GEOCRES No:
31E-245

DRAFT

LIQUEFACTION ASSESSMENT

OTTAWA AVE STRUCTURE, SBL

HIGHWAY 11 FOUR LANING

BURK'S FALLS TO SOUTH RIVER, ONTARIO

G.W.P. 759-93-00

Geocres Number: 31E-245

Report to

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BURK'S FALLS TO SOUTH RIVER, ONTARIO

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ENGINEERING DISCUSSION AND RECOMMENDATIONS

1 GENERAL

This report presents the results of an analysis of the liquefaction potential of the foundation soils beneath the approach fills to the proposed structure carrying HWY 11 south bound lane over Ottawa Ave. This assessment is based on the proposed design provided by MMM dated May 21, 2004.

The discussion and recommendations presented in this report are based on our understanding of the project and on the existing factual data obtained in the course of the MTO terms and requirements for foundation investigation. The subsurface data is summarized in the foundation report for the crossing and is not duplicated herein. No specific field investigation was carried out for this liquefaction assessment. This report should therefore be read in conjunction with the report entitled:

Foundation Investigation and Design Report
Ottawa Avenue Overpass SBL
Highway 11 Burk's Falls to south River
GWP 759-93-00, W.P. 750-93-01, Site 44-414

The stratigraphy encountered in the boreholes near the Ottawa Ave. structure is described in detail in the above noted report, and generally consists of 28.9 to 30.4 m of glacio-fluvial overburden over Pre-Cambrian bedrock. The overburden materials encountered in the boreholes consist of topsoil, sand, sandy silt to sand and silt and cobbles and boulders.

2 ENGINEERING ANALYSIS METHODOLOGY

2.1 General

The methodology used to assess the liquefaction susceptibility of the foundation consists of several steps:

- Screening of subsurface information to delineate strata not considered susceptible to liquefaction.
- Estimate of future dynamic loads associated with the design earthquake and calculation of cyclic stress ratio (CSR) within the deposit
- Estimate existing cyclic resistance ratio (CRR) within the deposit based on available density and soil gradation data.
- Estimate change in cyclic resistance associated with construction of the embankment and the associated change in density.
- Delineate zones susceptible to potential liquefaction (where CRR<CSR). Estimate change in stability of embankment and potential deformation resulting from design seismic event.

2.2 Screening For Liquefaction Potential

An initial screening of the deposit was carried out to generally determine which portions of the soil strata would be susceptible based on soil type (cohesive or non-cohesive), relative density and location of the water-table. The screening was carried out using the following criteria:

Chinese Criteria (Wang, 1979):

The Chinese criteria suggests that soils that meet all of the criteria below will be subject to liquefaction. Soils that do not meet the criteria are considered to be cohesive and not susceptible to liquefaction.

▪ percentage of particles finer than 0.005 mm	<15%
▪ Liquid limit	<35%
▪ water content	>0.9 • Liquid limit
▪ liquidity index	< 0.75

Relative Density

Screening was also carried out based on the relative density of the deposit as indicated by SPT N-values. Soil strata with SPT N-values >30 were not considered to be susceptible to liquefaction.

Saturation

Initial screening was carried out to separate portions of the deposit which were above the seasonal water-table and not considered to be saturated.

Soils that were not considered susceptible to liquefaction based on the above screening criteria may still be subject to deformation at high Cyclic Stress Ratios (CSR) through settlement of unsaturated sand or cyclic mobility / loss of shear strength in cohesive materials. However, the CSR associated with the design seismic event are relatively low and the sand deposits have relatively high dynamic shear moduli (~300MPa) which will result in small shear strains occurring within the deposits.

2.3 Seismic Loading

The following seismic parameters have been used in the liquefaction assessment

- Velocity Related Seismic Zone 1
- Zonal Velocity Ratio 0.05
- Acceleration Related Seismic Zone 2
- Zonal Acceleration Ratio 0.1

In accordance with the CHBDC, the soil profile type at this site is classified as Type I (less than 60 m of stable sand, gravel or stiff clay), which according to Table 4.4.6.1 of the CHBDC is associated with a Site Coefficient of 1.0. A surface peak horizontal ground acceleration (PHA) of 0.11g, where g is the acceleration due to gravity, has therefore been used in this analysis. This PHA value corresponds to a probability of exceedance of 10% in 50 years.

For the purpose of estimating the cyclic loading for assessment of liquefaction the cyclic loading is calculated using an "equivalent uniform CSR" equal to 65% of the single peak CSR. This uniform CSR is applied over a duration of shaking representing a standard magnitude 7.5 earthquake. The cyclic stress ratio is calculated using the following formula:

$$CSR = 0.65\ (a_{max}/g)\ (\sigma_{vo}/\sigma'_{vo})\ r_d$$

The variation of CSR within the deposit is calculated using a stress reduction coefficient, r_d. This coefficient accounts for the flexibility of the soil column and varies with the type soil profile. The relationship proposed by Seed and Idriss (1971) is commonly used to calculate r_d for routine analysis.

2.4 Evaluation of Liquefaction Resistance

The resistance of the deposit to liquefaction under seismic loading is calculated as the cyclic resistance ratio (CRR). In this study, the CRR parameter at a given location is estimated primarily from the measured SPT N-value and the fines content as recovered in the split spoon sampler. The CRR of a deposit is estimated based on a comparison with database from a large number of case histories where liquefaction was observed or not observed following historic seismic events. The current CRR envelope recommended for use in assessment of liquefaction has been summarized by Youd et al (2001).

To allow comparison with other test data, the field N-values were first corrected for hammer energy, overburden pressure, borehole diameter, rod length and sampler type. The corrected SPT values are referred to as $(N_1)_{60}$.

The fines content influences the apparent CRR, with higher CRR values associated with higher fines content. The SPT N-values were corrected based on fines content to produce an "equivalent clean sand" value using the method of Cetin et al (2004).

The CRR is also dependant on the effective overburden stress because higher stress tends to increase the resistance to liquefaction. Surface manifestations of liquefaction occurring at depths greater than 15 m are rarely documented. Correction for overburden stresses are therefore typically not carried out as part of conventional liquefaction assessment above 15 m.

The CRR also must be adjusted for the duration of shaking induced by earthquakes of different magnitudes. The CRR curves have been developed for the duration of shaking associated with a M7.5 earthquake. Calculation of CRR for other magnitudes is achieved by the use of a Magnitude Scaling Factors (MSF). The frequency of large M7.5 earthquakes in relatively stable central Canada is much less than along the Pacific Rim where the majority of the liquefaction case histories originate. The design acceleration (probability of 10% in 50 years) in central Canada, is produced by a spectrum of potential seismic events, each with different magnitudes and epicentral distances from the site. The design acceleration that the site will experience could therefore be a result of a smaller magnitude events at closer distance or from larger magnitude event farther from the site. Deaggregation of the seismic hazard for major Canadian cities has been presented by Halchuk and Adams (2004). For data from the southern Ontario region, the mean magnitude for short period accelerations with a probability of 10% in 50 years is about M6.2. The MSF for a M6.2 earthquake was therefore applied as per Youd et al (2001).

2.5 Embankment Construction

The immediate settlement associated with the proposed embankment loading will change the in-situ stresses and relative density of the foundation from that originally measured and therefore must be considered in the liquefaction analysis. The change in relative density of the deposit was calculated based on the Tangent Modulus method of Janbu, adjusted for the site conditions encountered in the boreholes. The changes in stress were calculated using 2-dimensional stress formulation for an embankment on an elastic half-space.

