre clay foundation  
 H08T0843.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

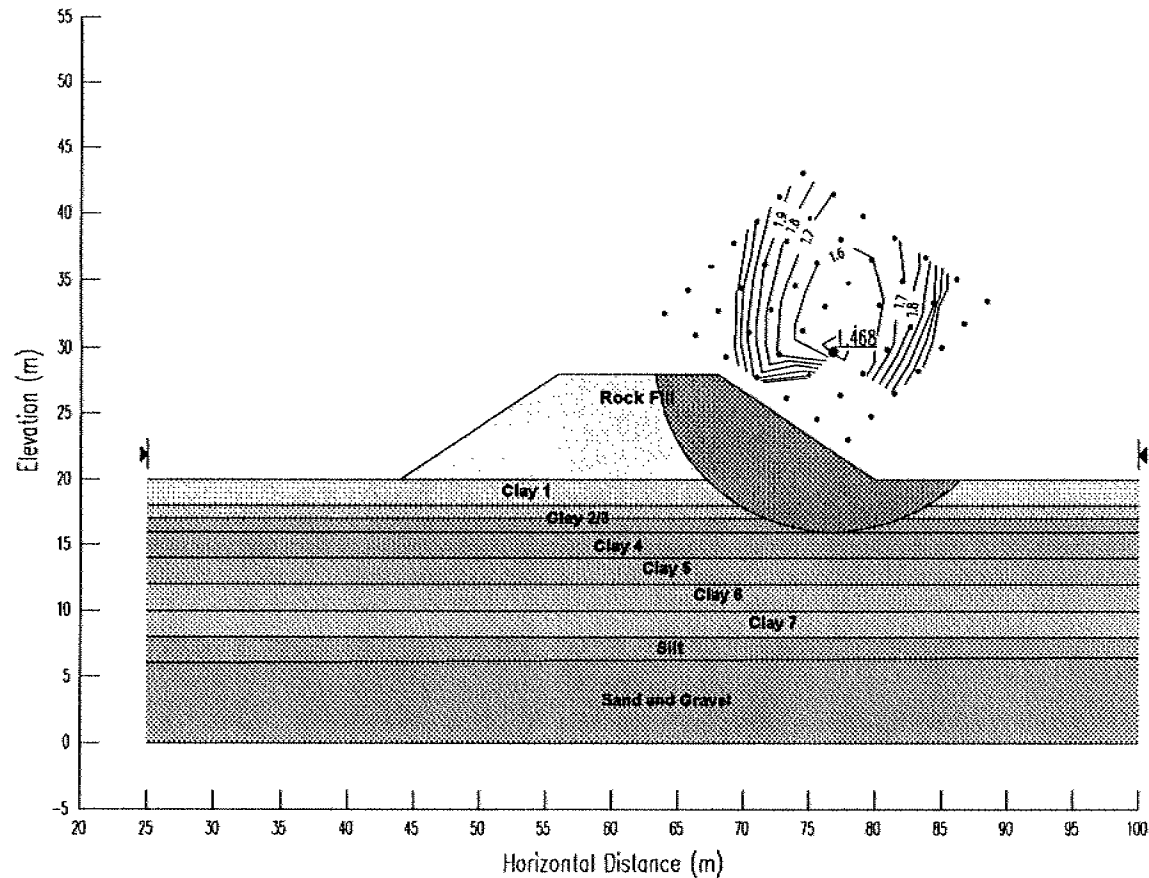
Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 7 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 8 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 9 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 with 5m spaced Wick Drains  
 8 metre embankment height, 1.5:1 overall side slope  
 12 metre clay foundation  
 H08T1253.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 7 - Clay 6  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

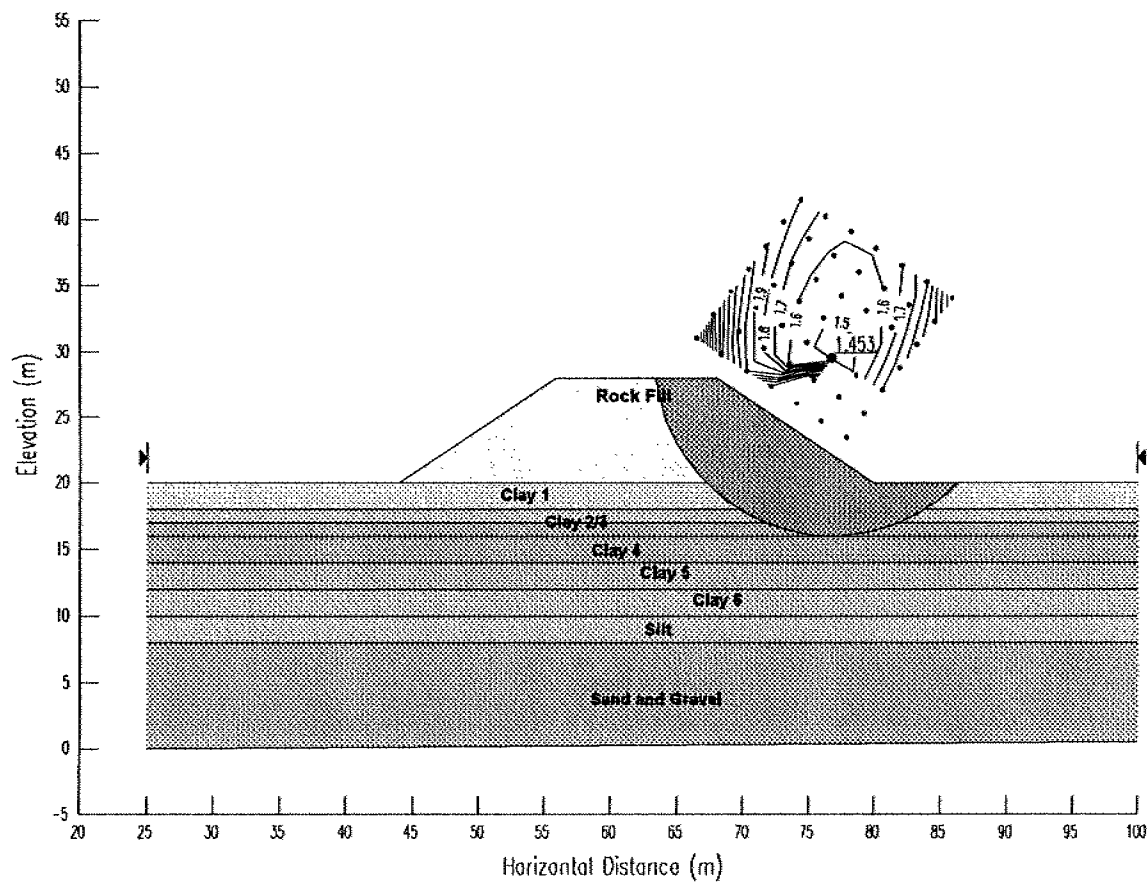
Soil 8 - Clay 7  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 9 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 10 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 11 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 with 5m spaced Wick Drains  
 8 metre embankment height, 1.5:1 overall side slope  
 10 metre clay foundation  
 H08T1053.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 28

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 28

Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

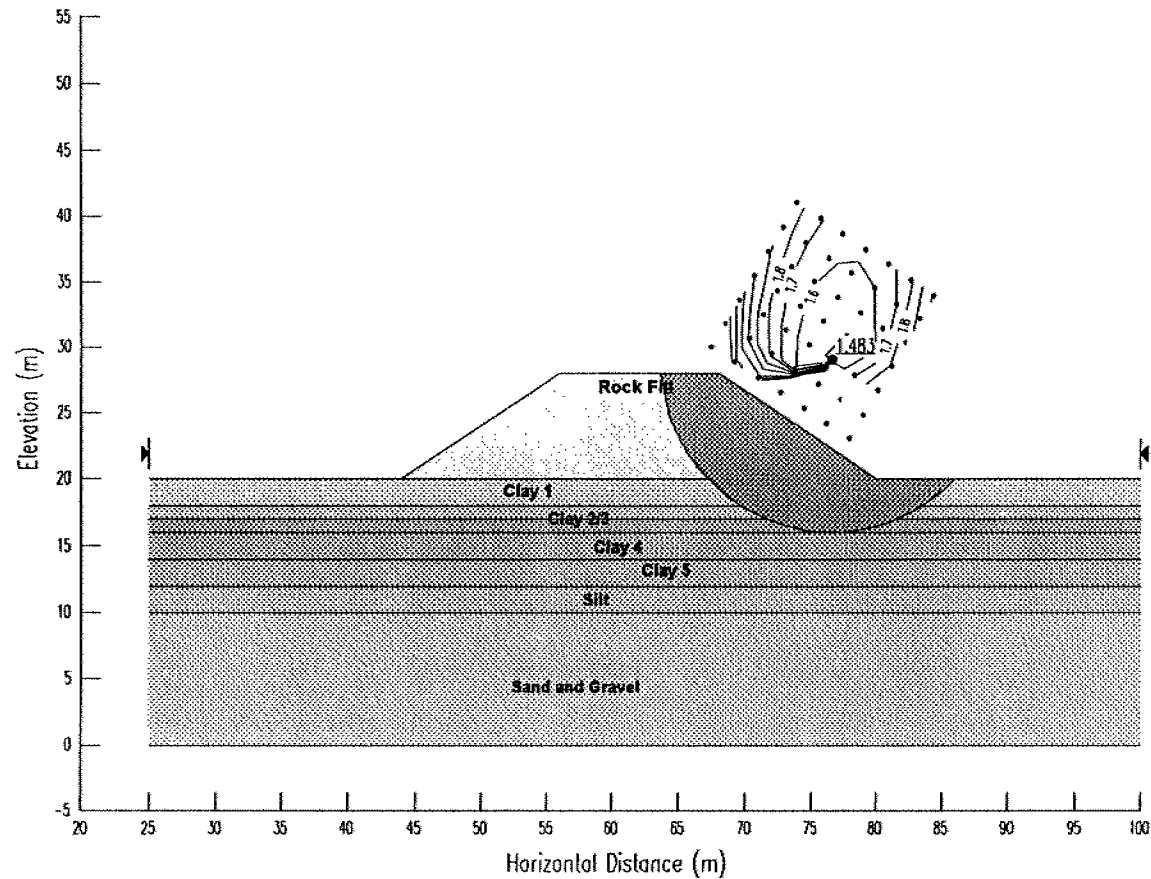
Soil 7 - Clay 6  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 28

Soil 8 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 9 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 10 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 with 5m spaced Wick Drains  
 8 metre embankment height, 1.5:1 overall side slope  
 8 metre clay foundation  
 H08T0853.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

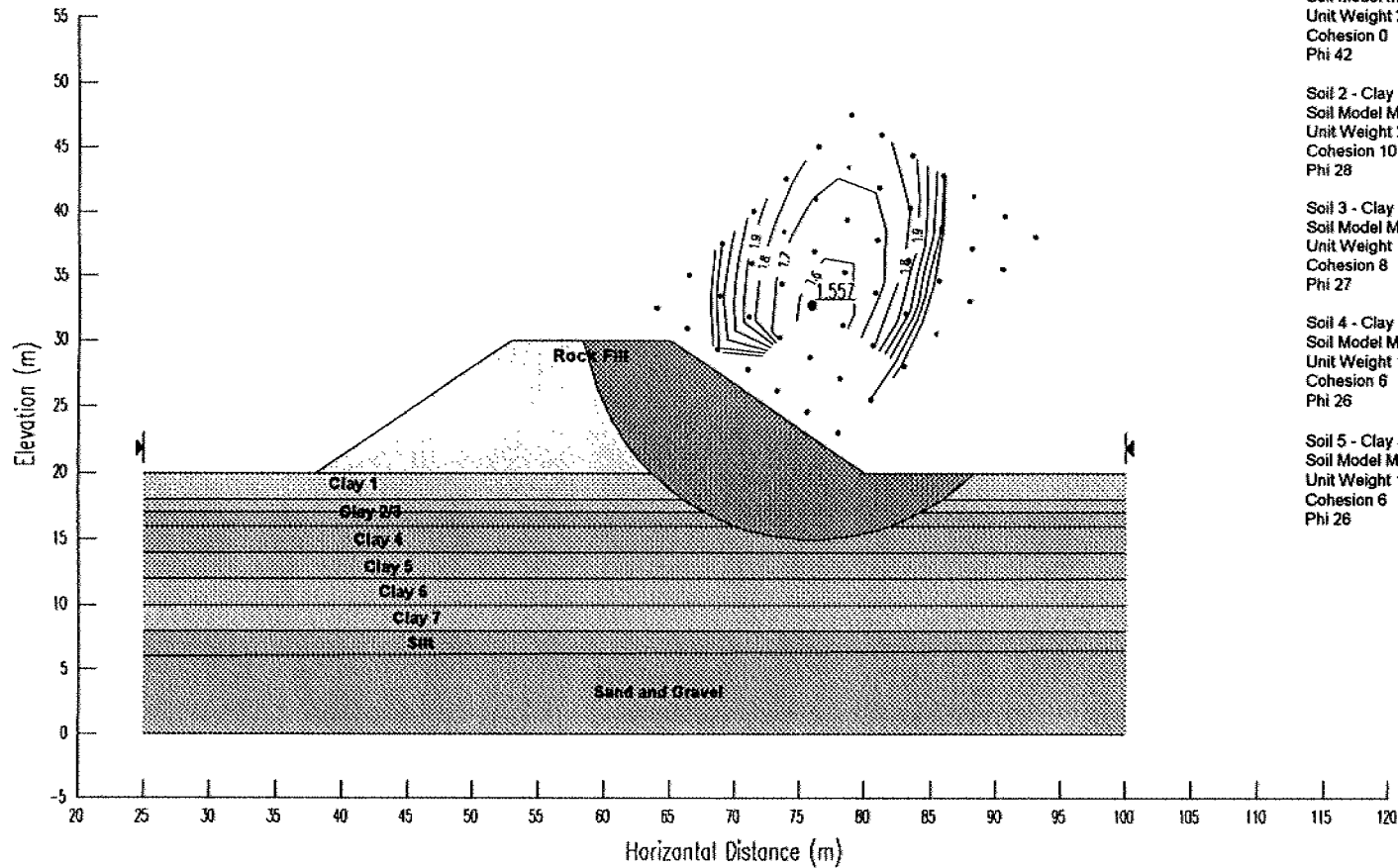
Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 7 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 8 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 9 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179) -  
 Effective Stress Analysis - End of Construction at 3 months  
 with 3m spaced Wick Drains  
 10 metre embankment height, 1.5:1 overall side slope  
 12 metre clay foundation  
 H10T1233.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 7 - Clay 6  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

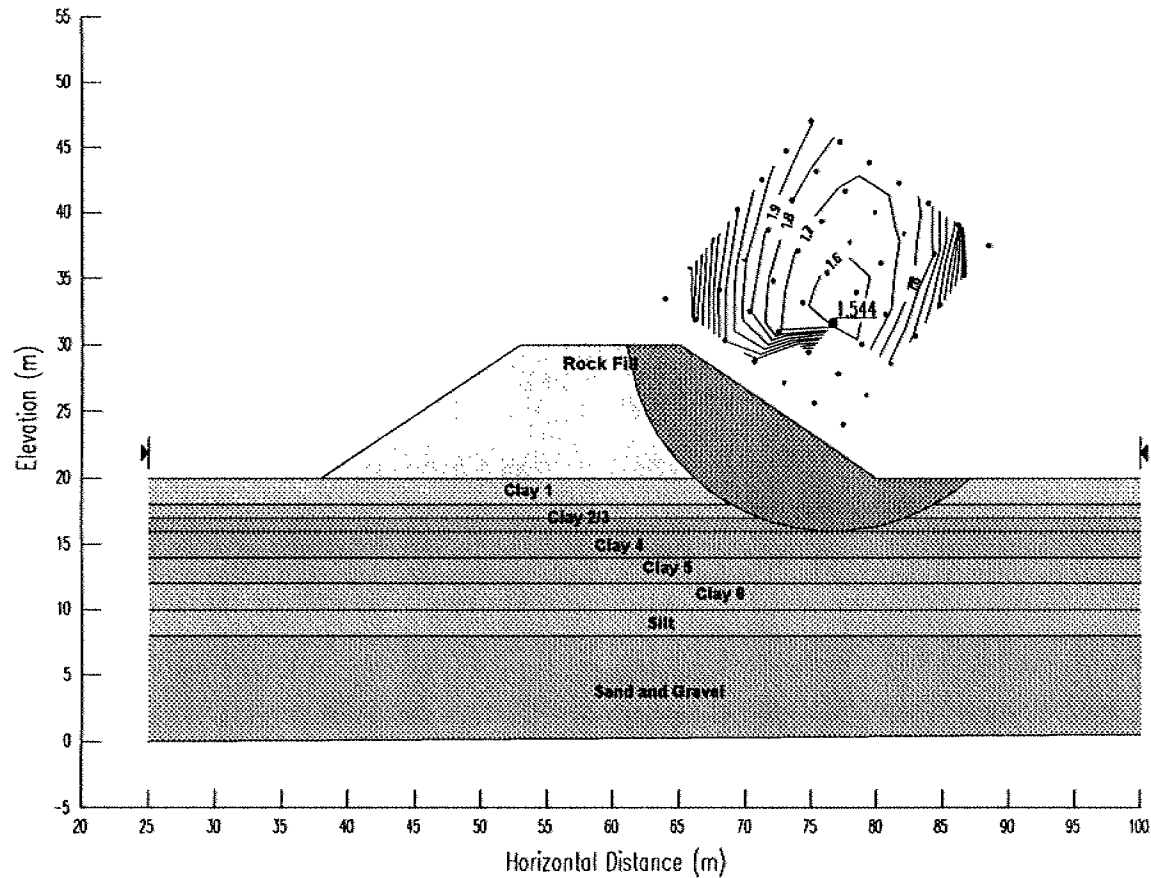
Soil 8 - Clay 7  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 9 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 10 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 11 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 with 3m spaced Wick Drains  
 10 metre embankment height, 1.5:1 overall side slope  
 10 metre clay foundation  
 H10T1033.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