2.6 Post-Liquefaction Behaviour

2.6.1 Stability

The stability of the embankment was assessed during a post-liquefaction scenario, based on the method of Olson and Stark (2003). This method utilizes a reduced undrained yield strength for zones of liquefied soils. The dynamic inertial force within the slide mass was not analysed concurrently with the reduced undrained strength as the relatively low CSR values and the modest number of cycles for smaller magnitude earthquakes in central Canada make it unlikely that liquefaction would occur until near the end of the earthquake.

The foundation strength was modelled by reducing the shear strength of zones that are considered susceptible to liquefaction. The full static shear strength was assigned to zones where liquefaction is not anticipated. For zones that are susceptible to liquefaction, the undrained shear strength ratio was set based on the SPT N-value recorded in that zone according to the relationship based on the method of Olson and Stark (2003).

The analysis was carried out using Bishops' modified limit equilibrium method using the program GSlope produced by Mitre software.

2.6.2 Settlement

Liquefaction of sediments will result in the densification of both saturated and unsaturated materials. Post-liquifaction settlements of saturated sediments were estimated using the method of Ishihara (1985).

2.6.3 Lateral Spreading

Liquefaction beneath a flood plain or adjacent to a river or stream often results in liquefaction-induced lateral spreading of the deposit usually affecting large areas. The mode of failure can be characterized by two types of behaviour 1) lateral spread toward a free face and 2) lateral spread of an inclined surface (generally inclined at 0.1 % to 6%). The potential for this type of behaviour is assessed using the empirical method developed by Barlett and Youd (1995).

This mode of failure is not considered possible given the geometry of the terrain adjacent to the Highway 11 SBL.

3 EMBANKMENT DESIGN

3.1 General

The proposed embankment for SBL of Highway 11 varies in height from approximately 6 m at the south approach to 10.8 m at the north approach. The proposed embankment for Ottawa Ave. is less than 2.5 m in height. Based on the proposed embankment geometry and the subsurface conditions encountered in the boreholes, the north approach embankment is considered the most susceptible to liquefaction induced instability or settlements. A critical section was therefore selected from this area for detailed analysis.

The soil conditions encountered at the site generally consist of a layer of sandy silt overlying sand. The water-table is at the existing ground surface.

3.2 Liquefaction Potential

The screening for liquefaction potential of the deposits indicate that the overlying sandy silt is relatively low to non-plastic and may be susceptible to liquefaction if subjected to an earthquake of sufficient acceleration and duration. The sandy silt strata was therefore included in the more detailed analysis to assess the CRR relative to the design CSR.

The CRR of the entire deposit was calculated according to the methods described above for each soil type and with increasing depth. The results are summarized in Figure 1 and Figure 2. The results indicate that approximately 20 to 30% of the deposit would be susceptible to liquefaction during the design earthquake. The locations of the susceptible areas are shown in Figure 3.

The susceptible areas are typically located between 5 to 18 m depth and are concentrated in boreholes drilled by Golder using hollow-stem auger methods. The use of hollow stem auger methods in cohesion-less saturated deposits can result in inaccurate SPT N-values. Control of fluid pressures and soil heaving within the borehole can be best achieved using wet drilling methods. All borehole data has been included in the analysis to fill out the data set and provide a lower bound estimate of the relative density of the deposit.

3.3 EMBANKMENT RESPONSE

3.3.1 Embankment Stability

The stability of the embankment immediately following an earthquake was assessed using reduced shear strength in potentially liquefied zones as indicated by the analysis described in the preceding section. The distribution of SPT N-values adjusted for construction of the embankment and location of potential liquefaction for the design section is shown in Figure 5.

It is expected that the presence of a sandy silt layers within the sand will impede drainage of excess pore pressure following the earthquake, resulting in elevated pore pressures and reduced shear strength for several hours following the earthquake. Analysis of embankment sideslopes and headslopes were carried out as summarized below.

Embankment Sideslopes

The results of the analysis indicate that the embankment sideslope will be have a factor of safety F<1 for a finite period following the event. Based on the stability analysis the potential instability will likely affect an area from the toe of slope extending to approximately 6 m inside of the edge of road shoulder. Displacements are expected to be several metres. The results of the analysis are shown in Figure 4.

Head Slopes

Stability analysis of the head slope at the bridge abutment using post-liquefaction shear strength indicates a Factor of Safety of 2. The results of the analysis are shown in Figure 5 in Appendix A.

3.3.2 Embankment Foundation Settlement

There is potential for settlement associated with drainage of excess pore pressure and the densification of loose sand following soil liquefaction. The upper bound estimate of settlement based on the method of Ishihara, 1985, indicates a maximum volumetric strain of 5%. However for this site, the estimated configuration will be a 6 m liquefied layer located beneath a 5 m layer of intact soil. In this case, the maximum surface settlement is expected to be less than 200 mm. The settlements are expected to occur over zones where liquefaction within the foundation occurs. The location of the maximum settlement is therefore predicted to be beneath the sides and toe of the embankment and no significant settlement is expected beneath the central portion of the embankment.

3.3.3 Lateral Spreading

The analysis indicates that lateral spreading is not kinematically possible for the South-Bound Embankment geometry. A floodplain (Black Creek) is located east of the proposed mainline embankment and lateral spreading is therefore considered for the North-bound lanes in a separate report.

3.4 DESIGN OPTIONS

3.4.1 General

Seismic design requirements for bridges are provided in the Canadian Highway Bridge Design Code. The requirements for bridges differ depending on the classification of the highway as a "Lifeline", "Emergency route" or "Other". The requirements for each classification are provided below:

Lifeline	Must remain open to all traffic following design earthquake
Emergency Route	Must be useable by emergency vehicles and for security/defence purposes immediately after the design earthquake

No specific guidelines are provided for embankments, but a similar level of performance to that required for structures can be inferred.

The following design options were considered to address the potential for liquefaction within the embankment foundation at this site:

- Reinforce embankment
- Construct stabilizing berm
- Combined berm with geogrid reinforcement
- Repair embankment after seismic event
- Foundation Treatment

3.4.2 Embankment Reinforcement

The preliminary analysis for a reinforced embankment shows no significant improvement in the stability of the embankment sideslopes. The results of the analysis are shown in Figure B1 in Appendix B. This option was therefore not considered further.

3.4.3 Stabilizing Berm

The stability of an embankment can be improved in some situations by adding berms at the toe of slope. Analysis of an embankment with stabilizing berms constructed on a foundation with shear strength representative of post-liquefaction conditions shows no significant improvement in the Factor of Safety. The results of the analysis are shown in Figure B2 in Appendix B. Since the instability of the embankment is expected to extend about 6 m back from the crest of the embankment, the potential for impact of instability on the traffic lanes could be decreased significantly by widening the embankment at full-height by at least 8 m horizontally.

3.4.4 Combined Berms and Embankment Reinforcement

An analysis was carried out using combined berm and geogrid reinforcement options applied together. The initial results of the undrained analysis, Figure B3, indicate that the stability of this configuration is marginal (F=0.95). The berms themselves have a factor of Safety, F=0.85, as shown in Figure B4, and are expected to undergo shear movements following seismic loading. However, the flattening of the berms as a result of slope movements is expected to improve the stability of the main embankment slope, as shown in Figure B5.

The analysis indicates that a Factor of Safety of 1.1 could be achieved for berms 20 m wide and half the height of the embankment. The reinforcement layers will require a combined capacity of 400 kN/m (5 layers with 80kN/m).

3.4.5 Repair After Seismic Event

The analysis indicates that liquefaction would occur beneath the toe of the embankment at some locations following the design earthquake. The impacts of the liquefaction have been estimated and are described above. Based on this analysis, the likely impact will be loss of serviceability of the outside lane of the facility in localized areas if no special design measures are adopted. The inside lanes of the NBL and SBL mainline are expected to remain serviceable following the design events. Based on the analysis the facility is expected to be useable by emergency vehicles following the event, but may have reduced capacity and speed limits until repairs can be made. The loss of strength in the foundation would be a short term effect, and repair of the embankment following liquefaction would be possible after the event.

3.4.6 Foundation Treatment

Densification of the portions of the foundation that are susceptible to liquefaction can be an effective method to reduce the risk of reduced serviceability of the highway following a seismic event. Methods of densification of loose, cohesion-less soil include: vibro-compaction, dynamic compaction or compaction piles.

The high water-table conditions at this site may preclude the use of dynamic compaction methods which typically requires a 2 m thick layer of unsaturated soils overlying saturated soils.

3.4.7 Comparison of Options

A simplified cost benefit analysis has been carried out to compare the improvement options described above. The results of this analysis are summarized in Table 1 following the text of the report. Based on this analysis the minimum cost is associated with the option to repair the embankment after the event. Therefore it is concluded that it may be more beneficial to repair the embankment after liquefaction had occurred than to design the embankment to withstand the event with minimal damage.

4 CLOSURE

The estimated liquefaction susceptibility is very sensitive to the SPT N-values recorded in the field. Conventional auger drilling methods may yield N-values that will result in conservative an overestimation the amount of liquefaction. Accordingly, specific drilling and sampling methods, and the use of such as Seismic Cone Penetration Test to correlate SPT N-values and equipment would be required to obtain more precise estimates of the potential for liquefaction. These specific investigation methods are outside the current scope of field investigation for the project.

Engineering analysis and report preparation by:

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NOV. 29.05
PROVINCE OF ONTARIO

Highway 11, SBL - Ottawa Avenue Approach Embankment
Cost - Benefit Analysis

Design Option	Estimated Capital Cost	Potential Repair Cost	Probability during 50 yrs	Present Value	Comments
Stabilizing Berm	$ 3,000	0	10%	$ 3,000	Widen embankment 8 m x 10 m high
Foundation Improvement					
Vibro-compaction	$ 14,200	0	10%	$ 14,200	20 m wide treatement area extending to 10 m depth
Dynamic Compaction	$ 5,800	0	10%	$ 5,800	May not be effective if high water-table conditions
Compaction Piles	$ 14,200	0	10%	$ 14,200	
Do-nothing	$ -	$ 1,500	10%	$ 150	Estimated repair (2005 dollars)

Notes:

1 Costs are per metre length of embankment
2 Cost estimates shown above are based on assumed design parameters for each mitigative option and are provided only for comparison of the relative costs.
3 Associated indirect costs resulting partial loss of service following a seismic event are not included.

TABLE 1

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Olson, S.M. and Stark, T.D. "Yield strength ratio and liquefaction analysis of slopes and embankments", *J. Geotech. Geoenviron. Eng.*, 129(8), 727-737.

Youd, T.L. et al. (2001). "Liquefaction resistance of soils: summary report from the 1996 NCEER and 19998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils". *J. Geotech. Geoenv Eng.* 127(1), 817-833.

Appendix A Liquefaction Analysis

THURBER

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

NOTE: Hierarchy of Soil Strength Prediction

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

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE

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

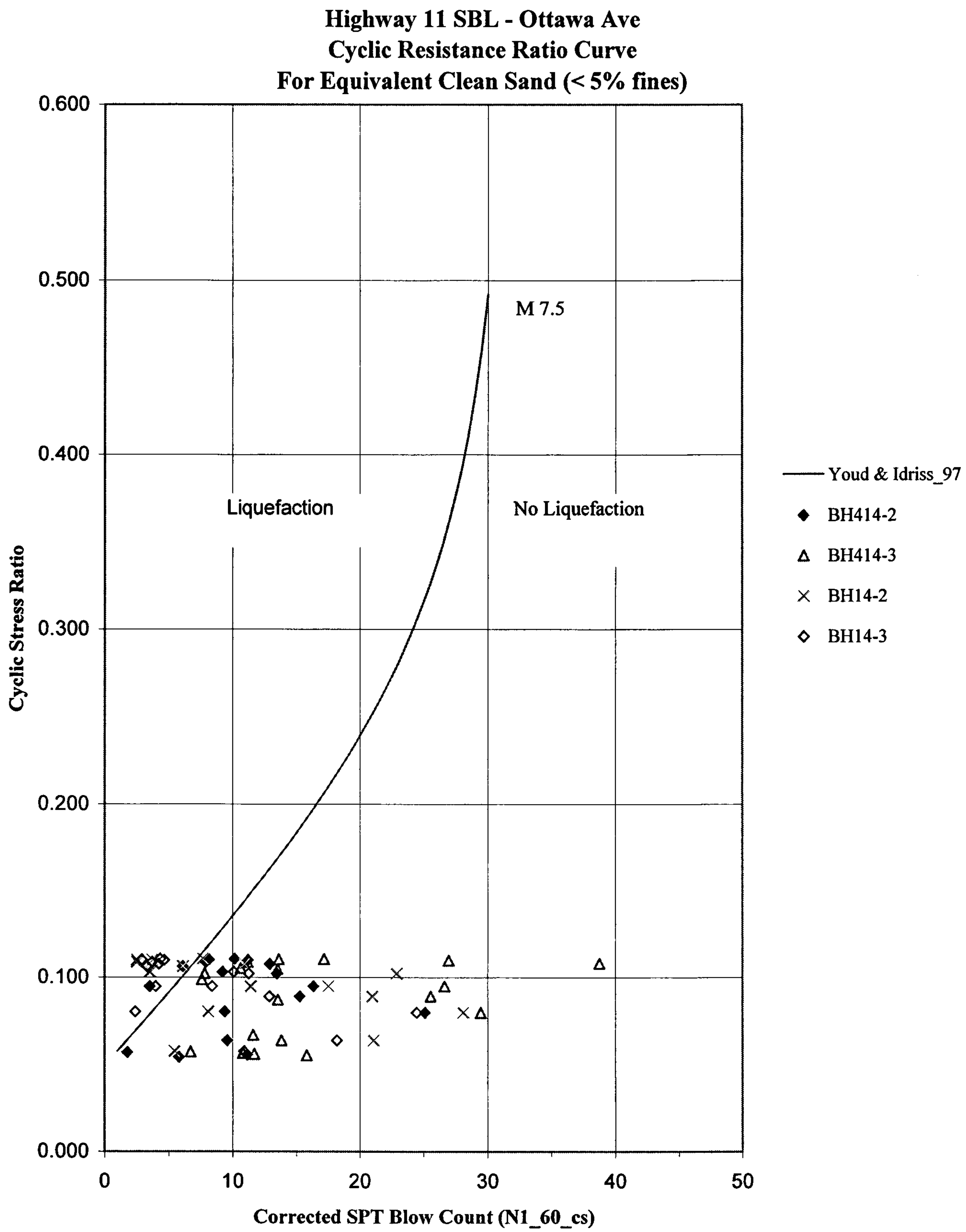
$$\text{Sensitivity} = \frac{\text{Undisturbed Shear Strength}}{\text{Remoulded Shear Strength}}$$