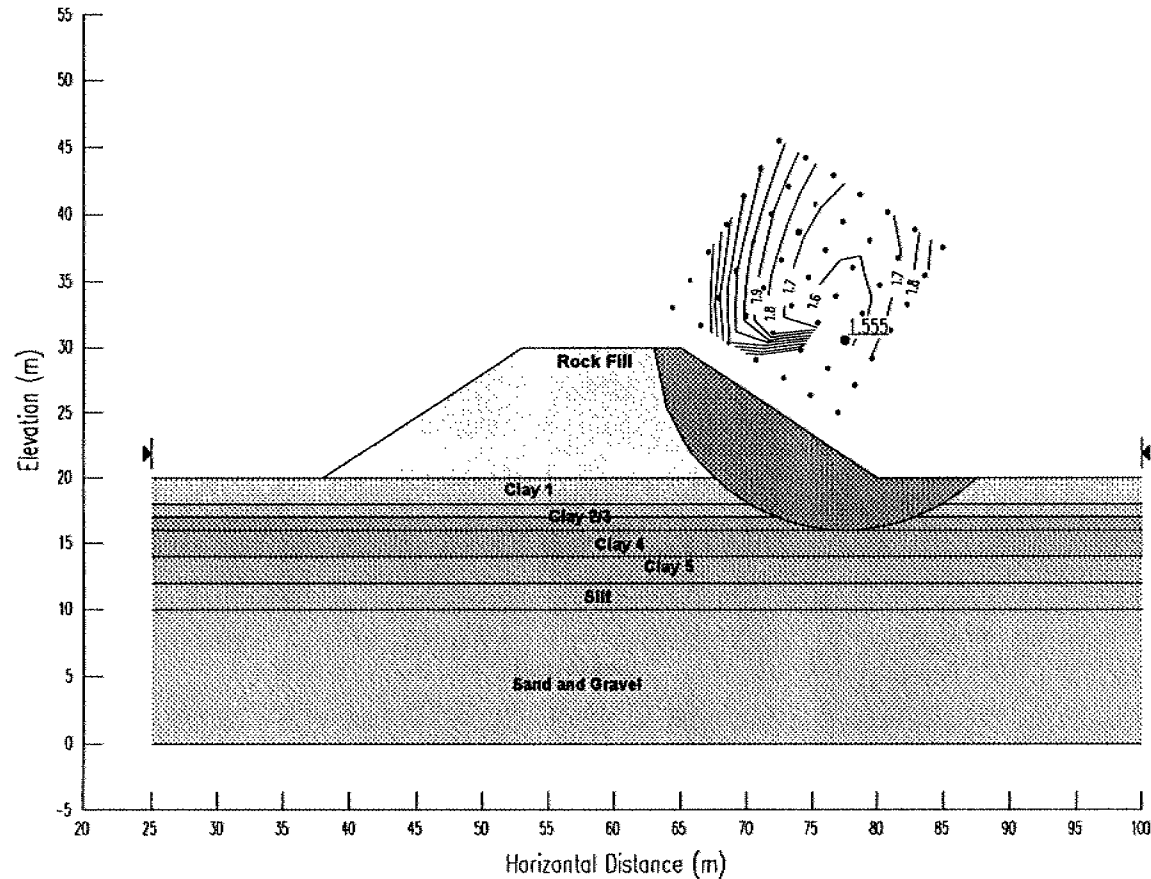
Soil 7 - Clay 6  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 8 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 9 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 10 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 with 3m spaced Wick Drains  
 10 metre embankment height, 1.5:1 overall side slope  
 8 metre clay foundation  
 H10T0833.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

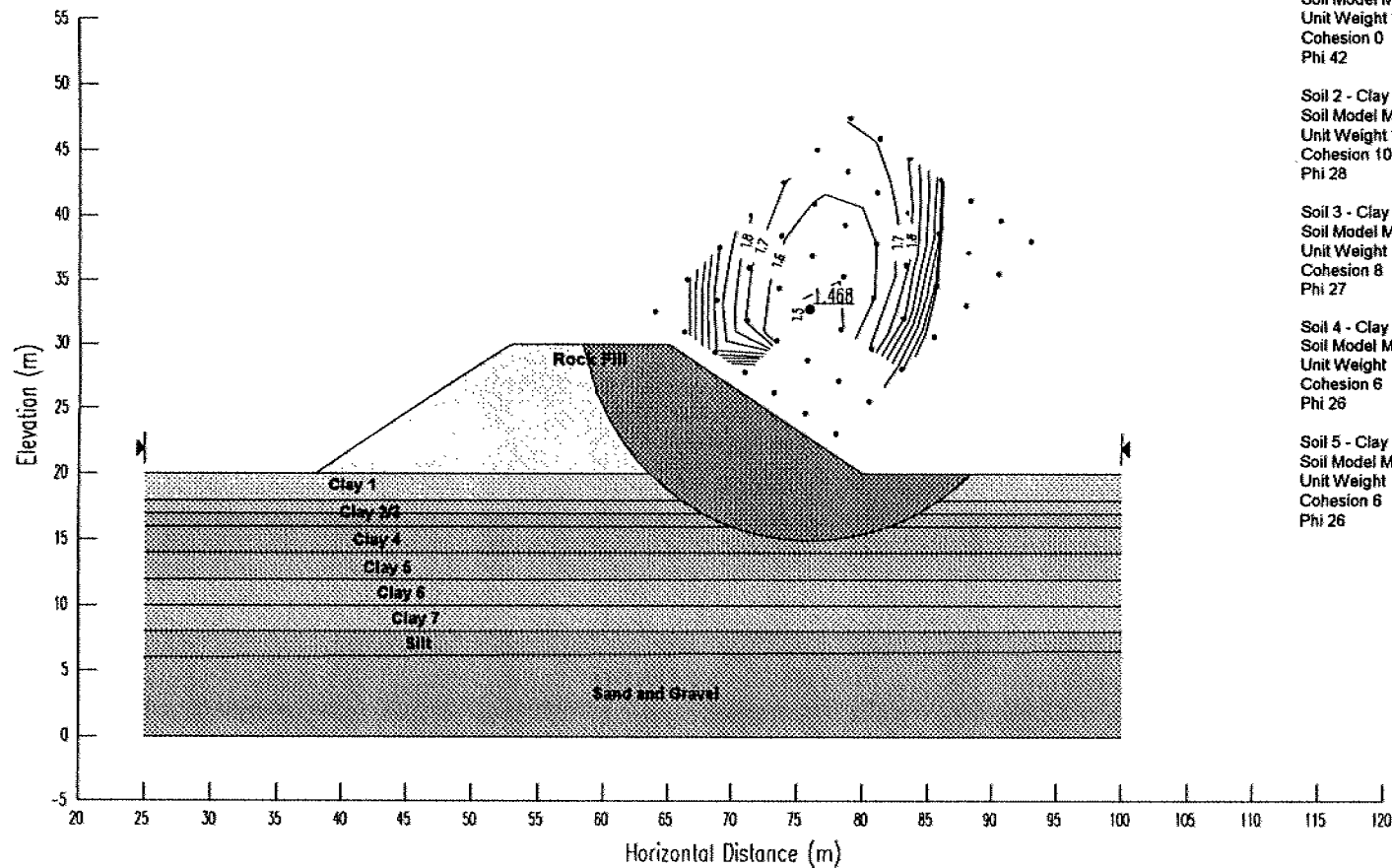
Soil 7 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 8 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 9 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1



Trout Creek - South Interchange Highway Embankments (F98179) -  
 Effective Stress Analysis - End of Construction at 3 months  
 with 4m spaced Wick Drains  
 10 metre embankment height, 1.5:1 overall side slope  
 12 metre clay foundation  
 H10T1243.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 7 - Clay 6  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

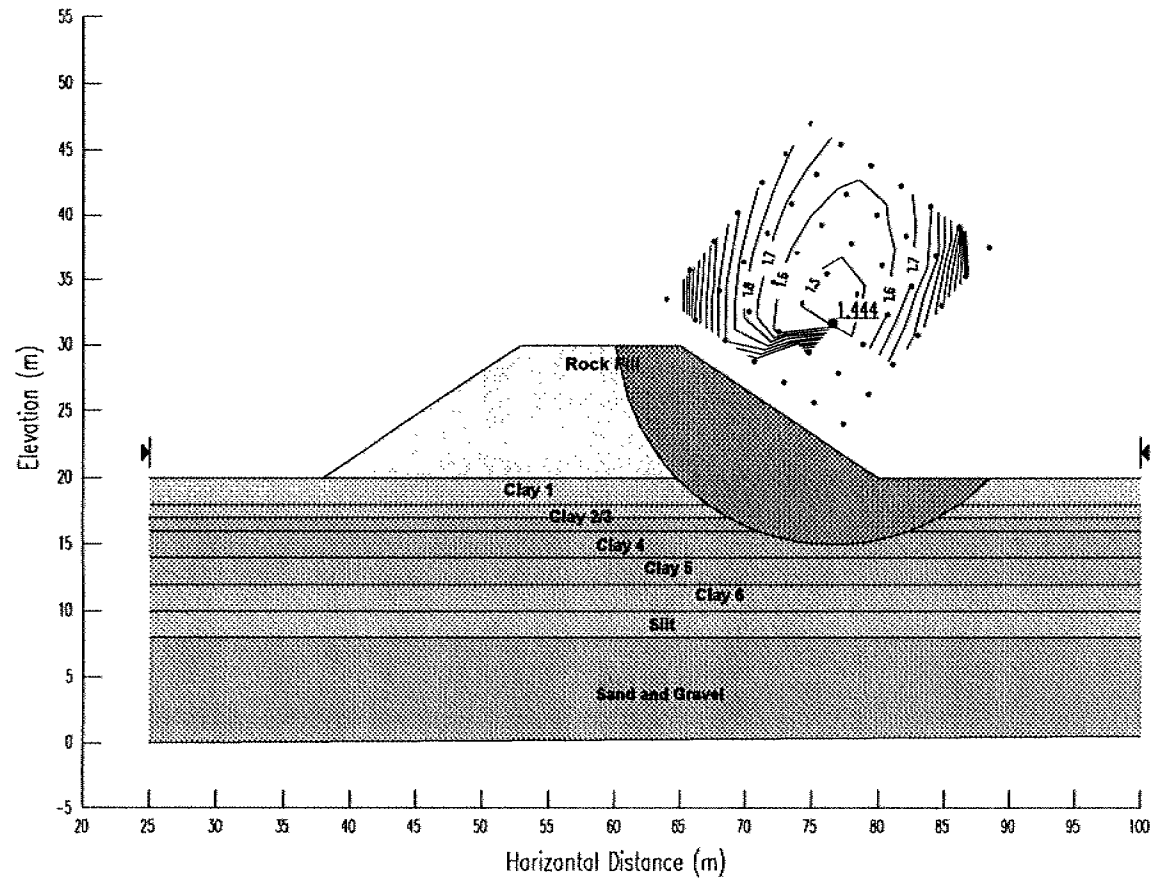
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 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 9 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 10 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 11 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)-  
 Effective Stress Analysis - End of Construction at 3 months  
 with 4m spaced Wick Drains  
 10 metre embankment height, 1.5:1 overall side slope  
 10 metre clay foundation  
 H10T1043.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

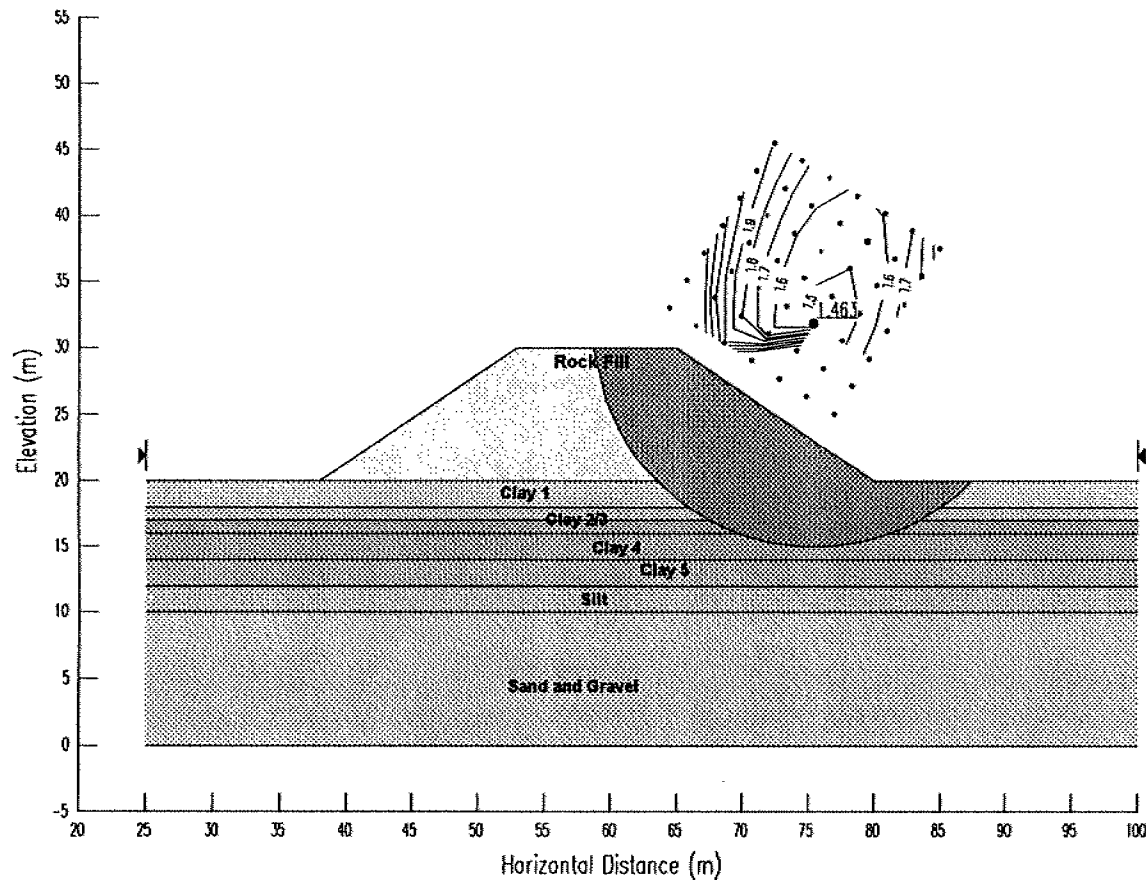
Soil 7 - Clay 6  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 8 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 9 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 10 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 with 4m spaced Wick Drains  
 10 metre embankment height, 1.5:1 overall side slope  
 8 metre clay foundation  
 H10T0843.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 28

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

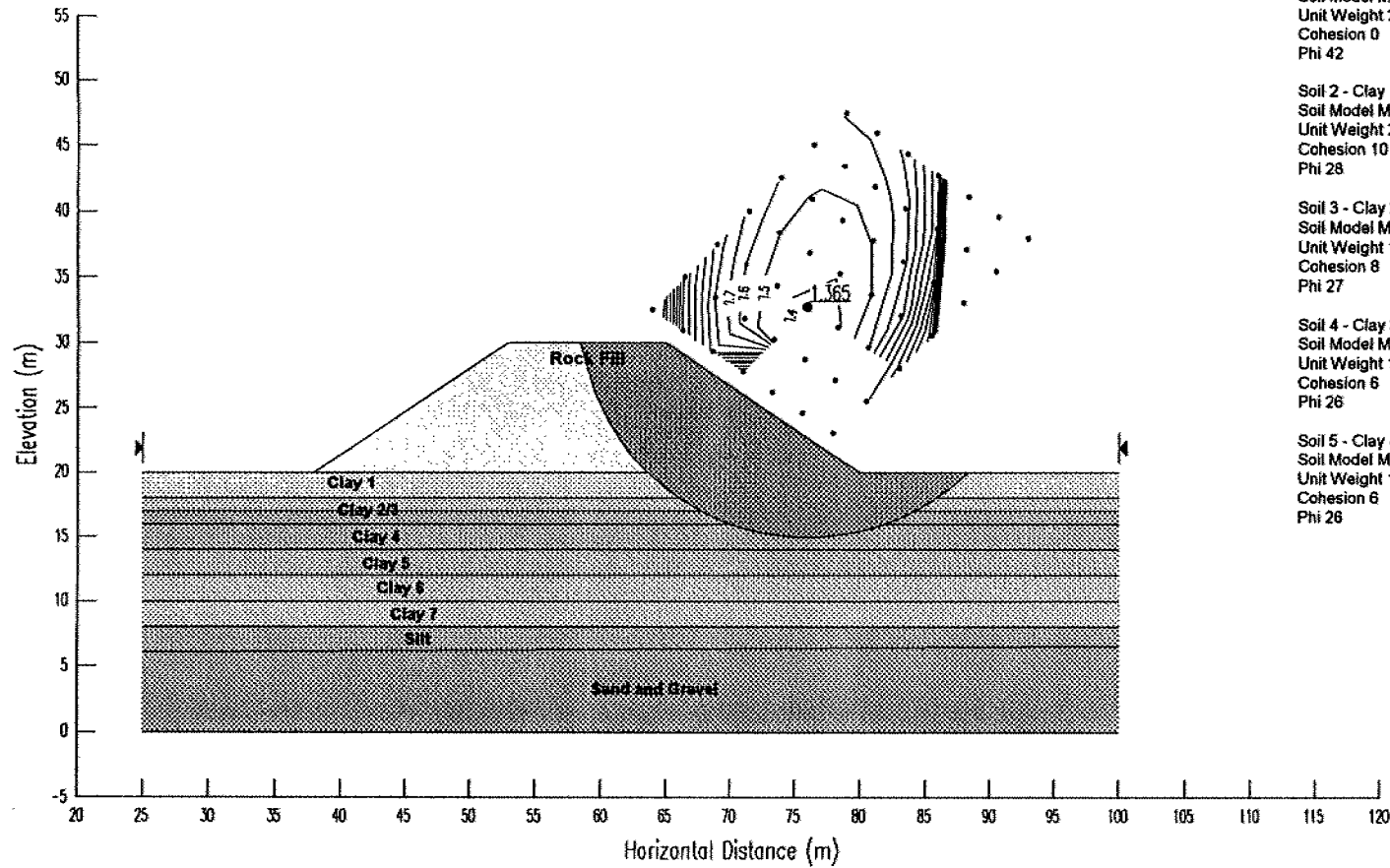
Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 7 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

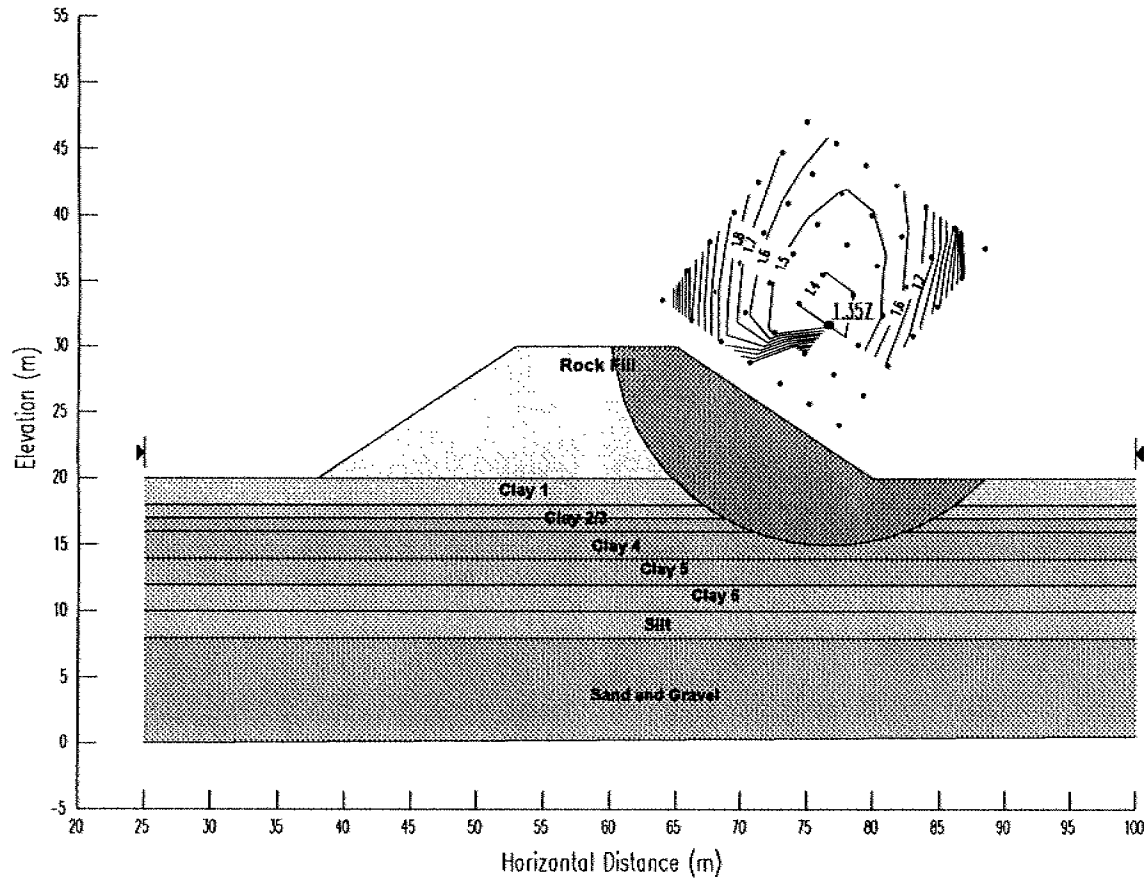
Soil 8 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 9 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 with 5m spaced Wick Drains  
 10 metre embankment height, 1.5:1 overall side slope  
 12 metre clay foundation  
 H10T1253.SLP



Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 with 5m spaced Wick Drains  
 10 metre embankment height, 1.5:1 overall side slope  
 10 metre clay foundation  
 H10T1053.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

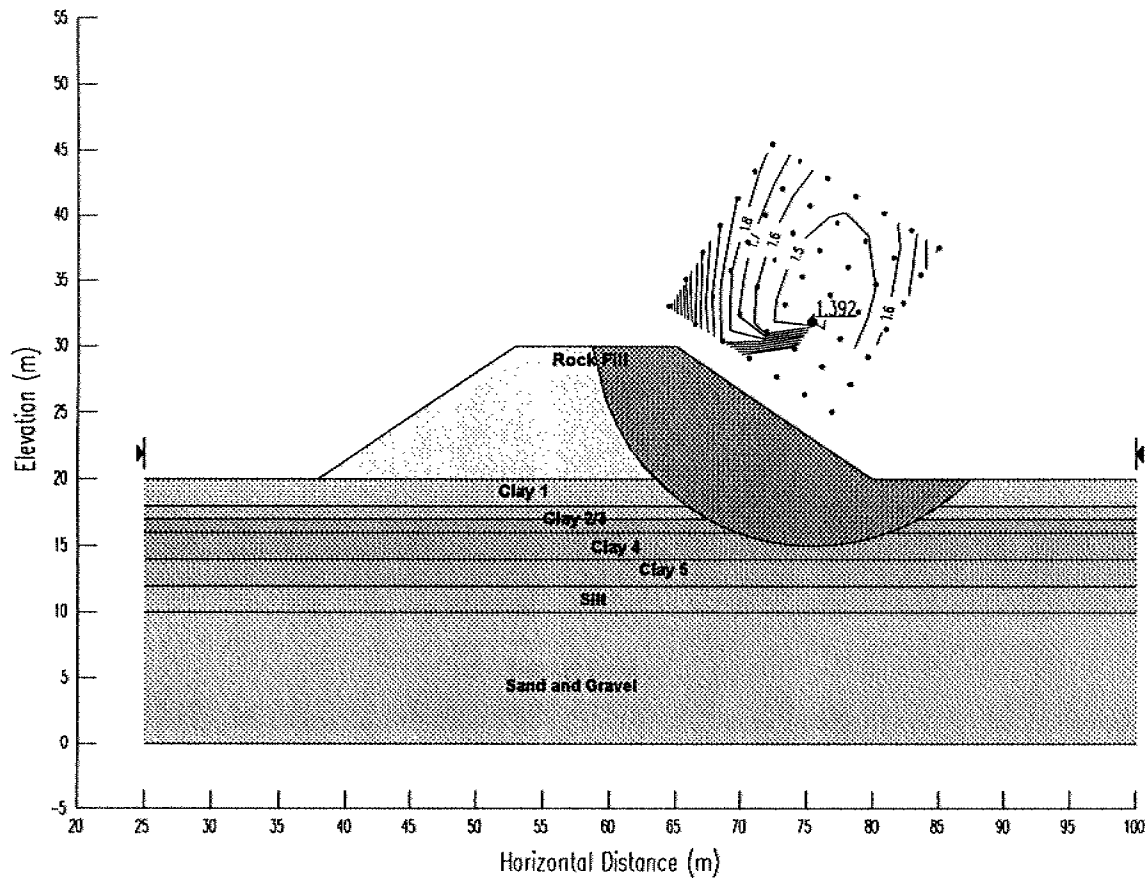
Soil 7 - Clay 6  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 8 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

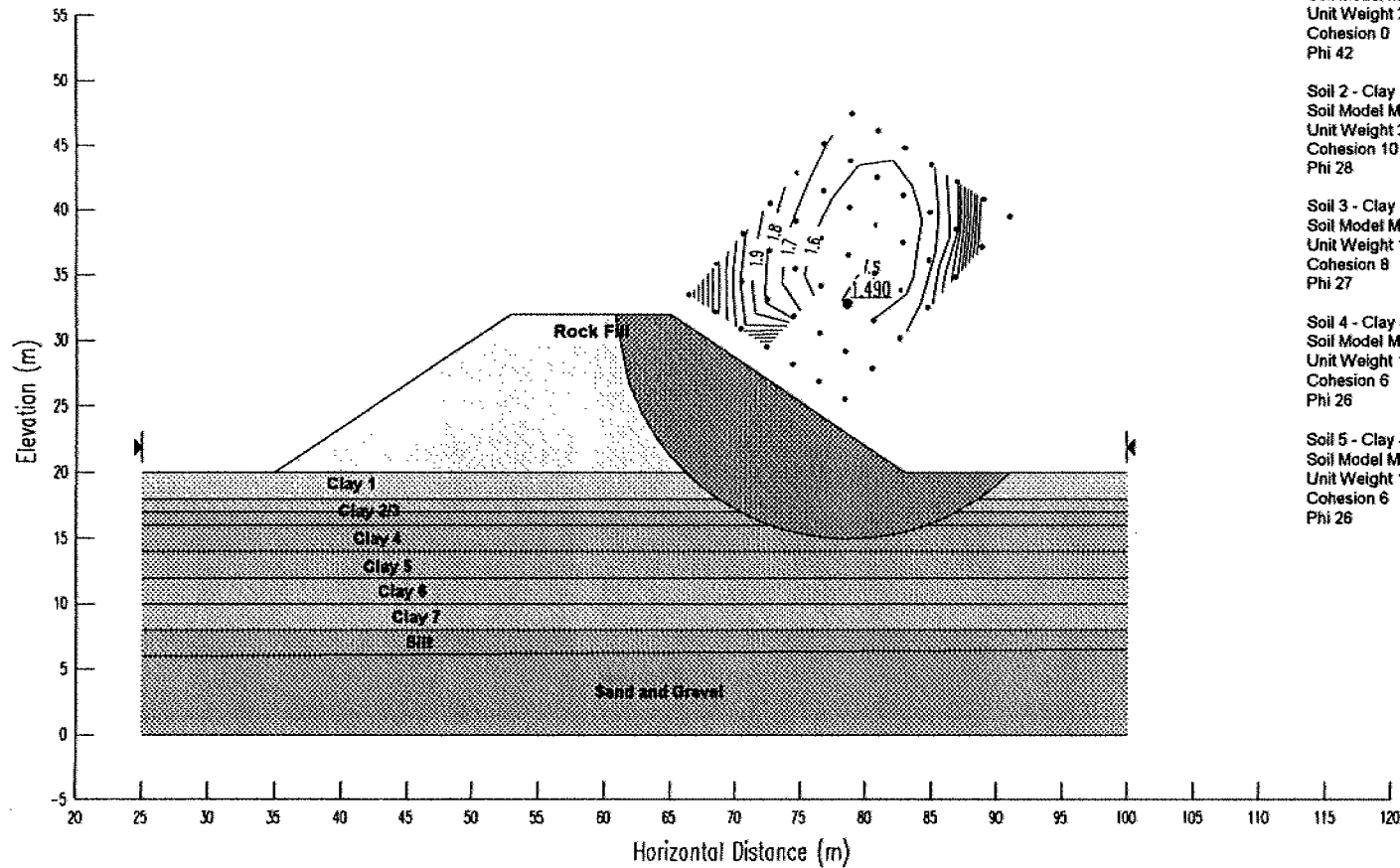
Soil 9 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 10 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 with 5m spaced Wick Drains  
 10 metre embankment height, 1.5:1 overall side slope  
 8 metre clay foundation  
 H10T0853.SLP



Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 with 3m spaced Wick Drains  
 12 metre embankment height, 1.5:1 overall side slope  
 12 metre clay foundation  
 H12T1233.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 7 - Clay 6  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 8 - Clay 7  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 9 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 10 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 11 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 with 3m spaced Wick Drains  
 12 metre embankment height, 1.5:1 overall side slope  
 10 metre clay foundation  
 H12T1033.SLP

Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

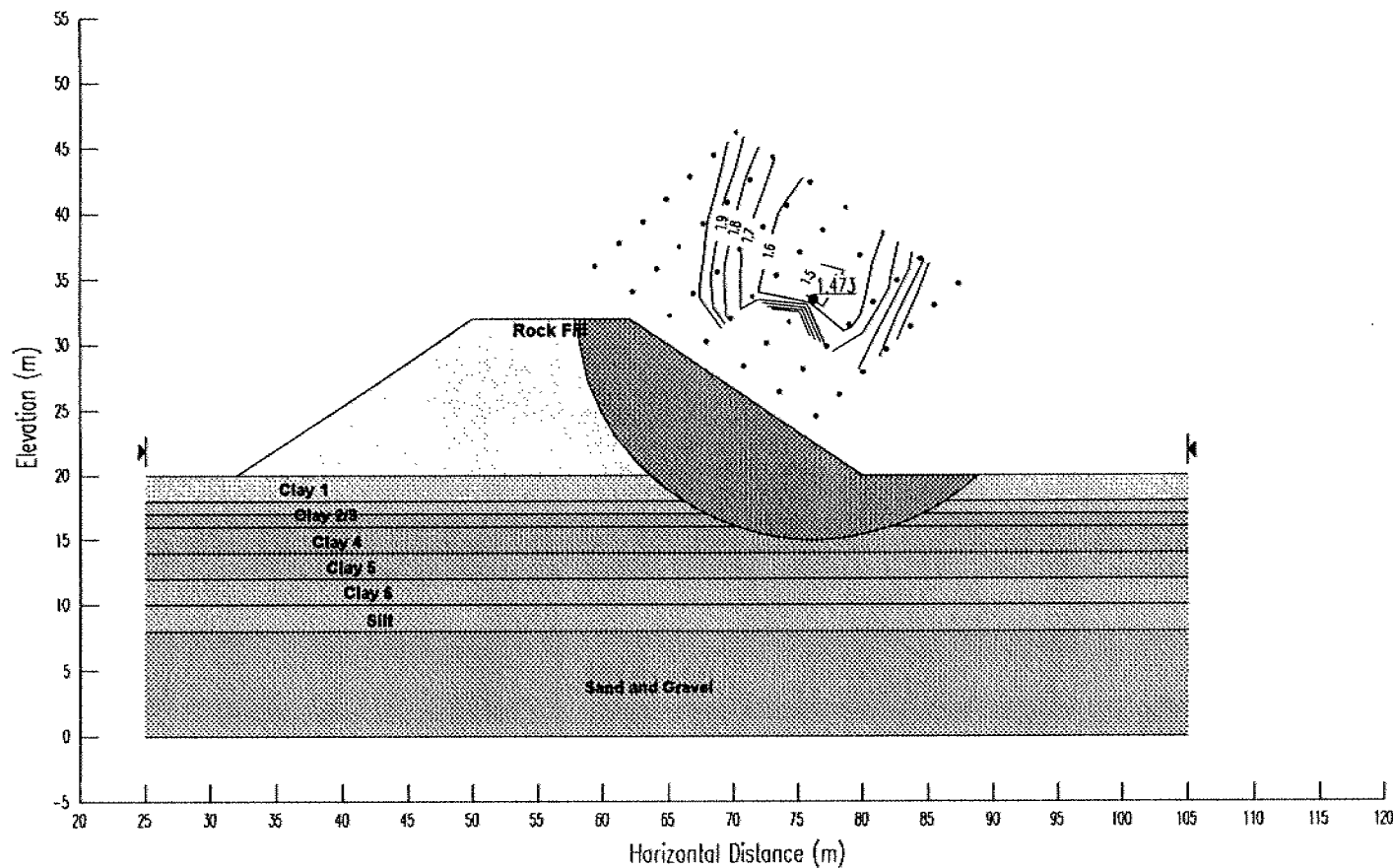
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 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 7 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

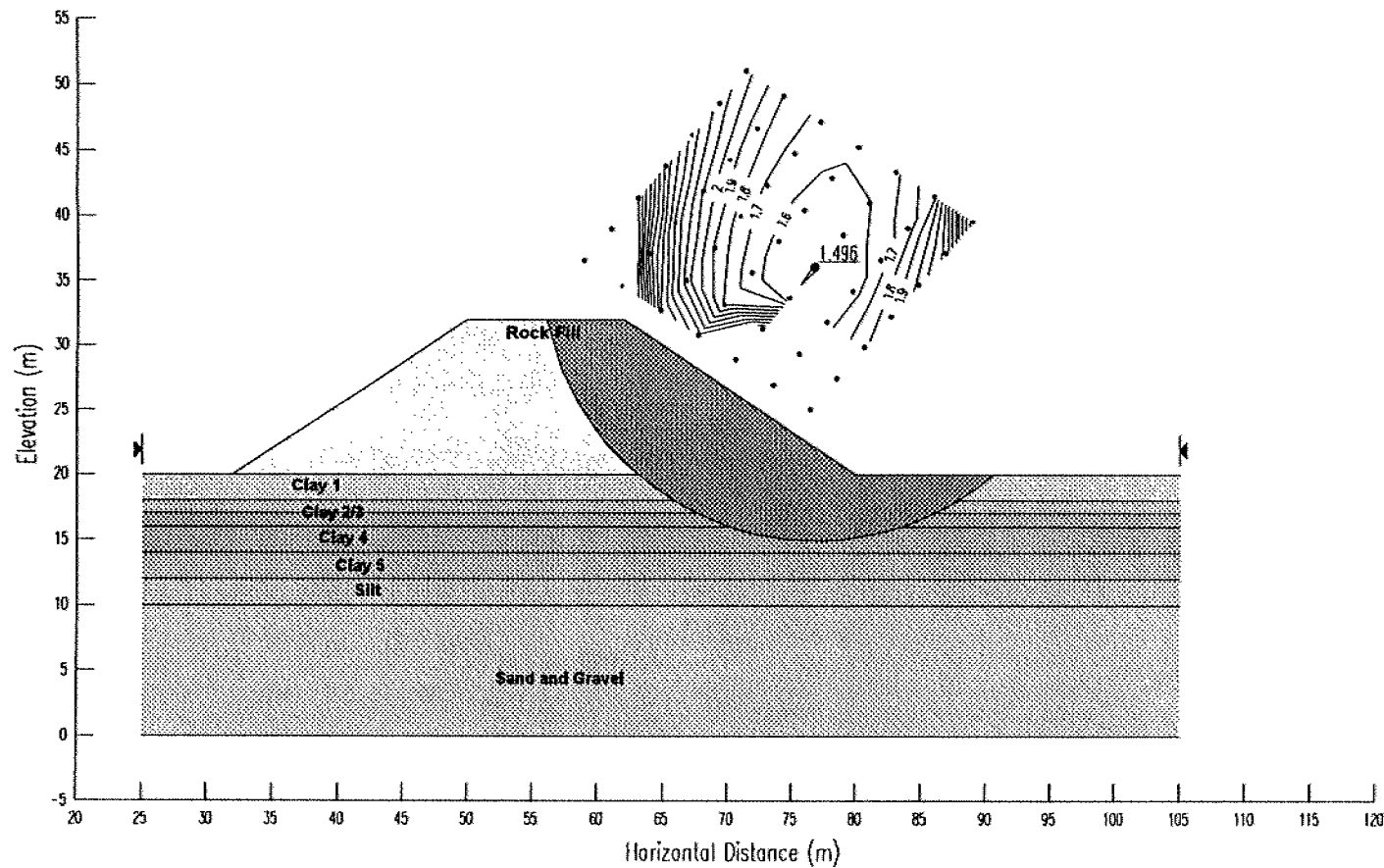
Soil 8 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 9 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1





Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 with 3m spaced Wick Drains  
 12 metre embankment height, 1.5:1 overall side slope  
 8 metre clay foundation  
 H12T0833.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

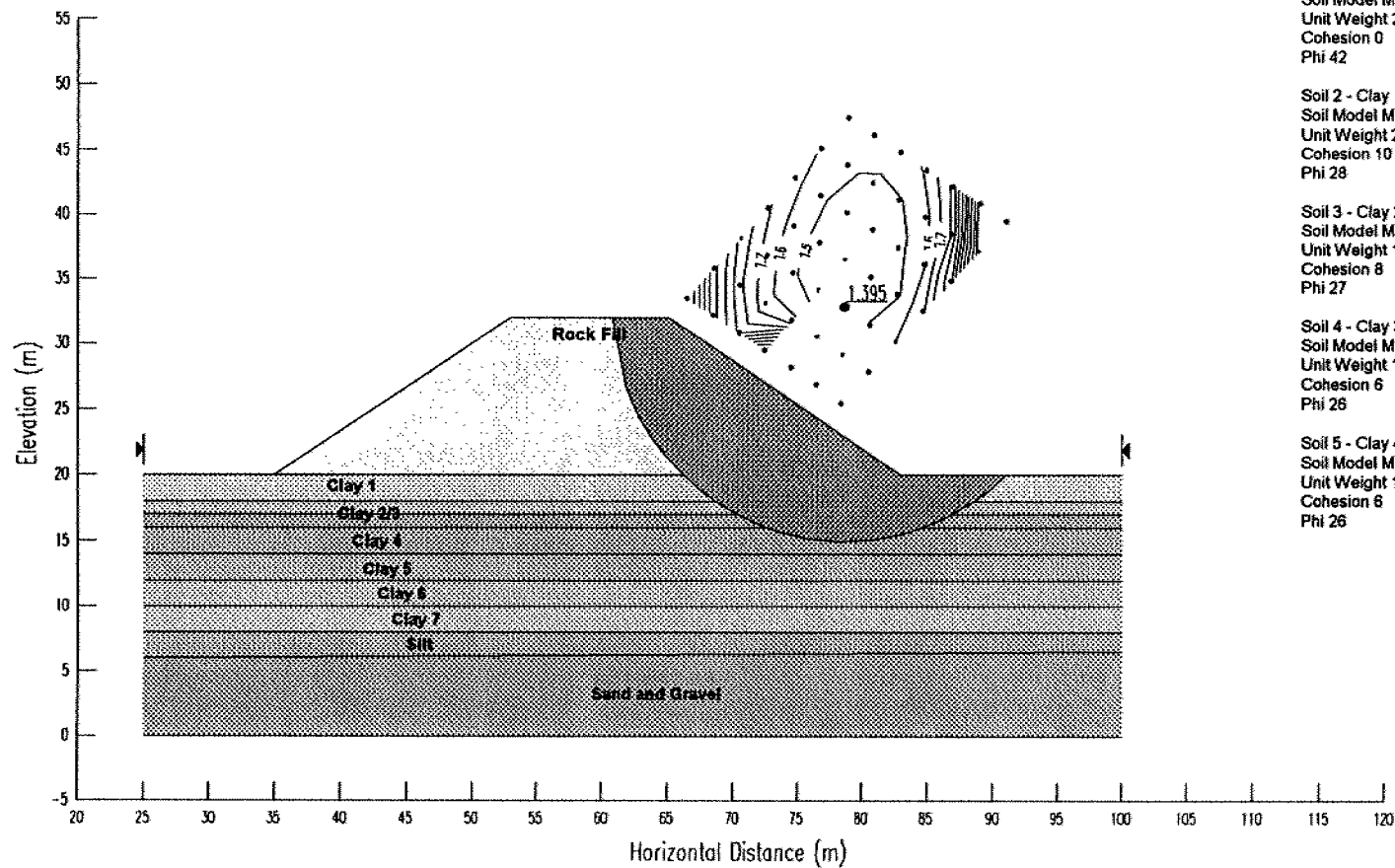
Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 7 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 8 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 9 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 with 4m spaced Wick Drains  
 12 metre embankment height, 1.5:1 overall side slope  
 12 metre clay foundation  
 H12T1243.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 7 - Clay 6  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 8 - Clay 7  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 9 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 10 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 11 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 with 4m spaced Wick Drains  
 12 metre embankment height, 1.5:1 overall side slope  
 10 metre clay foundation  
 H12T1043.SLP

Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

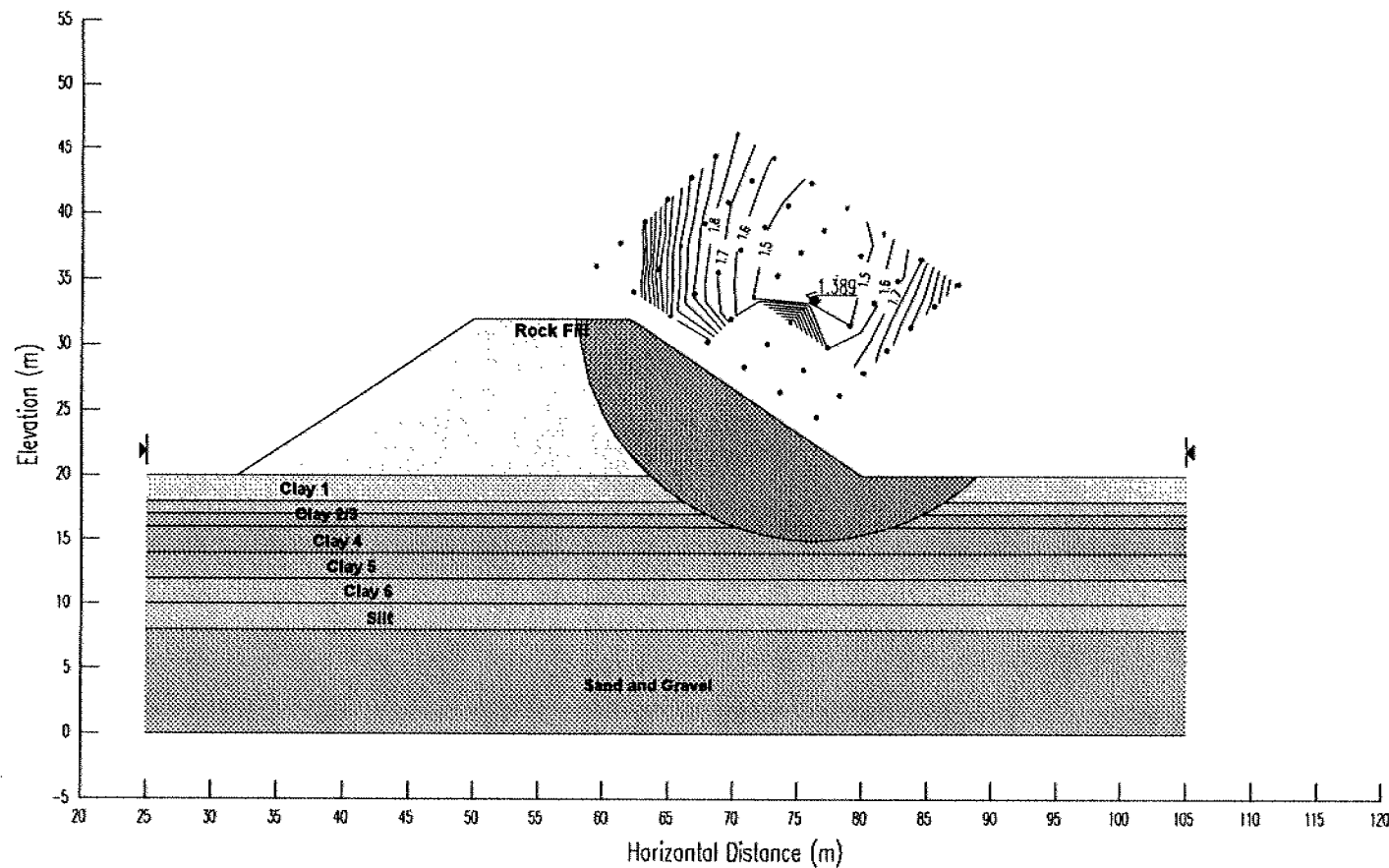
Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

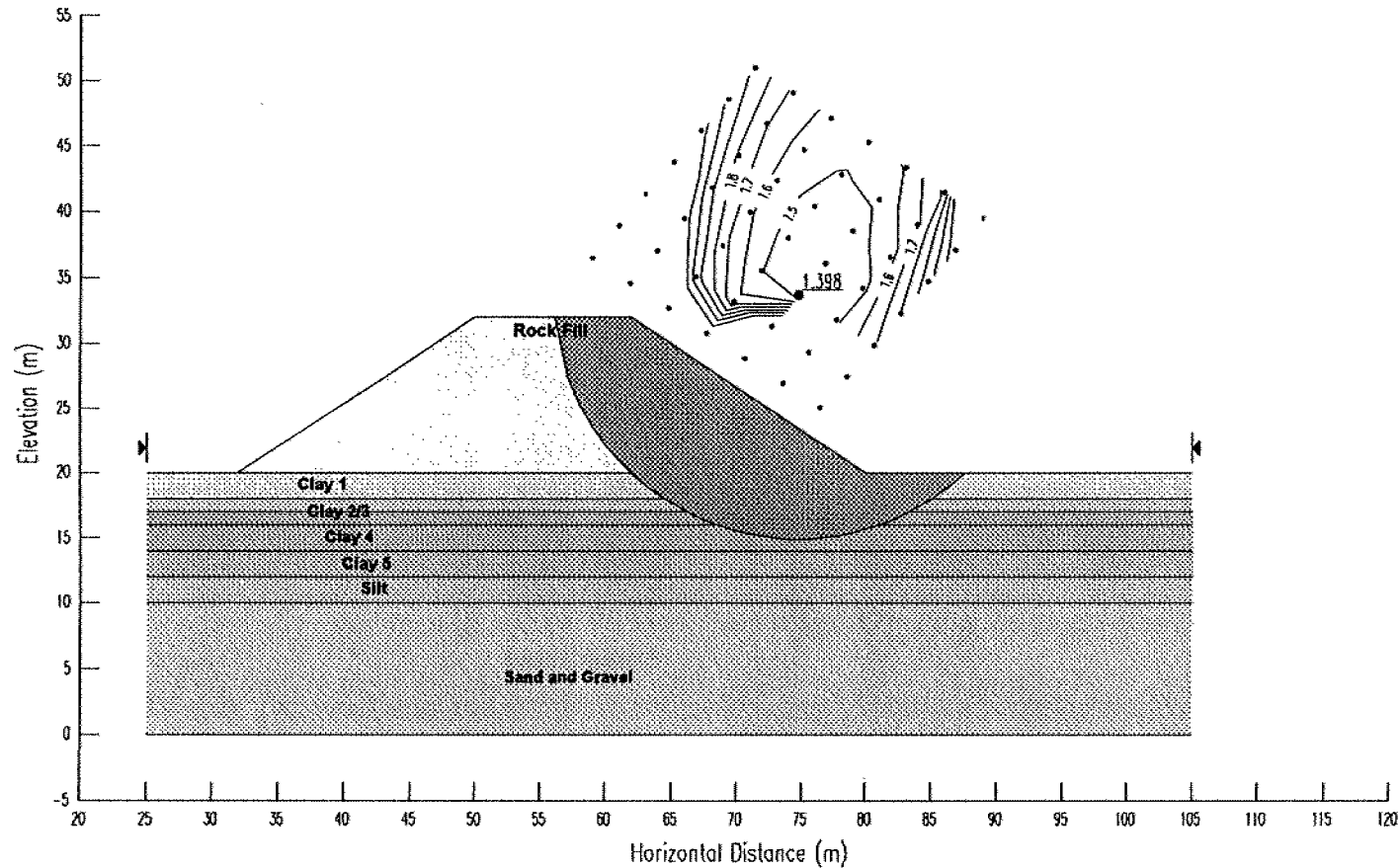
Soil 7 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 8 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 9 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1



Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 with 4m spaced Wick Drains  
 12 metre embankment height, 1.5:1 overall side slope  
 8 metre clay foundation  
 H12T0843.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

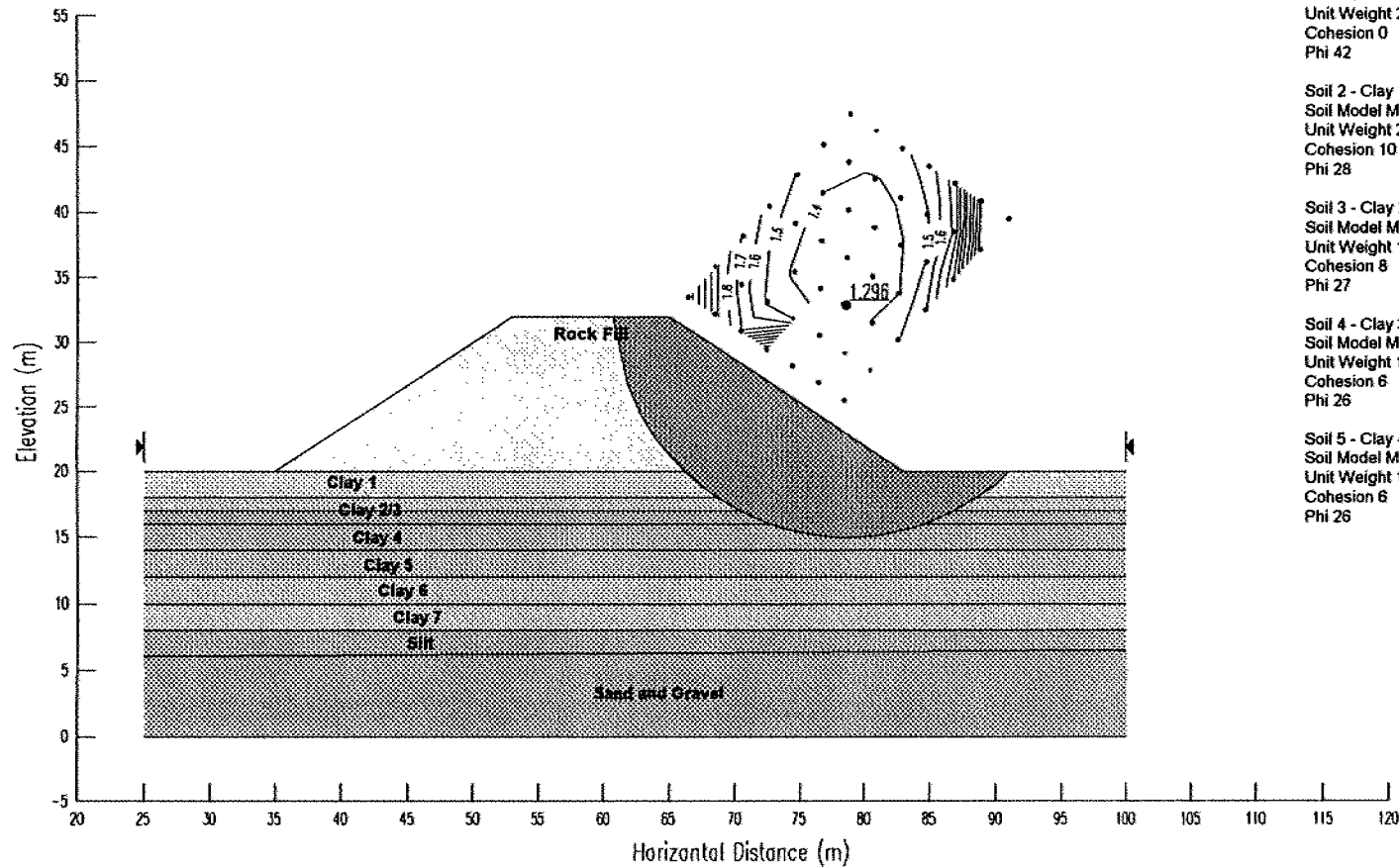
Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 7 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 8 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 9 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 with 5m spaced Wick Drains  
 12 metre embankment height, 1.5:1 overall side slope  
 12 metre clay foundation  
 H12T1253.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 7 - Clay 6  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 8 - Clay 7  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 9 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 10 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 11 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 with 5m spaced Wick Drains  
 12 metre embankment height, 1.5:1 overall side slope  
 10 metre clay foundation  
 H12T1053.SLP

Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

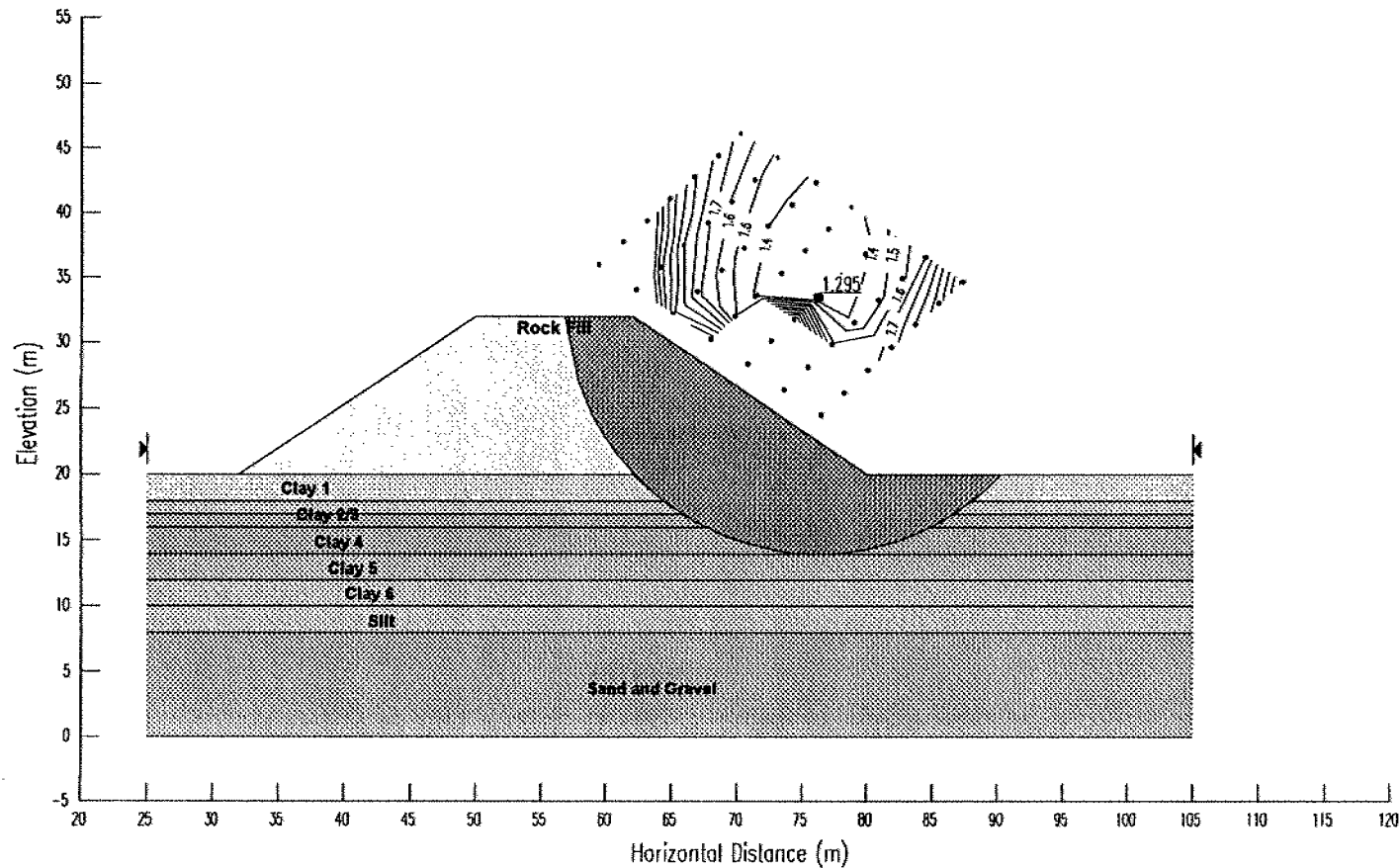
Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