$\overline{\nabla}$ Water Level

C_{pen} Shear Strength Determination by Pocket Penetrometer

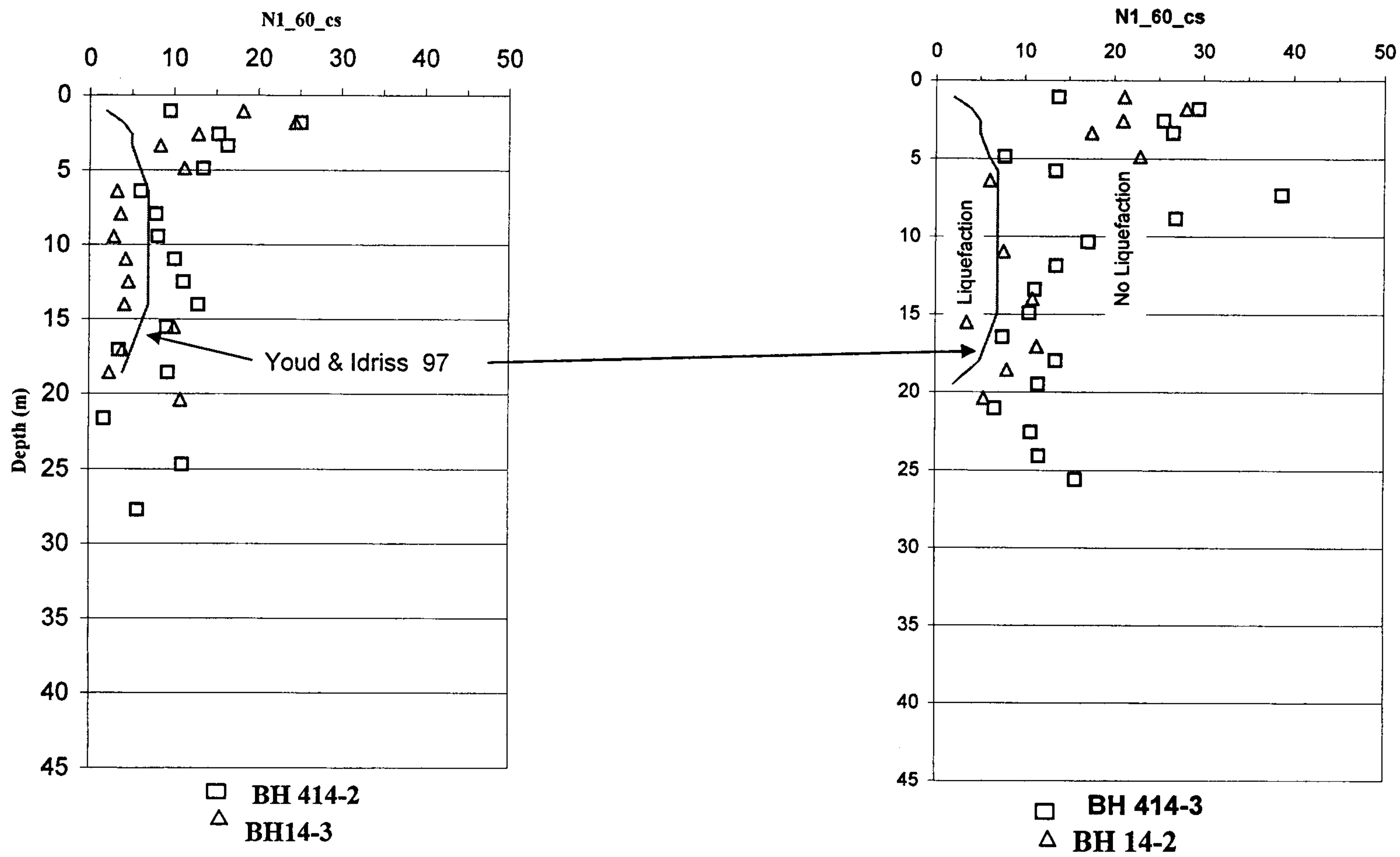
(1) SPT 'N' Value — Standard Penetration Test 'N' Value – refers to the number of blows from a 63.5kg hammer free falling a height of 0.76m to advance a standard 50 mm outside diameter split spoon sampler for 0.3 m depth into undisturbed ground.

(2) DCPT — Dynamic Cone Penetration Test – Continuous penetration of a 50 mm outside diameter, 60° conical steel point attached to "A" size rods driven by a 63.5 kg hammer free falling a height of 0.76 m. The resistance to cone penetration is the number of hammer blows required for each 0.3 m advance of the conical point into undisturbed ground.



Highway 11 SBL - Ottawa Ave
Cyclic Resistance Ratio Curve
For Equivalent Clean Sand (< 5% fines)
Cyclic Stress Ratio
0.600
0.500
0.400
0.300
0.200
0.100
0.000
0
10
20
30
40
50
Corrected SPT Blow Count (N1_60_cs)
M 7.5
Liquefaction
No Liquefaction
Youd & Idriss_97
BH414-2
BH414-3
BH14-2
BH14-3