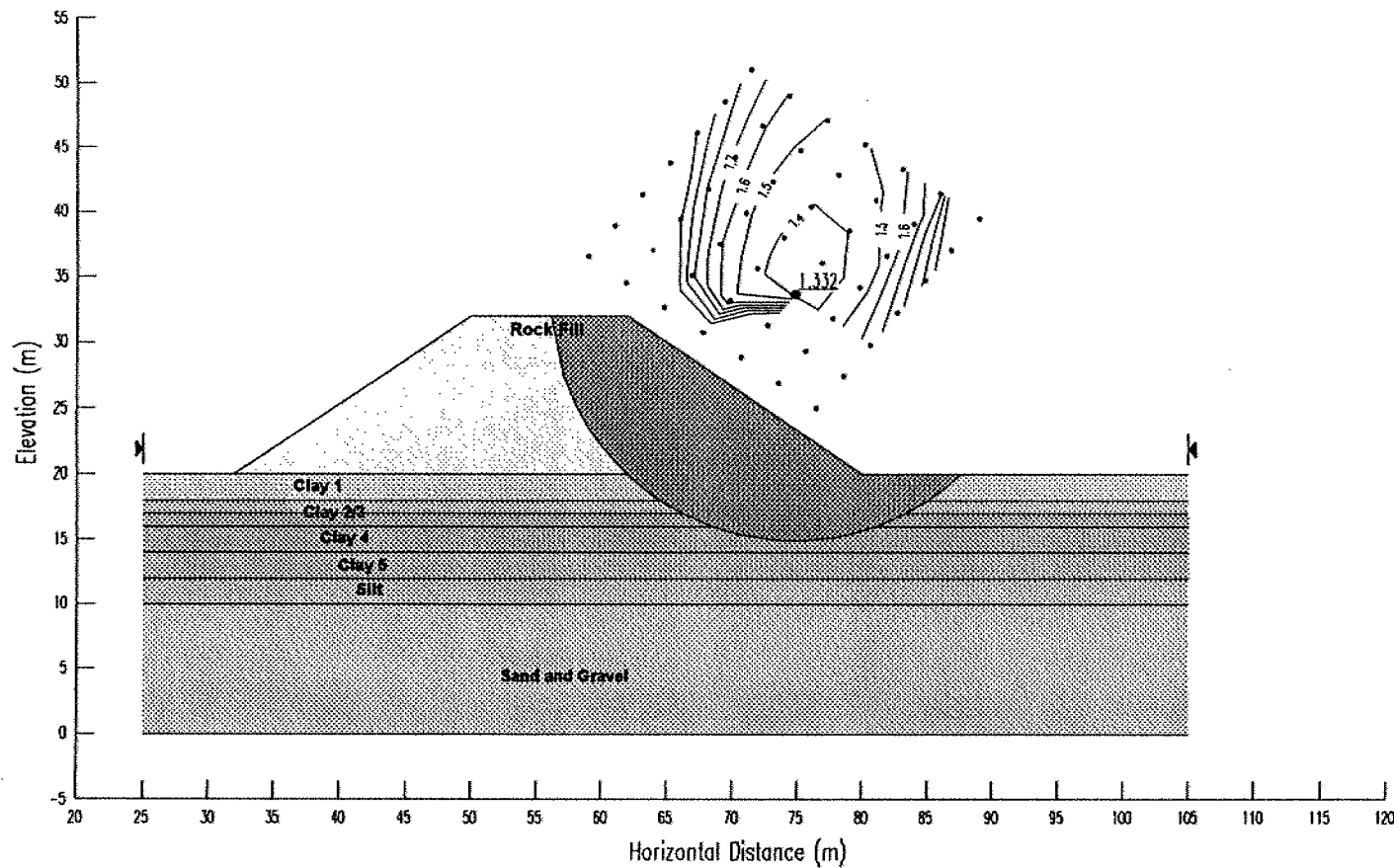
Soil 7 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 8 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 9 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1



Trout Creek - South Interchange Highway Embankments (F98179)  
 Effective Stress Analysis - End of Construction at 3 months  
 with 5m spaced Wick Drains  
 12 metre embankment height, 1.5:1 overall side slope  
 8 metre clay foundation  
 H12T0853.SLP



Soil 1 - Rock Fill  
 Soil Model Mohr-Coulomb  
 Unit Weight 20  
 Cohesion 0  
 Phi 42

Soil 2 - Clay 1  
 Soil Model Mohr-Coulomb  
 Unit Weight 20.5  
 Cohesion 10  
 Phi 28

Soil 3 - Clay 2  
 Soil Model Mohr-Coulomb  
 Unit Weight 18.3  
 Cohesion 8  
 Phi 27

Soil 4 - Clay 3  
 Soil Model Mohr-Coulomb  
 Unit Weight 18  
 Cohesion 6  
 Phi 26

Soil 5 - Clay 4  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.3  
 Cohesion 6  
 Phi 26

Soil 6 - Clay 5  
 Soil Model Mohr-Coulomb  
 Unit Weight 17.8  
 Cohesion 6  
 Phi 26

Soil 7 - Silt  
 Soil Model Mohr-Coulomb  
 Unit Weight 19.5  
 Cohesion 0  
 Phi 31

Soil 8 - Sand/Gravel  
 Soil Model Mohr-Coulomb  
 Unit Weight 21.5  
 Cohesion 0  
 Phi 35

Soil 9 - Bedrock  
 Soil Model Bedrock  
 Unit Weight -1

# MEMORANDUM



To: E. Gallant  
Senior Project Manager  
Planning and Design, Northern Region

May 5, 1999

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

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

Re: Wick Drain Design - South Interchange Embankments  
Hwy 11 Four Laning  
Trout Creek Bypass  
WP 774-93-00  
District 54, Sudbury

The draft report for the Wick Drain Design and Monitoring Program for the South Interchange Embankments has been reviewed by our office. Our review comments are contained in this memorandum.

We have scheduled a meeting with Thurber Engineering Ltd. to discuss the report. Following this meeting, Thurber Engineering Ltd will finalize the report and finalize the report for the North Interchange Embankments.

## 1. Introduction

Reference should be made to the "Terms of Reference".

Change number identification from 3 to 1.

## 8. Engineering Analysis

### 8.1 General

#### *Areas & Geometries*

A total of five(5) distinct areas of analysis have been identified. An illustration of the five(5) areas in plan view would assist in the identification of these areas

#### *Soil Properties*

An explanation of the decision to conduct settlement calculations using "most likely values" and "reduced values" is requested. Can the  $p_c$  value be obtained from the one dimensional consolidation tests conducted by Trow?



In the selection of the  $c_h$  values interpreted from the CPTU's, the soil is referred to as "normally consolidated". Is the  $p_c$  value equivalent to the  $p_o$  value?

## 8.2 Stability Analyses

### *Geometry*

It is indicated that "the width of the embankment at the top of the surcharge will be the same as the final design width" and that "the embankment side slopes above the berm will be temporarily steeper than 1.25H:1V". What is the slope geometry of the surcharge?

### *Surcharge*

The range of surcharge is 1 to 1.5 m above the embankment design height. Can we simplify the matter of surcharge and use a constant value?

### *Results*

- It should be confirmed that within the staged construction, the embankment can be constructed in a continuous and constant rate if there is no requirement for dissipation of excess pore pressure. Should there be a requirement for dissipation of excess pore pressure (EPP), there should be a recommendation that this requirement be explicitly stated in the contract documents.
- Location: East Approach to Bridge - CPTUS3  
  
The reason for the 8 m berm width geometry and the associated construction staging is not understood.
- The maximum magnitude of Excess Pore Pressure should be identified in the text. In addition, Excess Pore Pressure versus Time curves would assist in visualizing the time rate of excess pore pressure dissipation.

## 8.3 Settlement Analysis

### 8.3.2 One Dimensional Consolidation - No Wick Drains

It should be confirmed that no wick drains are required at the:

1. EW-N Ramp, S-EW Ramp and Hwy 11 south of the interchange location
2. Hwy 11 north of the Interchange and EW-S Ramp

### 8.3.3 Settlements due to Primary Consolidation - With Wick Drains

pg.18 - recommendations are given to stage the construction to allow a certain degree of consolidation. It is recommended that the basis of the waiting period is the degree of consolidation(monitored pore pressure dissipation) and not the predicted time period given.

#### 8.3.4 Settlements due to Secondary Consolidation

The magnitude of long term secondary consolidation settlements in our opinion are not excessive. We recommend that these settlements be managed by conventional maintenance procedures.

## 9 Embankment Design Recommendations

### 9.1 *Embankment Geometry and Construction Schedule*

1. An illustration of the general layout of the wick drains is recommended.
2. An eight(8) metre berm width has been chosen. An explanation for the **eight(8)** metre requirement is requested.

### 9.2 *Site Preparation*

A dewatering NSSP should be included in the Contract Documents alerting the Contractor.

A NSSP for the supply, placement and compaction of the Granular filter blanket should be included in the Contract Documents.

### 9.3 *Wick Drain Specification*

In our opinion, the wick drain specification is incomplete. The specification should include a component addressing:

1. Equipment
2. Installation
3. Payment

A wick drain NSSP used on a previous MTO project prepared by our office will be made available. The NSSP should be based on the MTO specification with site specific adjustments.

### 9.4 Instrument Monitoring Program

#### 9.4.1 Instrumentation

It is recommended that a layout of the instrumentation monitoring program be provided.

Regarding the instrumentation, do the settlement pins add value to the monitoring program? Is there a need for the shallow standpipes?

Slope indicators have been recommended to monitor horizontal displacements. A prediction of lateral displacement should be provided.

Specifications for the supply and installation of the instrumentation are required for inclusion in the contract documents.

#### 9.5 Trial Embankment

Our office endorses the trial embankment recommended by the Consultant.

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: E. Gallant, P. Eng.  
Senior Project Engineer  
Planning and Design, Northern Region

January 17, 1999

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

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

Re: Foundation Investigation Report Review  
South Interchange Bridge Structure  
Hwy 11/Boundary Rd  
Trout Creek By-Pass  
WP 774-93-00  
District 54, Sudbury

We have reviewed the draft report prepared by Trow Consulting Engineers Ltd for the proposed bridge structure and the associated approach embankments at the South Interchange. 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. 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..

Our review comments are presented under two separate categories:

- I. Engineering Discussion and Recommendations
- II. Foundation Investigation

## **I Engineering Discussion and Recommendations**

### **2.1 Introduction**

Disclosure of the span lengths and a description of the extent of the approach embankment work should be included.

### **2.2 Foundations**

In the second paragraph, "downdrag" is generally the term used to identify the phenomenon rather than "drag".

### 2.2.1 Pile Capacity and Length

There exists some inconsistency in the design pile capacities tabulated in Table 2-1. A reduction in the pile capacity has only been applied to the HP 310 x 79 piles. There is no reduction in pile capacity tabulated in Table 2-1 for the HP 310 x 110 and HP 310 x 132 piles.

An explanation for the difference in the downdrag loads between the west abutment and the east abutment should be provided. In addition, a 46% reduction in magnitude is considered to be large.

It should be clarified that the capacities in Table 2-1 are applicable to vertical piles and pile capacities shall be reduced for inclined piles. Recommendations for reductions for inclined piles shall be included in the report.

An explanation shall be included that describes the inapplicability of the axial capacity at the Serviceability Limit State.

Table 2-2 shall be revised such that the title "Estimated Pile Tip Elevation" is included.

There exists some redundancy in the final two paragraphs of this section. Perhaps one paragraph could be deleted.

### 2.2.2 Construction

The title is not representative nor specific to pile installation. It is recommended that the title be renamed to "Pile Installation".

### 2.3 Backfill

The report does not adequately address the application of active pressure or at rest pressure depending on the flexibility/rigidity of the structure.

Recommendations for weep holes and/or drainage subdrains should be included in the recommendations.

The report does not address lateral earth pressures due to compaction in accordance with Section 6-7 of the OHBDC..

### 2.4 Excavations

Recommendations for dewatering schemes should be included in the report.

### 2.5 Bridge Approach Fills and Embankments

In general, the embankment stability and settlement calculations have been carried out thoroughly and adequately address the requirements for the project. The results have also been presented effectively both in graphical and tabular form. However, it is recommended that the report identify the options regarding the embankment design and a discussion regarding the technical feasibility and cost effectiveness of each option describing the advantages and disadvantages should be included.

For example, the following subtitles could be considered:

1. Embankments on Native Soil
2. Embankments Supported on Wick Drain Foundation
3. Lightweight Fill Embankments
  - expanded polystyrene blocks
  - expanded pelletized blast furnace slag
4. Preload & Surcharge

#### 2.5.1 Embankment Stability

In the first sentence, it is stated that "*Highway embankments are typically constructed using rockfill as the principal component*". Highway embankments can be constructed using rockfill or earthfill. Perhaps it should be stated that embankments in the area are typically constructed using rockfill.

It is expected that the results of the stability analysis employing the simplified 1.5H:1V slope geometry are not significantly different than stability analyses based on employing a midheight 2 metre berm geometry. However, stability analyses conducted by the MTO are conventionally based on employing the actual geometry.

Table 2-4 that tabulates the Geotechnical Parameters for Slope Stability Analyses should separate the upper silt and the clay stratum. A cohesion intercept ( $c'$ ) of 6-10 kPa is considered high for these materials.

An identification of the thickness of each sublayer would assist the interpretation of the results of the embankment stability analyses.

In the results of the total stress analyses included in the Appendix, it should be confirmed that  $C_v$  represents the undrained shear strength of the soil. It is assumed that perhaps the  $C_v$  should be  $C_u$ .

##### 2.5.1.1 Total Stress Analysis

The report makes reference to a safety factor against foundation failure greater than the "*MTO accepted values of 1.3*". Does the Consultant employ a safety factor different than the safety factor of 1.3? The Consultant should state whether or not embankments will be stable.

##### 2.5.1.2 Total Stress Analysis with Stabilizing Berms

The analyses is based on stabilizing berms placed at one-third the embankment height. Conventionally, the MTO have also conducted analyses using mid-height berms.

### 2.5.5 Rockfill and Rockfill Settlement

The second paragraph refers to the installation of piles following the placement of rockfill. A maximum rock size of 50 mm to 150 mm has been recommended. It is suggested that alternative options be given for the pile installation following the placement of the rockfill.

## II Foundation Investigation related comments

### 1.2 Site Description and Geological Setting

Fourth paragraph: the first two sentences should be made into one sentence.

#### 1.3.2 Field Investigation

Clarification is needed regarding the number of boreholes. BH-1FP to BH-11FP represents 11 boreholes. BH-16FP to BH-24FP represents 9 boreholes. Consequently, the total number of boreholes equals 20 boreholes and not 21 as indicated in the third paragraph of this section.

Reference to the ASTM designation should be included for the Standard Penetration Test and the in situ vane testing. More details should be given regarding the in situ vane test (vane type, method of calculation of sensitivity)

### 1.4 Subsurface Conditions

#### 1.4.1 Organics

The colour of the organics should be included in the description.

#### 1.4.2 Clay

The results of Atterberg Limits plotted on the Plasticity Chart reveal clays of medium plasticity. These soils are categorized as silty clays under the MTO Soil Classification system. The plasticity chart should include a title block that identifies the soil type.

Conventionally, the results of one dimensional consolidation testing is presented in the  $e$  vs  $\log p$  form rather than the strain vs stress format. In addition, discussion of the magnitude of preconsolidation ( $p'_c$  -  $p'_o$ ) should be included in the text.

### Bedrock

The quality of the bedrock based on RQD's should be included.

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



# MEMORANDUM



To: E.I. Gallant  
Senior Project Manager

September 25, 1998

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

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

Re: Wick Drain Design  
Trout Cr. North/South Interchange  
WP 774-93-00  
District 54, Sudbury

Further to our E-mails, this memorandum summarizes some of our comments regarding the wick drain efficacy and contains recommendations for the management of the wick drain design.

## Efficacy of Wick Drains

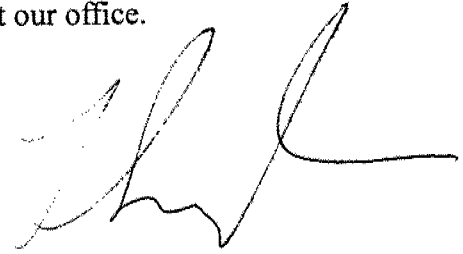
Wick drains are vertical drains manufactured from geosynthetic material that are installed to increase the rate of consolidation of compressible soils by decreasing the drainage path. There are several case histories that have shown that wick drains can successfully accomplish the task of accelerating the time rate consolidation settlement. As indicated at our meetings, the efficacy of wick drains have been demonstrated on two recent MTO projects at Hwy 416/County Rd 19(Contract 96-59) and the QEW/Twenty Mile Creek project(Contract 94-83).

Although it can be reasonably expected that the consolidation settlements will be accelerated, the process is time dependent and it is unrealistic to expect that no settlement will occur after paving. An instrumentation monitoring program will provide us with tangible results that will enable a determination of the status of settlement at any particular time. The instrumentation monitoring data will afford an assessment of the state of settlement. However, when embankment loadings induce pressures that exceed the preconsolidation pressure in the native subsoil, it is reasonable to expect that some settlement will occur following paving. The objective is to minimize the settlement to an acceptable magnitude that will not exceed a rating of "slight" severity as established by the Manual for Condition Rating of Flexible Pavements.

## Management of Wick Drain Design

Conceptually, wick drains have been identified as a viable option at the site. A detailed design will be necessary to optimize the wick drain system. Such a design requires expertise and proven experience. Consequently, it is recommended that a specialized subconsultant be retained to carry out this detailed design. Our office can supply the names of designated subconsultants or alternatively, the prime consultant can submit a designated subconsultant for our approval.

We trust this memorandum will assist in coordinating the embankment design at the site. If you require additional information, please do not hesitate to contact our office.

A handwritten signature in black ink, appearing to be 'T. Sangiuliano', written in a cursive style.

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-176DIST. 54 REGION W.P. No. 774-93-00CONT. No. W. O. No. STR. SITE No. HWY. No. 11LOCATION Trout Creek By-PassNorth Bound & South Bound LANESNo. of PAGES -=====OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. REMARKS:

GEOTECHNICAL # 24 116

**Geotechnical Foundation Report  
Embankment Section  
Trout Creek ByPass  
North Bound & Sound Bound Lanes  
Station 12+350 to 12+850  
GWP No. 774-93-00  
District 54, Sudbury**

Prepared for:

Marshall Macklin Monaghan  
80 Commerce Valley Drive, East  
Thornhill, ON  
L3T 7N4

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SO7524G/IF  
March, 1999

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**SO7524G/IF**

**MEMORANDUM NO. 2**

**TO:** R.D. Kivi, P.Eng.  
Senior Project Manager  
Marshall, Macklin, Monaghan

**FROM:** I.W. Gore, P.Eng.                      E.A. Gonneau, P.Eng.  
Principal Engineer                      Project Manager  
Trow Consulting Engineers Ltd.              Trow Consulting Engineers Ltd.

**SUBJECT:** Geotechnical Foundation Report  
Embankment Section  
Trout Creek ByPass, North Bound & South Bound Lanes  
Station 12+350 to 12+850  
GWP No. 774-93-00  
District 54 Sudbury

---

This memorandum addresses the geotechnical report for the design and construction of an embankment section between approximate stations 12+350 to 12+850 along the proposed NBL and SBL of the proposed Trout Creek ByPass, Highway 11, as part of GWP 774-93-00. The approximately 500 m long embankment is located from the existing McCarthy Street, extending southwesterly to the section where the existing ground rises, and the ByPass enters a "cut" section.

## 1.0 Introduction

This embankment section for the NBL and SBL of the proposed Trout Creek ByPass, Highway 11, runs from approximate station 12+350 up to approximate station 12+850, a distance of 500 m.

In the initial section, i.e. between stations 12+600 to 12+850, the terrain is flat and contains many tag-alders within the existing ground level at approximate elevation 315 m. In this area, the embankment is approximately 4 m high at station 12+800, rising to 8 m high at station 12+600. Beyond station 12+850, up to the present McCarthy Street at station 13+750, the proposed embankment height is less than 4 m, and the terrain is flat and open.

The existing ground is level, southwesterly from station 12+600, then rises gradually from the flat land at elevation 315 m through densely wooded terrain, up to approximate elevation 324 m at station 12+470. Correspondingly, the proposed NBL and SBL embankment grades also rise to elevation 327 m at this location. Southwesterly from station 12+325, the existing grade then rises steeply, and the proposed NBL and SBL enter into cut, at approximate station 12+300.

## 2.0 Field Work

The field work comprised thirteen (13) sampled boreholes (BH's 11F to 13IF, inclusive), together with five additional dynamic cone penetration tests driven adjacent to boreholes 11F, 6IF, 11IF, 12IF and 13IF. The locations of the boreholes are included on the attached site plan, Drawing 1, and on the individual borehole logs. The drilling was completed using a track-mounted soils drill equipped with hollow and solid stem flight augers between September 14 to 17, 1998, inclusive.

Details of the soil strata encountered in the boreholes and cones are included in the attached logs, and plotted on the profile included on Drawing 1. Further information on soil descriptions are contained on Drawings 2A and 2B.

### 3.0 Subsurface Conditions

#### 3.1 Station 12+600 to 12+850 (BH's 5IF to 12IF)

Based on the borehole data, the subsoil conditions along this flat terrain portion are reasonably uniform and consist of the following soil strata:

- *Organics*

The organics comprise a thin, surficial deposit of topsoil generally 100 mm to 300 mm thick. At borehole 11IF, however, the topsoil veneer is locally thicker, i.e. 600 mm.

- *Sand*

The predominant soil stratum throughout this section consists of sand, which has a generally loose to compact consistency. The deposit is at least 6 m thick.

Based on the laboratory gradation analyses (see attached data), the sand is predominantly fine-grained with less than 10% silt and clay sizes.

Typical standard penetration test results ("N" values) vary from 2 to 27 (average 12). The lower "N" values (less than 5) are not considered to be representative, since sampling disturbance likely occurred during the drilling and in-situ testing. It is noted that the dynamic cone penetration tests (cones driven adjacent to borehole 6IF, 11IF, 12IF and 13IF) confirm consistent blows, exceeding 20 blows/300 mm penetration.

Moisture contents of the fine sand are in the range of 18% to 22%.

- *Clayey Silt*

A localized deposit of clayey silt was encountered underlying the sand in boreholes 6IF and 11IF.

In borehole 6IF, the clayey silt deposit is less than 2.5 m thick and occurs at depths between 6.5 m (~El. 308.5 m) and 8.8 m (~El. 300.2 m). In borehole 11IF, the deposit is slightly thicker, i.e. 3.6 m



thick, and occurs at depths between 5.5 m (~El. 310.4 m) and 9.1 m (~El. 306.8 m). The clayey silt is stratified with intermittent, horizontal silt layers and undrained shear strengths (as measured with in-situ vanes and laboratory shear tests) varying from 22 kPa to 30 kPa.

Natural moisture contents range from 40% to 55%, with Atterberg limits of 22% (plastic limit) to 38% and 42% (liquid limit).

The clayey silt is similar in characteristics and properties to the clayey deposit encountered at the adjacent North Interchange. For the North Interchange, extensive detailed laboratory analyses have been undertaken.

- ***Silt***

A lower zone of loose to compact silt was encountered in borehole 11IF below the clayey silt. This silt stratum was intercepted at a depth of 9.2 m (~El. 306.8 m) and continues for approximately 4.5 m, i.e. down to a depth of 13.7 m (~El. 302.2 m). Standard penetration blows varied from 3 to 9 blows/300 mm, indicating a loose to compact consistency. Dynamic cone blows, through the deposit, exceed 40 blows/300 mm penetration. Laboratory shear strength tests and a field vane test (completed in "clayey" seams) confirmed undrained shear strength values exceeding 45 kPa. In-situ moisture contents of the deposit are in the range 25% to 35% with one Atterberg limit test confirming a plastic limit of 18% and a liquid limit of 28%.

- ***Silty Sand and Gravel Till***

A 2 m thick zone of dense silty sand and gravel till ("N" blows >40) was encountered at the base of borehole 5IF, i.e., below the upper compact sand.

### **3.2 Stations 12+350 to 12+600 (BH's 1IF to 4IF)**

As the ground rises in this section, the upper veneer of sand is thinner and the lower levels of the boreholes encountered a dense glacial till. The properties and sequence of soil deposits is described below:

- *Organics*

The organics in this section are thin and comprise less than 100 mm of topsoil.

- *Sand*

The sand is generally a fine-grained deposit with traces of silt and occasional gravel inclusions. The thickness of the sand varies from less than 2.5 m in boreholes 3IF and 4IF to 5.4 m in borehole 1IF.

Standard penetration tests established "N" blows of 6 to 16, confirming a loose to compact condition. In-situ moisture contents are less than 20%.

- *Clayey Silt*

A localized, approximately 1.5 m thick, pocket of clayey silt was intercepted in borehole 1IF at a depth of 5.4 m (~El. 317.6 m). The clayey silt has the following characteristics:

- undrained shear strength ~50 kPa (laboratory shear test)
- in-situ moisture content - 36%
- Atterberg limits      Plastic Limit - 21%  
                                    Liquid Limit - 33%
- "N" value - 5 blows/300 mm
- dynamic cone penetration blows - >20 blows/300 mm

- *Sand and Gravel Till*

Sand and gravel till was encountered at the base of the boreholes and varies in thickness from about 1.0 m in borehole 3IF to slightly greater than 3.5 m thick in borehole 2IF.

Standard penetration blows ("N" values) are in the range of 15 to 50 blows/300 mm penetration (average 30 blows), with in-situ moisture contents of 8% to 20%.

## 4.0 Groundwater

In the embankment portion where the existing terrain is flat and poorly drained (boreholes 5IF to 13IF, stations 12+600 to 12+850), the groundwater table is at grade.

In the higher terrain (boreholes 1IF to 4IF, station 12+350 to 12+600), the groundwater table is slightly lower, i.e. at a depth of 1 m to 2 m below grade.

Seasonal fluctuations in the level of the groundwater table can be expected.

## 5.0 Recommendations

The proposed NBL and SBL will require embankment heights of up to about 8 m.

Since the majority of the subsoil is competent, granular soils (loose to compact sands and/or dense till), no stability nor long-term consolidation problems are envisaged. However, where the localized clayey silt deposit was encountered, estimated to be within a localized section beneath the NBL and SBL between stations 12+615 and 12+645, some precautionary measures must be considered. In these sections, the following procedures are recommended:

- ***Stability***

The embankment height(s) in this localized section is 7.5 m. The soil strata consists of 5.5 m of compact sand over 3.6 m of loose to compact, clayey silt. The undrained shear strength of the clayey silt is 22 kPa (minimum). Although the factor of safety, under these conditions, meets or exceeds 1.3, (see stability analysis, Figure 4 to 6), it is nevertheless recommended that the embankment be constructed gradually, increasing the height over a minimum of 3 months.

- ***Settlement***

The calculated settlement of the embankments, due to consolidation of the 3.7 m thick zone of clayey silt, is estimated to be in the order of 150 mm. However, 50% of this settlement will be completed within 3 months and 90% within a period of 7 months, after full height has been reached. As such,

all the anticipated consolidation settlement should occur within 8 months, once the embankment has been constructed to full height.

## 6.0 Embankment Design

The proposed embankments are 8 m in height or less. Since only fairly minor consolidation settlements are expected, i.e. a maximum of 150 mm within one localized section, which, in any event, should occur within 8 months after construction, it is recommended that the embankments be constructed either with rock fill having a side slope of 1.25H:1V, allowing for a 2.0 m wide, mid-height bench where the height exceeds 6 m, or the bouldery material contained within the cut area to the south of Trout Creek, having a side slope of 2.5H:1V, allowing for a 3.0 m wide, mid-height bench.

Settlement within the rock fill or till fill embankment itself should be expected over the first 2 to 3 years after construction. It is likely that this movement could be in the order of 1% to 2% of the height, i.e. up to about 150 mm beneath the 8 m high embankment section. This settlement could, however, be reduced significantly (to less than 50 mm) if the rock fill was restricted in size to a maximum of 600 mm diameter and then thoroughly compacted, using a 10 ton vibratory drum roller, with four or five passes in 1 m lifts of rock fill.

To allow for future potential grade raises, the rock fill embankments should be constructed a minimum of 1 m wider than standard on each side, to the bottom of the subgrade, in the section between stations 12+615 and 12+645.

## 7.0 Construction Considerations


The upper organic veneer 200 mm (average) of topsoil should be stripped off down to firm bottom before placement of the fill.

## 8.0 Closure

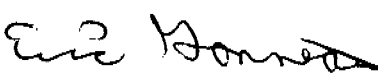
The field investigations were supervised by Mr. E.A. Gonneau, P.Eng., Project Manager. The memorandum report was written by Mr. I.W. Gore, M.Sc., P.Eng., Principal Geotechnical Engineer, and reviewed by Mr. S.E. Gonsalves, P.Eng.

Yours truly,

**TROW CONSULTING ENGINEERS LTD.**

  
I.W. Gore, M.Sc., P.Eng.  
Principal Engineer

  
S.E. Gonsalves, P.Eng.  
Vice-President

  
E.A. Gonneau, P.Eng.  
Project Manager

Encl.  
Dist:

**NOTES ON SAMPLE DESCRIPTIONS**

1. All descriptions included in this report follow the I.S.S.M.F.E. as suggested in the Canadian Foundation Manual. The laboratory grain-size analysis also follows this classification system. Others may designate the unified classification system as their source; a comparison of the two is shown for your information. Please note that, with the exception of those samples where the grain-size analysis has been carried out, all samples are classified visually and the accuracy of visual examination is not sufficient to differentiate between the classification systems or exact grain sizing.

UNIFIED SOIL CLASSIFICATION	Fines (silt or clay)			Sand			Gravel			Cobbles		
				Fine	Medium	Coarse	Fine	Coarse				
I.S.S.H.F.E. SOIL CLASSIFICATION	Clay	Silt			Sand			Gravel			Cobbles	
		Fine	Medium	Coarse	Fine	Medium	Coarse	Fine	Medium	Coarse		
		Sieve Sizes										
		Particle Size (mm)										
		<div><div>0.001</div><div>0.002</div><div>0.003</div><div>0.004</div><div>0.006</div><div>0.008</div><div>0.01</div><div>0.02</div><div>0.03</div><div>0.04</div><div>0.06</div><div>0.075</div><div>0.08</div><div>0.1</div><div>0.2</div><div>0.3</div><div>0.4</div><div>0.5</div><div>0.6</div><div>0.8</div><div>1.0</div><div>2.0</div><div>3.0</div><div>4.0</div><div>6.0</div><div>8.0</div><div>10</div><div>20</div><div>30</div><div>40</div><div>60</div><div>80</div></div>										

2. **FILL:** Where fill is designated on the borehole log, it is defined as indicated by the sample recovered during the boring process. The reader is cautioned that fills are heterogeneous in nature and variable in density or degree of compaction. The borehole description may therefore not be applicable as a general description of the site fill material. All fills should be expected to contain obstructions such as large concrete pieces of subsurface basements, floors, tanks, etc.; none of these may have been encountered in the borehole. Since boreholes cannot accurately define the contents of fill, test pits are recommended to provide supplementary information. Despite the use of test pits, the heterogeneous nature of fill will leave some ambiguity as to the exact and correct composition of the fill. Most fills contain pockets, seams, or layers of organically contaminated soil. This organic material can result in the generation of methane gas and/or significant on-going and future settlements. Some fill material may be contaminated by toxic waste that renders it unacceptable for deposition in any but designated land fill sites. Unless specifically stated, the fill on this site has not been tested for contaminants that may be considered hazardous. This testing and a potential hazard study can be carried out if you so request. In most residential/commercial areas undergoing reconstruction, buried oil tanks are common but are not detectable using conventional geotechnical procedures.
3. **TILL:** The term till on the borehole logs indicate that the material originates from a geological process associated with glaciation. As a result of this geological process, the till must be considered heterogeneous in composition and, as such, may contain pockets and/or seams of material such as sand, gravel silt or clay. As till often contains cobbles (60 to 200 mm) or boulders (over 200 mm), contractors may encounter them during excavation, even if they are not indicated by the borings. It should be appreciated that normal sampling equipment cannot differentiate the size, or type of any obstruction. Because of the horizontal and vertical variability of till, the sample description may be applicable to a very limited areas; caution is therefore essential when dealing with sensitive excavations or dewatering programs in till material.

## NOTES ON SAMPLE DESCRIPTIONS (Cont'd)



Project No: S07524G/IP

Drawing No: 2B

4. The following table gives a description of the soil based on particle sizes. With the exception of those samples where grain-size analyses have been performed, all samples are classified visually. The accuracy of visual examination is not sufficient to differentiate between this classification system or exact grain size.

Soil Classification		Terminology	Proportion
Clay	< 0.002 mm	"trace" (eg. trace sand)	1% - 10%
Silt	0.002 to 0.06 mm	"some" (eg. some sand)	10% - 20%
Sand	0.06 to 2 mm	adjective (eg. sandy)	20% - 35%
Gravel	2 to 60 mm	and (eg. and sand)	> 35%
Cobbles	60 to 200 mm	noun (eg. boulders)	> 35% and main fraction
Boulders	> 200 mm		

Classification system as suggested in the Canadian Foundation Engineering Manual, 3rd Edition, unless otherwise noted.

The compactness of cohesionless soils and the consistency of cohesive soils are defined by the following:

Cohesionless Soil		Cohesive Soil	
Compactness	Standard Penetration Resistance "N" Blows/0.3 m	Consistency	Undrained Shear Strength (kPa)
Very Loose	0 to 4	Very Soft	< 12
Loose	4 to 10	Soft	12 - 25
Compact	10 to 30	Firm	25 - 50
Dense	30 to 50	Stiff	50 - 100
Very Dense	Over 50	Very Stiff	100 - 200
		Hard	> 200

### 5. Rock Coring

Where rock drilling was carried out, the term RQD (Rock Quality Designation) is used. The RQD is an indirect measure of the number of fractures and soundness of the rock mass. It is obtained from the rock cores by summing the length of core recovered, counting only those pieces of sound core that are 100 mm or more in length. The RQD value is expressed as a percentage and is the ratio of the summed core lengths to the total length of core run. The classification based on the RQD value is given below.

RQD Classification	RQD
Very poor quality	< 25
Poor quality	25 - 50
Fair quality	50 - 75
Good quality	75 - 90
Excellent quality	90 - 100

$$\text{Recovery Designation \% Recovery} = \frac{\text{Length of Core Per Run}}{\text{Total Length of Run}} \times 100$$

# RECORD OF BOREHOLE BH-11F

1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station 12+387, on centreline of Northbound Lane

ORIGINATED BY S.M.

DIST 54 HWY 11

BOREHOLE TYPE Standard augers / Dynamic cone

COMPILED BY M.D.

DATUM Geodetic

DATE September 14, 1998

CHECKED BY I.G.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)		CONE PENETRATION TEST		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA	NUMBER	TYPE			BLOWS/0.3m	20	40	60					
323.00	GROUND SURFACE														
0.00	TOPSOIL, ~50 mm over SAND, fine to medium grading to fine silty sand with depth, brown then grey below ~3.5 m depth, moist then wet below ~2 m depth. (loose to compact)		1	SS	8										0% 92% 8%
			2	SS	16										
			3	SS	13										
317.60	CLAYEY SILT, grey, wet. (loose)		4	SS	5										
316.00	SILTY SAND & GRAVEL TILL, brown. (compact)		5	SS	14										
314.92	END OF BOREHOLE														
8.08	Probable TILL														
313.25	END OF CONE TEST DUE TO "BOUNCING" REFUSAL ON PROBABLE BEDROCK OR POSSIBLE BOULDER														
9.75	Notes: 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Water level was at ~2.3 m & hole was open to ~5.1 m depth on completion. 3) Dynamic cone penetration test driven adjacent BH-11F.														





# RECORD OF BOREHOLE BH-2IF 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station ~12+450, on centreline of Northbound Lane ORIGINATED BY S.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / COMPILED BY M.D.  
 DATUM Geodetic DATE September 14, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION					
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	20	40	60	80	wp	w	wp	w			10	20	30	40	kN/m <sup>3</sup>
321.30 0.00	GROUND SURFACE																					
	TOPSOIL, ~75 mm over SAND, fine with SILT content, occasional stiff CLAYEY SILT layers, brown then grey below ~3 m depth, wet. (loose to compact)		1	SS	6																	
		2	SS	10																		
317.30 4.00	SILTY SAND & GRAVEL TILL, some cobble sizes, grey. (compact to dense)		3	SS	27																	
		4	SS	44																		
313.98 7.32	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER																					
	Notes: 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Water level was at ~2.0 m & hole was open to ~4.1 m depth on completion.																					



# RECORD OF BOREHOLE BH-3IF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station 12 + 495, on centreline of Median  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers /  
 DATUM Geodetic DATE September 14, 1998  
 ORIGINATED BY S.M.  
 COMPILED BY M.D.  
 CHECKED BY I.G.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION  GR SA (SI & CL)
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60	80	wp	w	wl		
320.00	GROUND SURFACE															
0.00	TOPSOIL, 50 mm over SAND, fine, brown, moist. (compact)															
318.60																
1.40	SAND & GRAVEL TILL, brown, moist. (dense)		1	SS	42											
317.71																
2.29	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER  Notes: 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Borehole was dry & open to ~0.8 m depth on completion.															



# RECORD OF BOREHOLE BH-4IF 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station 12+550, on centreline of Median ORIGINATED BY S.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / COMPILED BY M.D.  
 DATUM Geodetic DATE September 14, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT				UNIT WEIGHT  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION	
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20   40   60   80				wp ——— w ——— wl					
								SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)					
							● UNCONFINED QUICK TRIAXIAL	⊗ FIELD VANE LAB SHEAR									
318.50	GROUND SURFACE																
0.00	TOPSOIL, ~100 mm over SAND, fine, trace of SILT, brown to grey, wet at base. (loose to compact)																
316.50			1	SS	7										0%   77%   23%		
2.00	SILTY SAND & GRAVEL TILL, cobbles & possible boulders, grey. (compact to dense)		2	SS	27												
			3	SS	48												
312.75																	
5.75	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER																
	Notes: 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Water level was at ~2.4 m & hole was open to ~3.1 m depth on completion.																



# RECORD OF BOREHOLE BH-5IF 1 OF 1

METRIC

W.P. 774-93-00 LOCATION Station ~12+600, on centreline of Median ORIGINATED BY S.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / COMPILED BY M.D.  
 DATUM Geodetic DATE September 14, 1998 CHECKED BY I.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT				UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE	BLOWS/0.3m			20	40	60	80	wp	w	wl			
								SHEAR STRENGTH: Cu, KPa ● UNCONFINED QUICK TRIAXIAL    X FIELD VANE LAB SHEAR 20 40 60 80				WATER CONTENT (%) 10 20 30 40					
316.40 0.00	GROUND SURFACE <b>TOPSOIL</b> , ~100 mm over <b>SAND</b> , fine, trace of SILT, brown, moist to wet. (compact to dense)						316									GR SA (SI & CL)	
			1	SS	27		315										
							314										
313.90 2.50	<b>SILTY SAND &amp; GRAVEL TILL</b> , grey. (dense)		2	SS	48		313										
311.98 4.42	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER  Notes: 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Water level was at ~0.6 m & hole was open to ~2.2 m depth on						312										



# RECORD OF BOREHOLE BH-6IF

1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station 12 + 650, on centreline of Median

ORIGINATED BY S.M.

DIST 54 HWY 11

BOREHOLE TYPE Standard augers / Dynamic cone

COMPILED BY M.D.