FIGURE A1



Existing Conditions

FIGURE A2

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

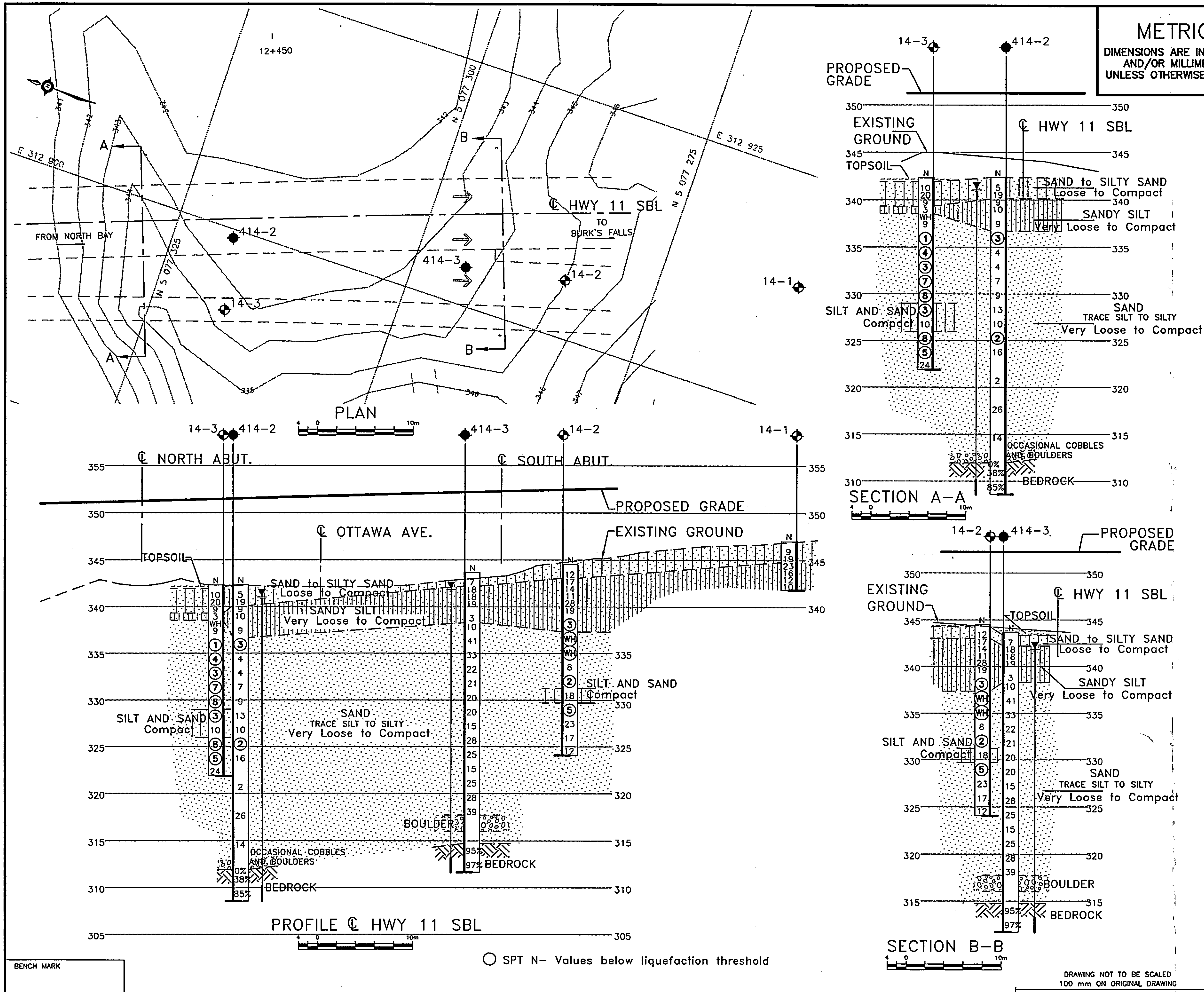
HWY 11
CONT No
WP No 750-93-01

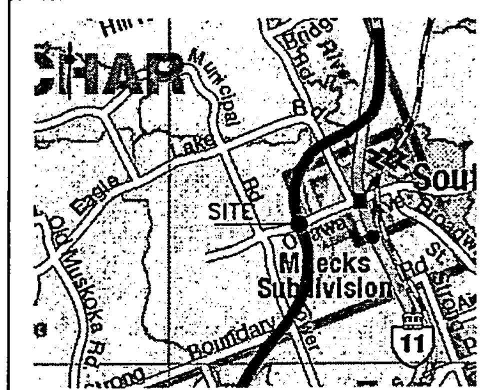
OTTAWA AVENUE
HWY 11 SBL
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

Marshall Macklin Monaghan
PROJECT MANAGERS • ENGINEERS • SURVEYORS • PLANNERS

THURBER ENGINEERING LTD.





KEYPLAN

LEGEND

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

NO	ELEVATION	NORTHING	EASTING
414-2	342.3	5 077 318.8	312 899.1
414-3	343.6	5 077 294.6	312 904.0
14-1	347.0	5 077 260.8	312 913.3
14-2	344.5	5 077 284.3	312 906.0
14-3	342.3	5 077 317.2	312 891.6

— NOTE —

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

FIGURE A3

REVISIONS			
NOV. 05	SMS	BRIDGE MOVE TO EXISTING OTTAWA AVENUE	
DATE	BY	DESCRIPTION	

DESIGN AEG	CHK	CODE CHBDC 2000	LOAD CL-625-ONT	DATE SEPT. 2004	
DRAWN SS	CHK AEG	SITE 44-414	STRUCT .	SCHEME .	DWG 2

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Embankment	22	2	32	0	0
silty Sand	21	0	30	0	1
Sandy Silt_dilt	20	25	0	0	1
Sandy Silt_cont	19.9	0	0	.07	1
Sand1	19.5	0	0	.08	1
Sand2	19.5	0	0	.09	1
Sand3	19.5	0	0	.1	1
Sand4	19.8	0	31	0	1
Bedrock	26	500	30	0	2

Thurber Engineering Ltd. - Toronto
19-1423-26
Hwy 11 - Burk's Falls
March 2005
Ottawa Ave, SBL
Liquefaction Analysis

Side Slope

Potential Slope instability associated with liquefaction event

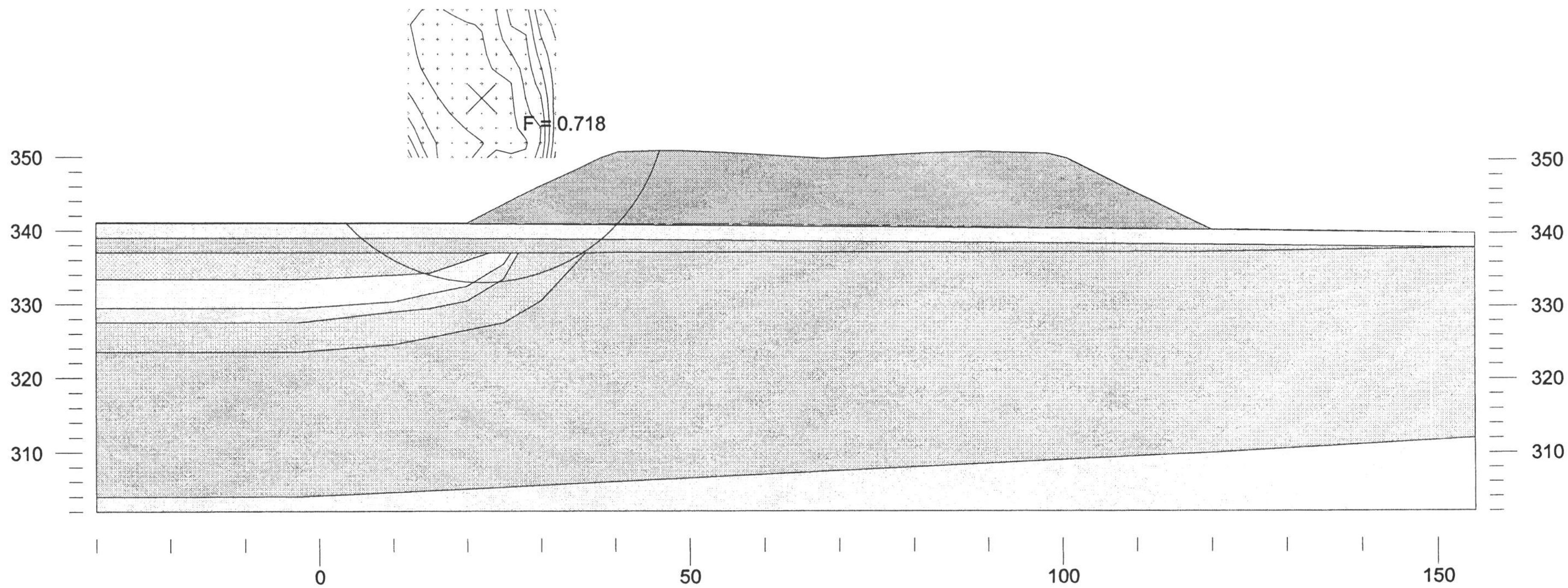


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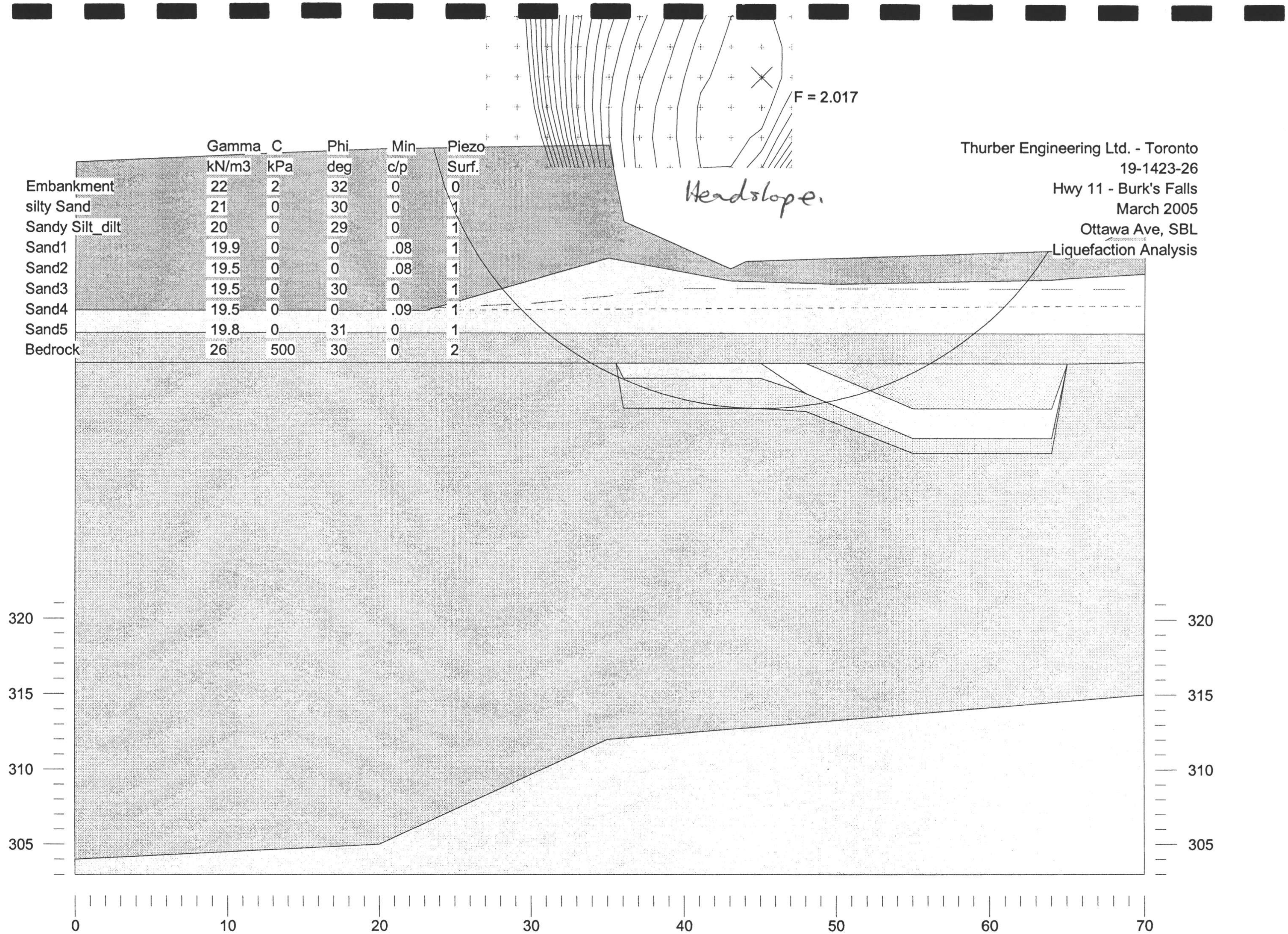
Figure A4



	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Embankment	22	2	32	0	0
silty Sand	21	0	30	0	1
Sandy Silt_dilt	20	0	29	0	1
Sand1	19.9	0	0	.08	1
Sand2	19.5	0	0	.08	1
Sand3	19.5	0	30	0	1
Sand4	19.5	0	0	.09	1
Sand5	19.8	0	31	0	1
Bedrock	26	500	30	0	2

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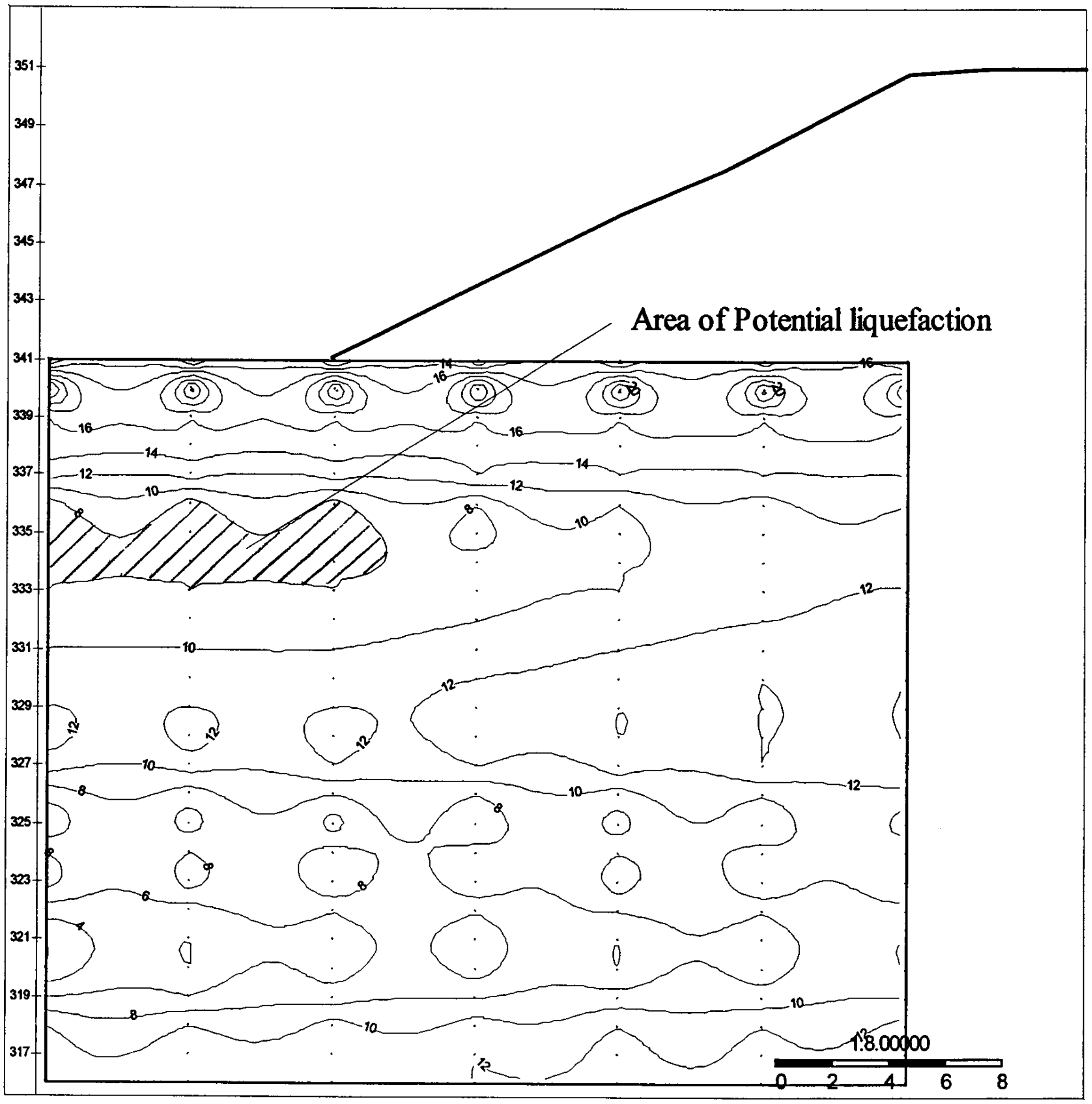
FIGURE A5



HWY 11 SBL - OTTAWA AVE

Plot of Estimated SPT After Embankment Loading

FIGURE A6

Appendix B Embankment Design Analysis (Post-liquefaction)

Figure B1	Embankment Stability with Geogrid Reinforcement
Figure B2	Embankment Stability with Berm
Figure B3	Embankment Stability with both Berm and Geogrid
Figure B4	Berm stability
Figure B5	Embankment Stability allowing berm movement

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Embankment	22	2	32	0	0
silty Sand	21	0	30	0	1
Sandy Silt_dilt	20	25	0	0	1
Sandy Silt_cont	19.9	0	0	.07	1
Sand1	19.5	0	0	.08	1
Sand2	19.5	0	0	.09	1
Sand3	19.5	0	0	.1	1
Sand4	19.8	0	31	0	1
Bedrock	26	500	30	0	2