DATUM Geodetic

DATE September 17, 1998

CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			UNCONFINED QUICK TRIAXIAL	Cu, KPa					
315.00	GROUND SURFACE					315							
0.00	PEATY TOPSOIL, ~200 mm over SAND, mostly fine with a trace of SILT & occasional GRAVEL sizes, grey brown, wet. (loose to compact)		1	SS	7								
			2	SS	13								
			3	SS	2								
308.50			4	SS	0								
6.50	CLAYEY SILT, grey, wet. (loose)		5	SS	4								
306.16													
8.84	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON PROBABLE BEDROCK OR POSSIBLE BOULDER												

Notes:  
 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass.  
 2) Water level was at ~0.2 m & hole was open to ~4.0 m depth on completion.  
 3) Dynamic cone penetration test driven adjacent BH-6IF.



# RECORD OF BOREHOLE BH-7IF 1 OF 1

## METRIC

W.P. 774-93-00 LOCATION Station +12+700, on centreline of Median ORIGINATED BY S.M.  
 DIST 54 HWY 11 BOREHOLE TYPE Standard augers / COMPILED BY M.D.  
 DATUM Geodetic DATE September 17, 1998 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION		
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	SHEAR STRENGTH: Cu, KPa				WATER CONTENT (%)				
								UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR	wp	w	wl				
315.00	GROUND SURFACE															
0.00	PEATY TOPSOIL, ~300 mm over SAND, fine with a trace of SILT content, grey, wet. (loose to compact)															
			1	SS	4											
			2	SS	17											
			3	SS	20											
			4	SS	27											
308.45	END OF BOREHOLE															
6.55	<b>Notes:</b> 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Water level was at ~0.1 m & hole was open to ~1.8 m depth on completion.															



# RECORD OF BOREHOLE BH-8IF

1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station ~12+750, on centreline of Median

ORIGINATED BY S.M.

DIST 54 HWY 11

BOREHOLE TYPE Standard augers /

COMPILED BY M.D.

DATUM Geodetic

DATE September 17, 1998

CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	20	40					
315.00 0.00	GROUND SURFACE													
	PEATY TOPSOIL, ~300 mm over SAND, mostly fine with a trace of SILT, brown to grey, wet. (compact)		1	SS	10									
			2	SS	15									
			3	SS	13									
			4	SS	26									
308.45 6.55	END OF BOREHOLE													
	Notes: 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Water level was at ~0.1 m & hole was open to ~2.6 m depth on completion.													



# RECORD OF BOREHOLE BH-9IF

1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station 12+800, on centreline of Median

ORIGINATED BY S.M.

DIST 54 HWY 11

BOREHOLE TYPE Standard augers /

COMPILED BY M.D.

DATUM Geodetic

DATE September 17, 1998

CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			BLOWS/0.3m	20	40	60	80	wp	w		
315.00	GROUND SURFACE					315									
0.00	TOPSOIL, ~150 mm over SAND, mostly fine with a trace of SILT, grey. (loose to compact)		1	SS	5										
			2	SS	16										
			3	SS	14										
			4	SS	12										
308.45	END OF BOREHOLE					309									
6.55	<p>Notes:</p> <p>1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass.</p> <p>2) Water level was at ~0.5 m &amp; hole was open to ~1.9 m depth on completion.</p>														





# RECORD OF BOREHOLE BH-10IF 1 OF 1

METRIC

W.P. 774-93-00  
 DIST 54 HWY 11  
 DATUM Geodetic

LOCATION Station ~12 + 850, on centreline of Median  
 BOREHOLE TYPE Standard augers /  
 DATE September 17, 1998

ORIGINATED BY S.M.  
 COMPILED BY M.D.  
 CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)		CONE PENETRATION TEST		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	UNCONFINED QUICK TRIAXIAL	FIELD VANE LAB SHEAR	Wp	Wl	Wp	Wl	Wp	Wl		
314.80 0.00	GROUND SURFACE																
	PEATY TOPSOIL, ~300 mm over SAND, mostly fine with a trace of SILT, brown to grey, wet below ~0.6 m depth. (compact)		1	SS	8												0% 97% 3%
			2	SS	15												
			3	SS	14												
			4	SS	12												
308.25 6.55	END OF BOREHOLE																
<p>Notes:</p> <p>1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass.</p> <p>2) Water level was at ~0.7 m &amp; hole was open to ~2.3 m depth on completion.</p>																	



# RECORD OF BOREHOLE BH-11IF 1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station ~12+630, on centreline of Southbound Lane

ORIGINATED BY S.M.

DIST 54 HWY 11

BOREHOLE TYPE Hollow stem augers / Dynamic cone

COMPILED BY M.D.

DATUM Geodetic

DATE September 15, 1998

CHECKED BY I.G.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST				WATER CONTENT (%)				UNIT WEIGHT kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER	TYPE			20	40	60	80	10	20	30	40		
315.90 0.00	GROUND SURFACE															
	PEATY TOPSOIL, ~610 mm over SAND, mostly fine with a trace of SILT & occasional GRAVEL sizes, grey brown, wet below ~1.0 m depth. (loose to compact)		1	SS	9											
			2	SS	17											
			3	SS	2											
310.40 5.50	CLAYEY SILT, grey, stratified with thin wet silt seams. (loose to compact)		4	TW												
			5	SS	2											
306.75 9.15	SILT, trace of CLAY, wet, grey. (loose to compact)		6	SS	3											
			7	SS	5											
			8	SS	9											
302.18 13.72	END OF BOREHOLE DUE TO REFUSAL TO AUGER ON BEDROCK OR BOULDER															
	Notes: 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Water level was at ~1.3 m & hole was open to ~3.3 m depth on completion. 3) Dynamic cone penetration test driven ~1.5 m east of BH-11IF.															



# RECORD OF BOREHOLE BH-12IF 1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station 12+750, on centreline of Southbound Lane

ORIGINATED BY S.M.

DIST 54 HWY 11

BOREHOLE TYPE Hollow stem augers / Dynamic cone

COMPILED BY M.D.

DATUM Geodetic

DATE September 15, 1998

CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value)		CONE PENETRATION TEST		PLASTIC LIMIT		NATURAL MOISTURE CONTENT		LIQUID LIMIT		UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION
ELEV. DEPTH	DESCRIPTION	STRATA PLOT	NUMBER			TYPE	BLOWS/0.3m	20	40	60	80	wp	w	wp	w		
315.30	GROUND SURFACE																
0.00	PEATY TOPSOIL, ~360 mm over SAND, mostly fine with a trace of SILT, wet below ~0.8 m depth. (compact)		1	SS	14												
			2	SS	14												
			3	SS	11												
			4	SS	12												
307.22	END OF BOREHOLE		5	SS	10												
8.08	Notes: 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 522 Overpass. 2) Borehole caved wet at ~0.9 m depth on completion.																



# RECORD OF BOREHOLE BH-13IF 1 OF 1

METRIC

W.P. 774-93-00

LOCATION Station ~12+850, on centreline of Southbound Lane

ORIGINATED BY S.M.

DIST 54 HWY 11

BOREHOLE TYPE Hollow stem augers / Dynamic cone

COMPILED BY M.D.

DATUM Geodetic

DATE September 14, 1998

CHECKED BY I.G.

SOIL PROFILE		SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE (metres)	SPT TEST (N-Value) CONE PENETRATION TEST	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION		
ELEV. DEPTH	DESCRIPTION	STRATA PLLOT	NUMBER							TYPE	BLOWS/0.3m
314.80 0.00	GROUND SURFACE										
	PEATY TOPSOIL, ~300 mm over SAND, mostly fine with a trace of SILT, wet below ~0.9 m depth. (compact)		1	SS	13						
			2	SS	17						
			3	SS	10						
			4	SS	5						
306.72 8.08	END OF BOREHOLE		5	SS	15						

Notes:  
 1) This borehole forms part of the New Alignment of Highway 11, north of the Highway 11 Overpass.  
 2) Borehole caved wet at ~1.0 m depth on completion.  
 3) Dynamic cone penetration test driven ~1.5 m west of BH-13IF.



# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT

SAND

GRAVEL

FINE

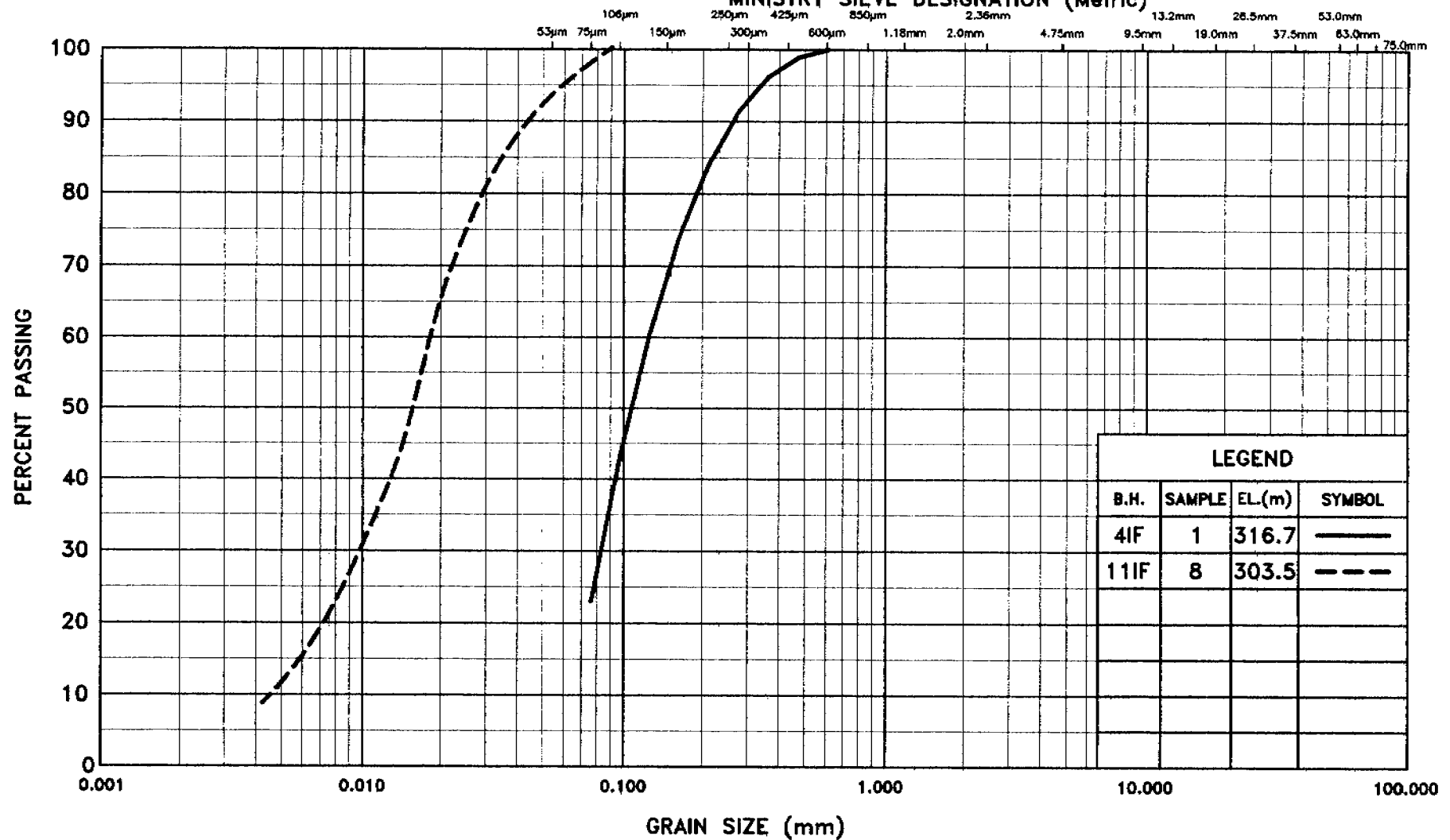
MEDIUM

COARSE

FINE

COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION

SILTY SAND & SILT

FIGURE 1

W.P 774-93-00

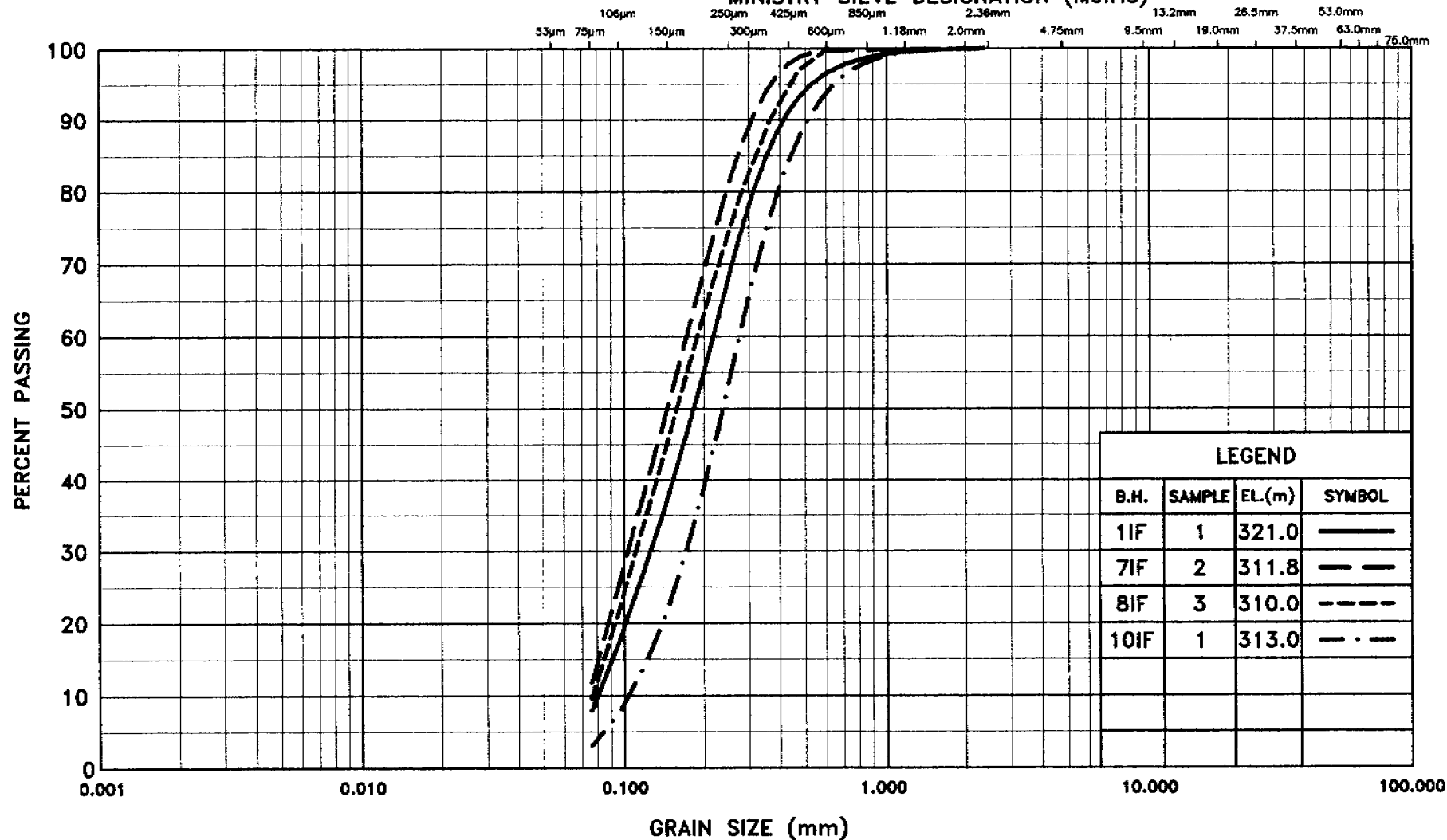


PROJ. No. S07524GIF

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION

SAND

FIGURE 2

W.P 774-93-00

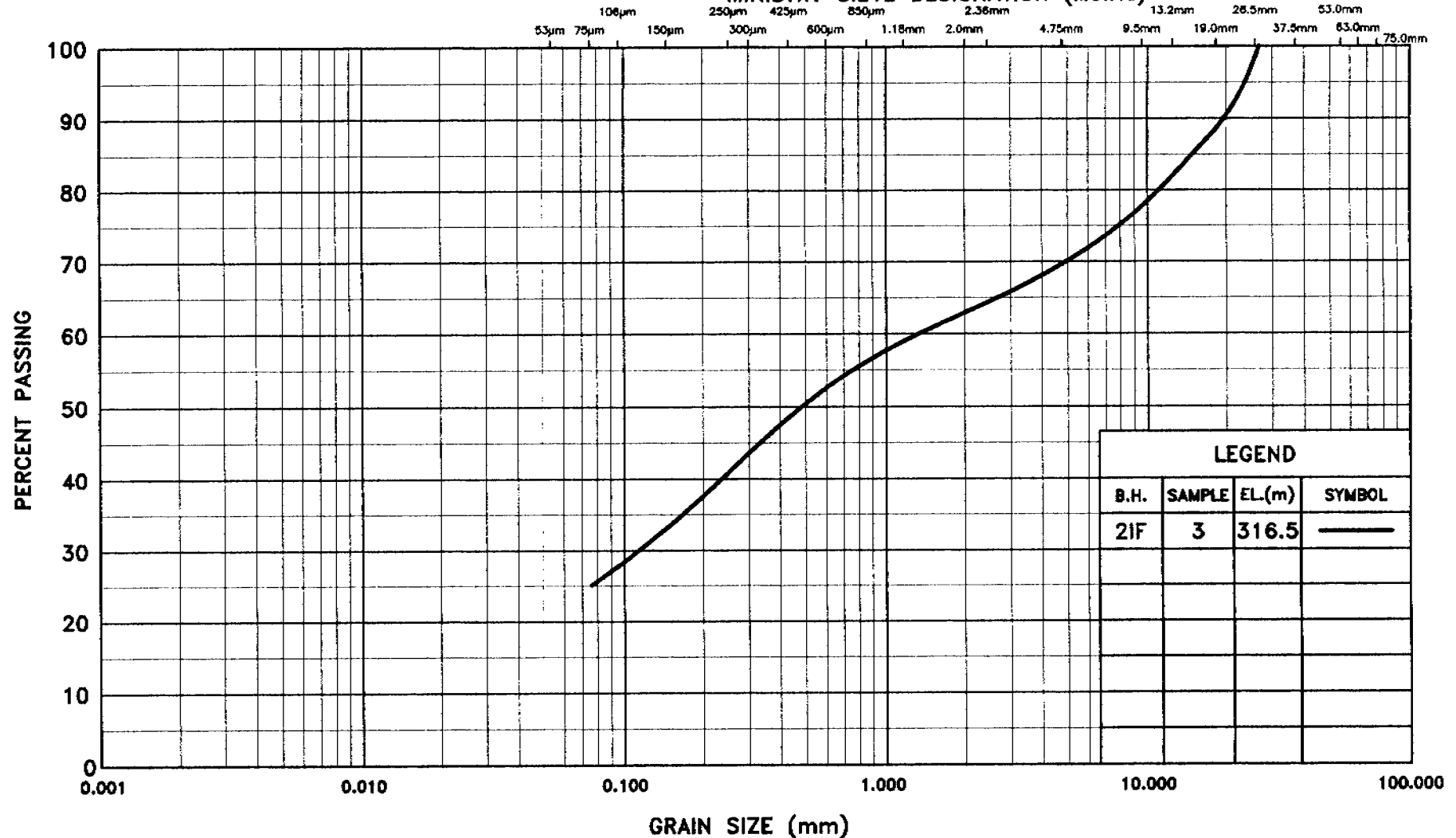


PROJ. No. S07524GIF

# UNIFIED SOIL CLASSIFICATION

CLAY AND SILT	SAND			GRAVEL	
	FINE	MEDIUM	COARSE	FINE	COARSE

## MINISTRY SIEVE DESIGNATION (Metric)



Ministry of  
Transportation

METRIC

ALL SAMPLES

GRAIN SIZE DISTRIBUTION

SAND & GRAVEL TILL

FIGURE 3

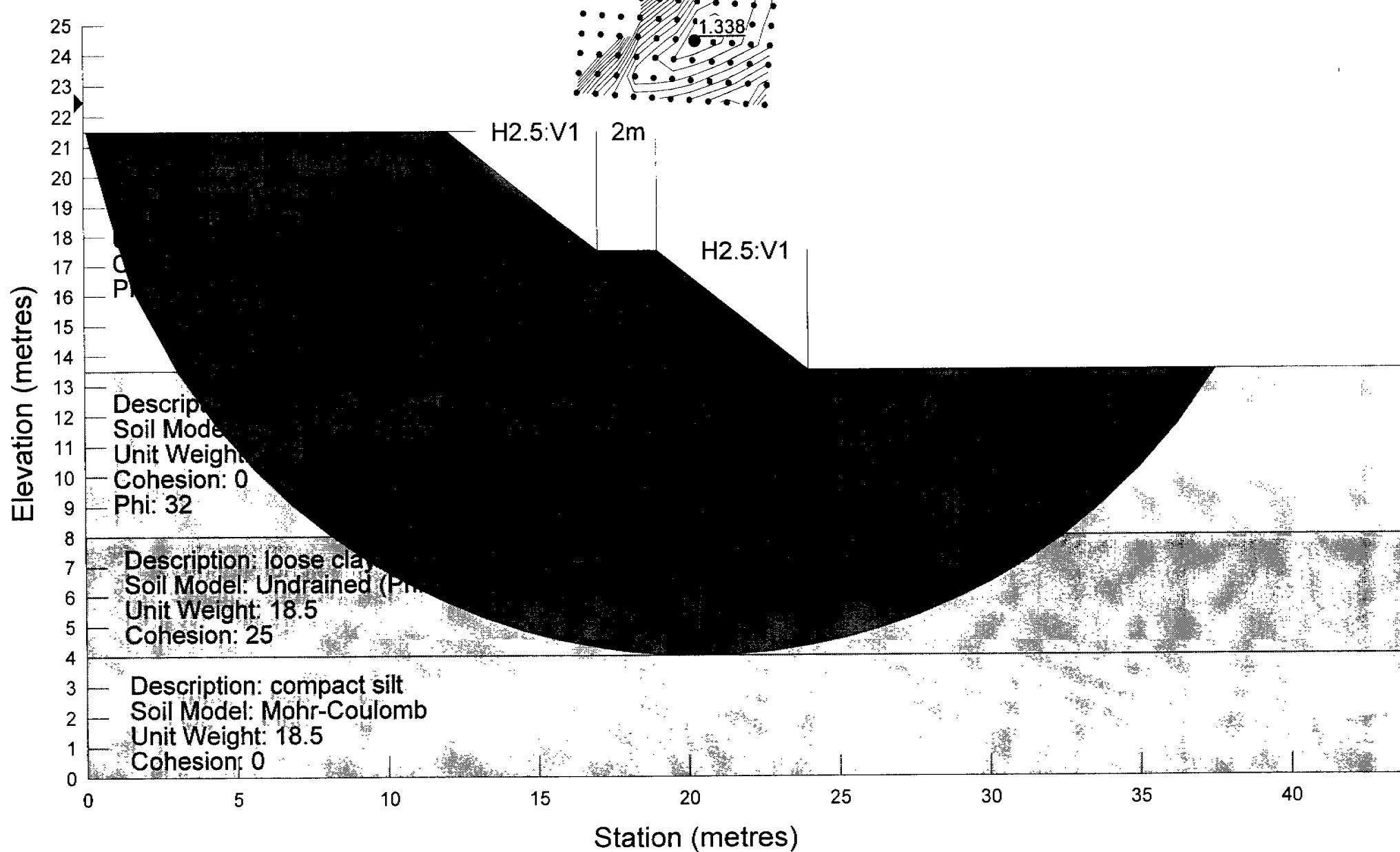
W.P 774-93-00



PROJ. No. S07524GIF

S07524G/IF  
Trout Creek By-Pass  
Station 12+500 to 13+500

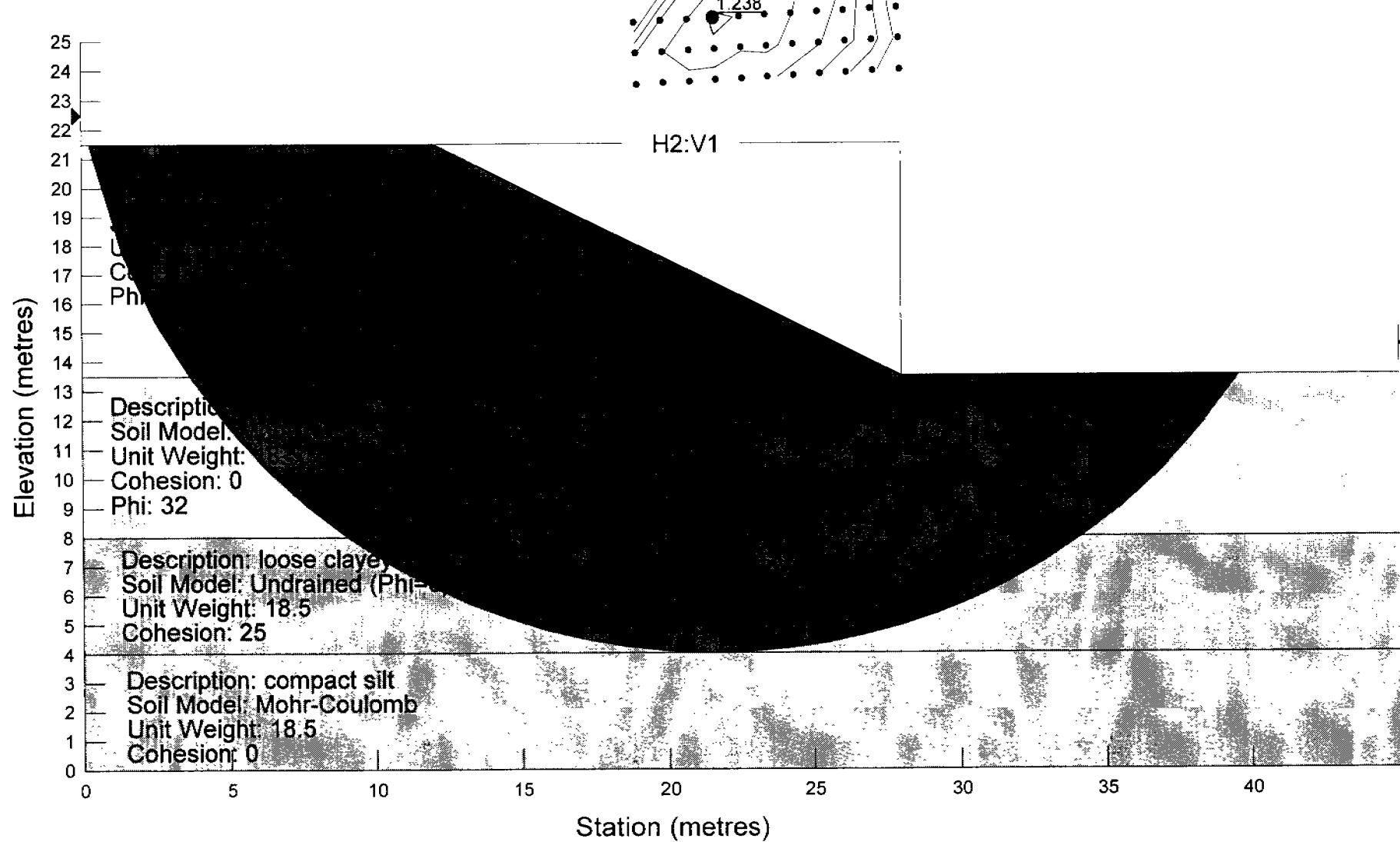
Figure 4.





S07524G/IF  
Trout Creek By-Pass  
Station 12+500 to 13+500

Figure 5



S07524G/IF  
 Trout Creek By-Pass  
 Station 12+500 to 13+500

Figure 6

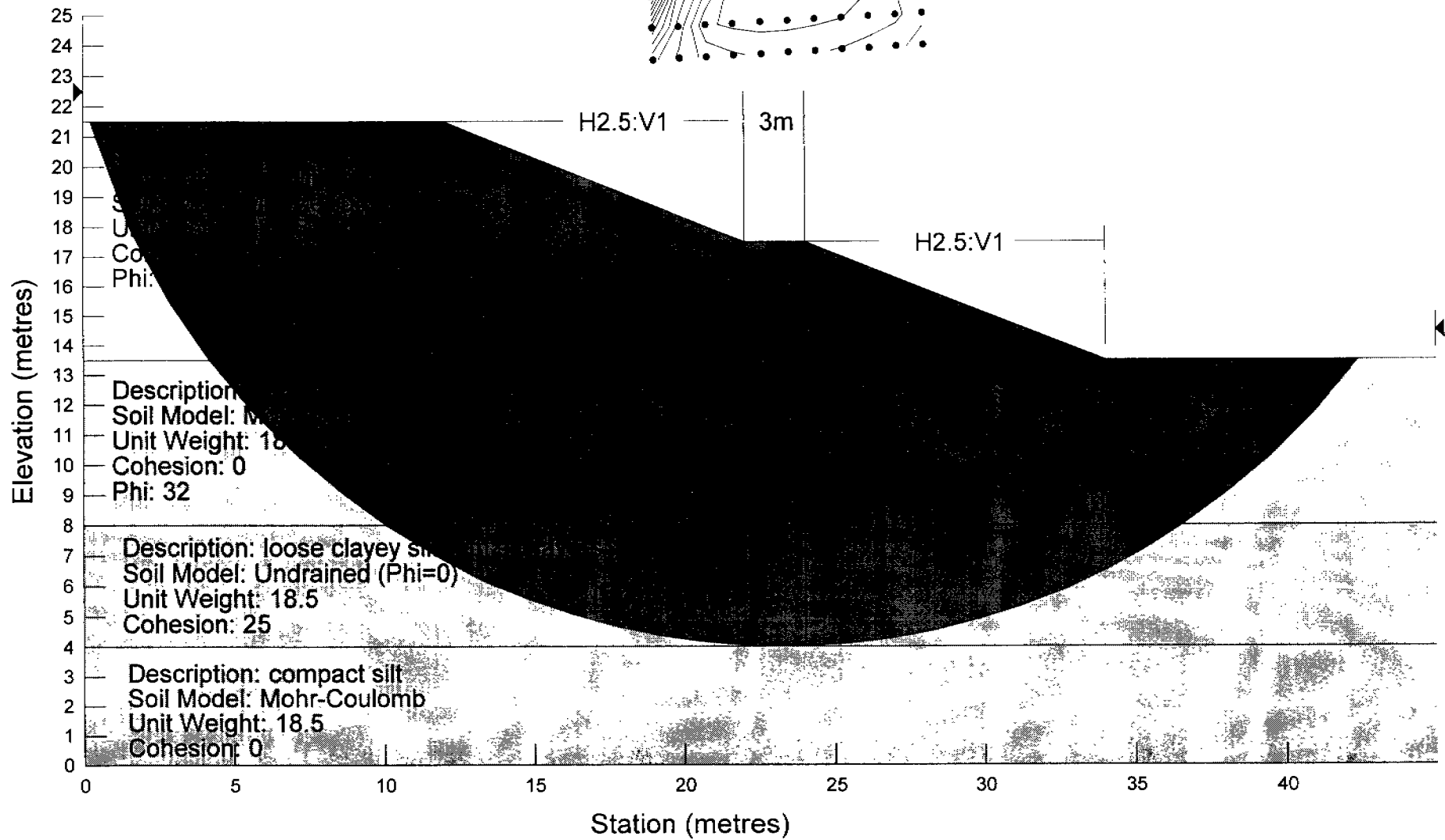
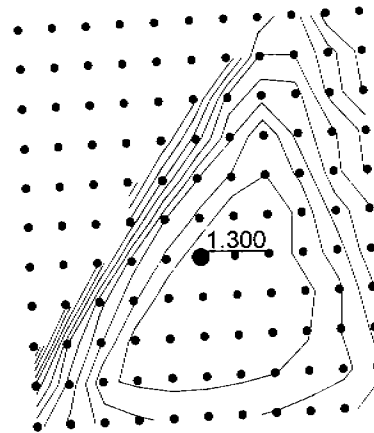
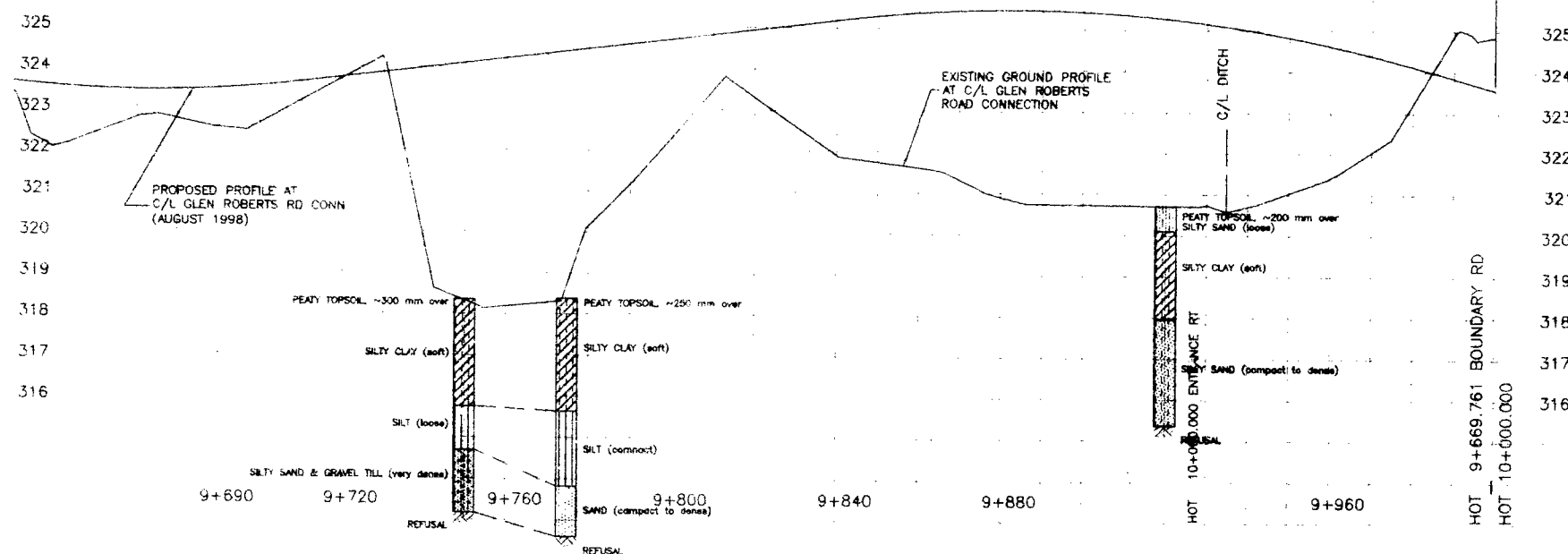
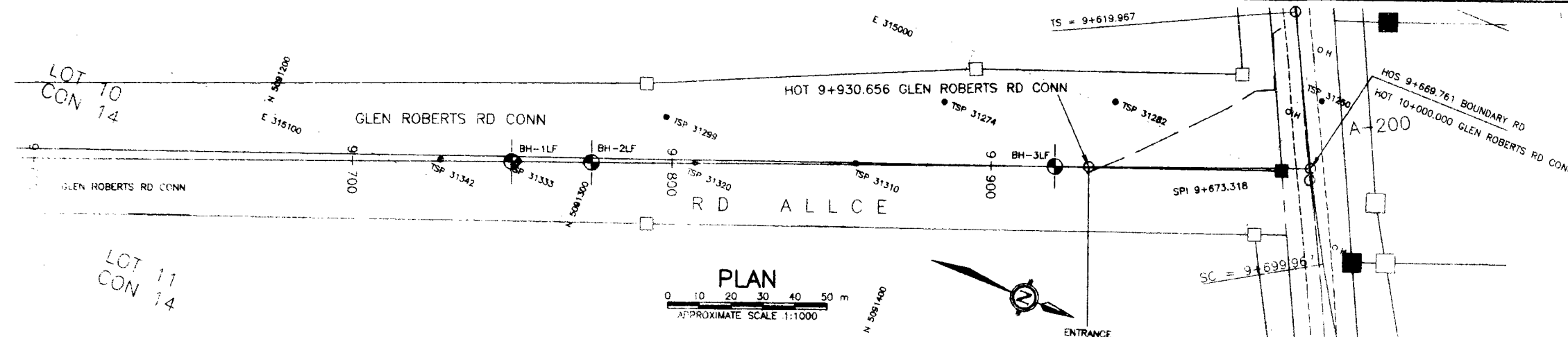


PLATE No  
 DRAWING No  
 CONT No  
 WP No

774-93-00

SHEET



PROFILE  
 HORIZONTAL SCALE 1:1000  
 VERTICAL SCALE 1:100

Note  
 Borehole elevations, northings and eastings are interpolated from plans and profiles provided by Marshall Macklin Monaghan

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

TROW CONSULTING ENGINEERS LTD. SUDBURY, ONTARIO PROJ. No. S07524GLF DWG. No. 1		
MINISTRY OF TRANSPORTATION ENGINEERING OFFICE SURVEYS AND PLANS SECTION		
SITE PLAN & CROSS SECTION GLEN ROBERTS ROAD CONNECTION AT HIGHWAY 11 TROUT CREEK BYPASS		
GEOG. TWP. OF LAURIER LOT 10/11		DIST. OF PARRY SOUND CON 14
SCALE AS SHOWN	DISTRICT 54, SUDBURY	REGION NORTHERN
SURVEY DATE: AUGUST 1998 PLAN DATE: 98/08		
SITE		PLAN



G.I.-30 SEPT. 1976

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 31E-176

DIST. 54 REGION \_\_\_\_\_

W.P. No. 774-93-00

CONT. No. \_\_\_\_\_

W. O. No. \_\_\_\_\_

STR. SITE No. \_\_\_\_\_

HWY. No. 11

LOCATION Trout CREEK By-Pass

North Bound + South Bound LANES

No of PAGES -

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. \_\_\_\_\_

REMARKS: \_\_\_\_\_

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