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Liquefaction Analysis

Potential Slope instability associated with liquefaction event

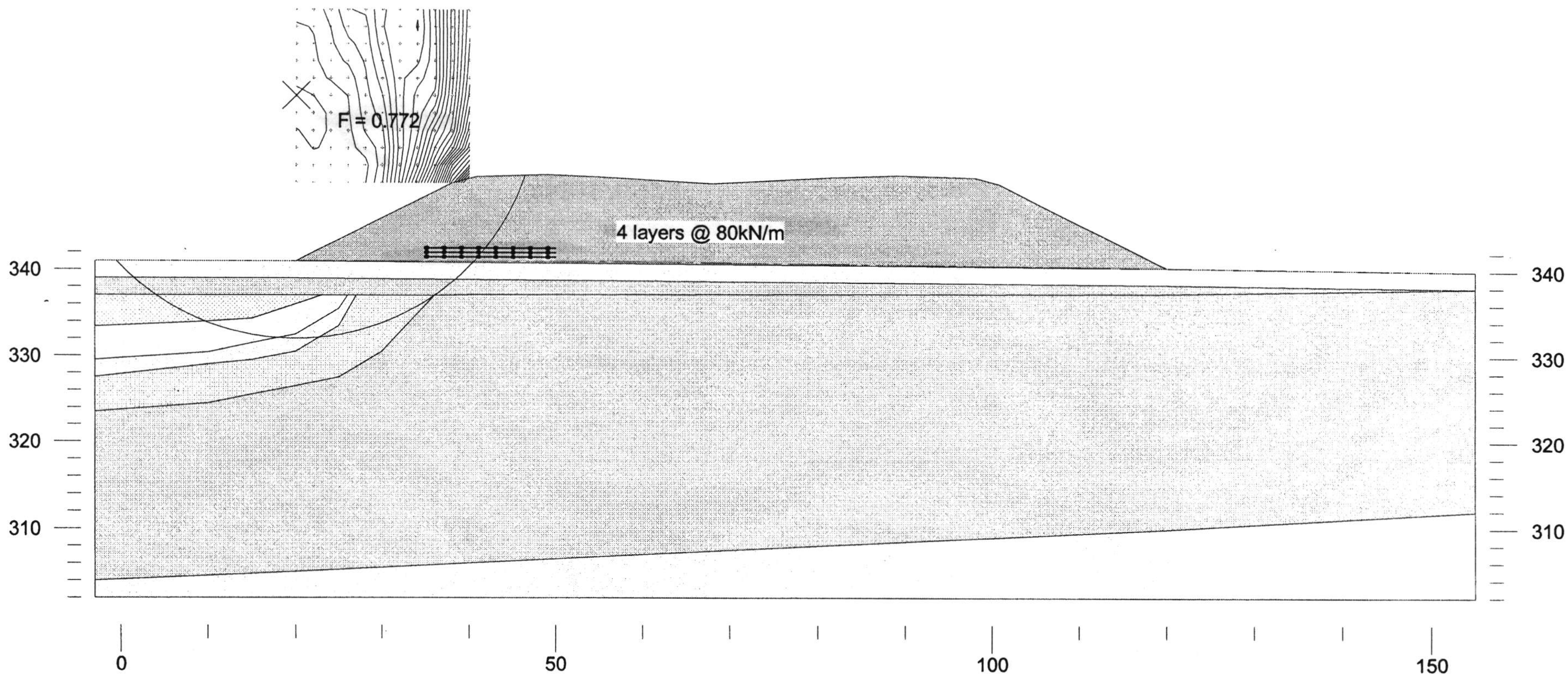

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FIGURE B1

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Embankment	22	2	32	0	0
silty Sand	21	0	30	0	1
Sandy Silt_dilt	20	25	0	0	1
Sandy Silt_cont	19.9	0	0	.07	1
Sand1	19.5	0	0	.08	1
Sand2	19.5	0	0	.09	1
Sand3	19.5	0	0	.1	1
Sand4	19.8	0	31	0	1
Bedrock	26	500	30	0	2

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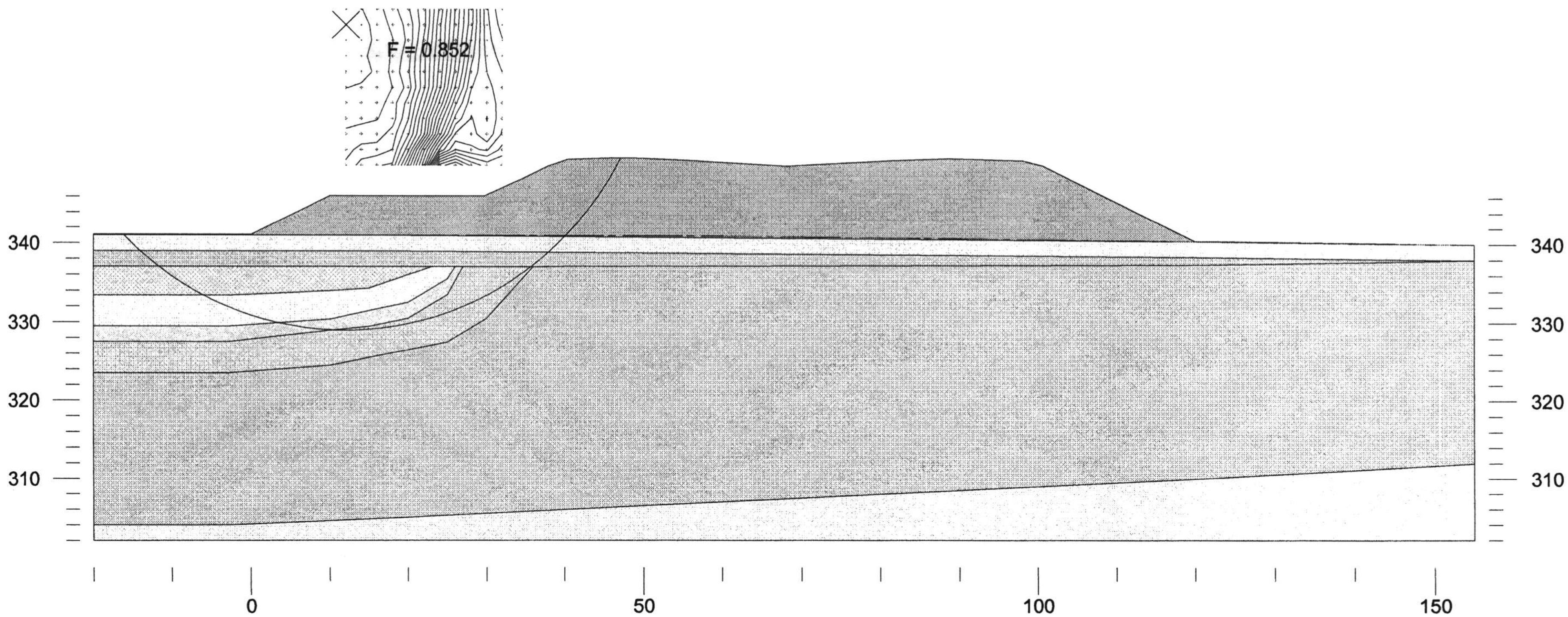

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FIGURE B2

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Embankment	22	2	32	0	0
silty Sand	21	0	30	0	1
Sandy Silt_dilt	20	25	0	0	1
Sandy Silt_cont	19.9	0	0	.07	1
Sand1	19.5	0	0	.08	1
Sand2	19.5	0	0	.09	1
Sand3	19.5	0	0	.1	1
Sand4	19.8	0	31	0	1
Bedrock	26	500	30	0	2

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Potential Slope instability associated with liquefaction event

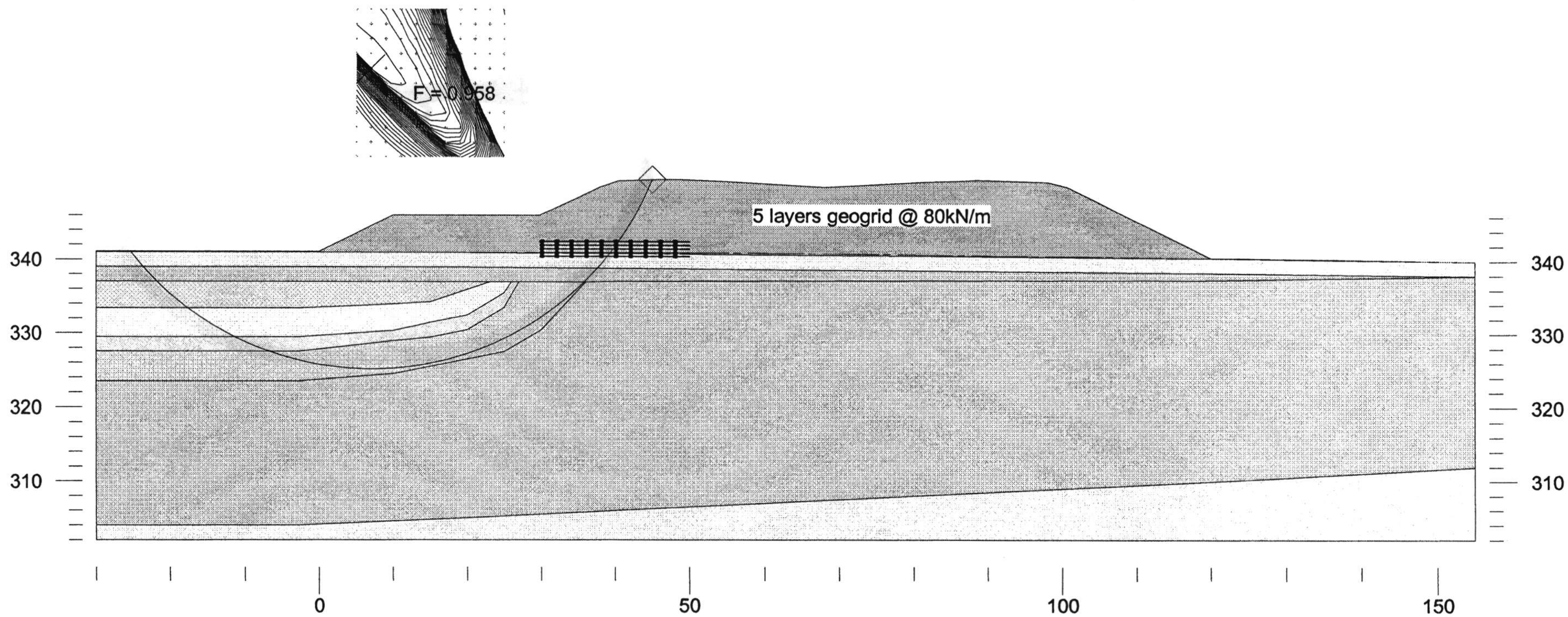

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FIGURE B3

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Embankment	22	2	32	0	0
silty Sand	21	0	30	0	1
Sandy Silt_dilt	20	25	0	0	1
Sandy Silt_cont	19.9	0	0	.07	1
Sand1	19.5	0	0	.08	1
Sand2	19.5	0	0	.09	1
Sand3	19.5	0	0	.1	1
Sand4	19.8	0	31	0	1
Bedrock	26	500	30	0	2

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Liquefaction Analysis

Potential Slope instability associated with liquefaction event

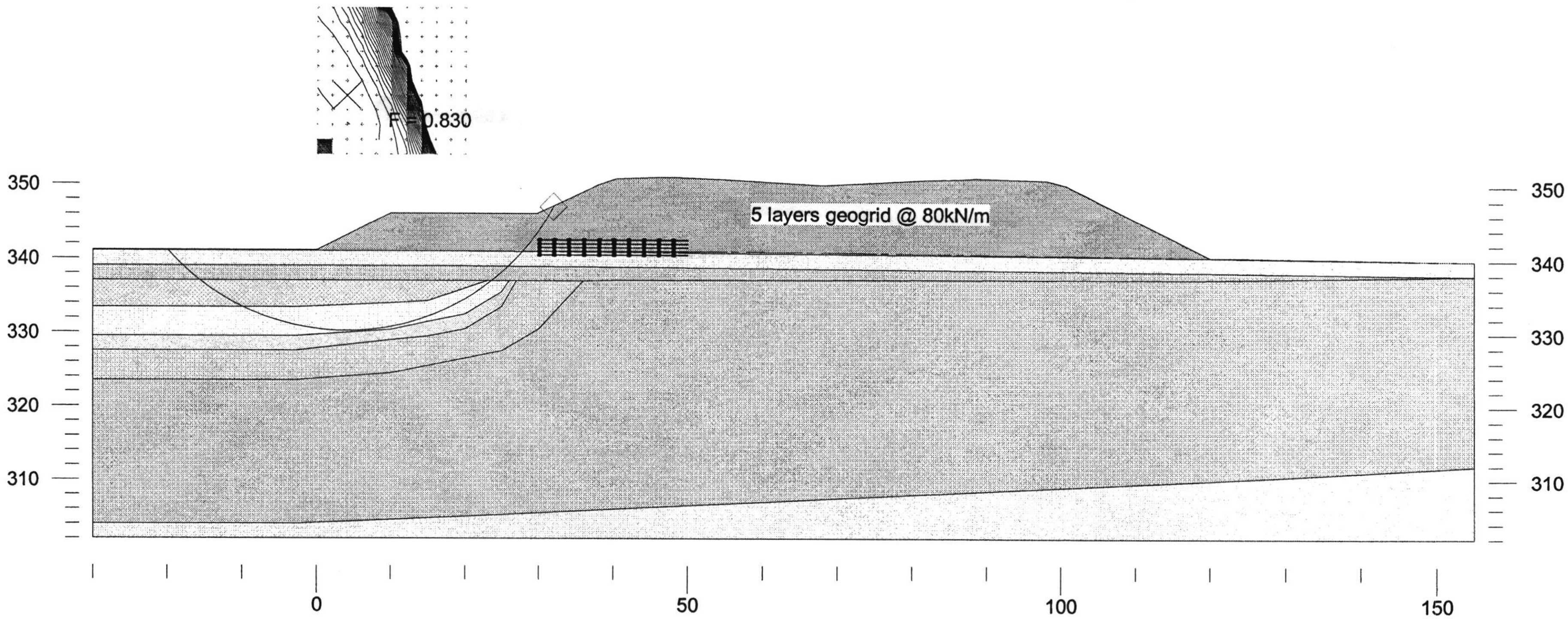

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FIGURE B4

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Embankment	22	2	32	0	0
silty Sand	21	0	30	0	1
Sandy Silt_dilt	20	25	0	0	1
Sandy Silt_cont	19.9	0	0	.07	1
Sand1	19.5	0	0	.08	1
Sand2	19.5	0	0	.09	1
Sand3	19.5	0	0	.1	1
Sand4	19.8	0	31	0	1
Bedrock	26	500	30	0	2

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Potential Slope instability associated with liquefaction event

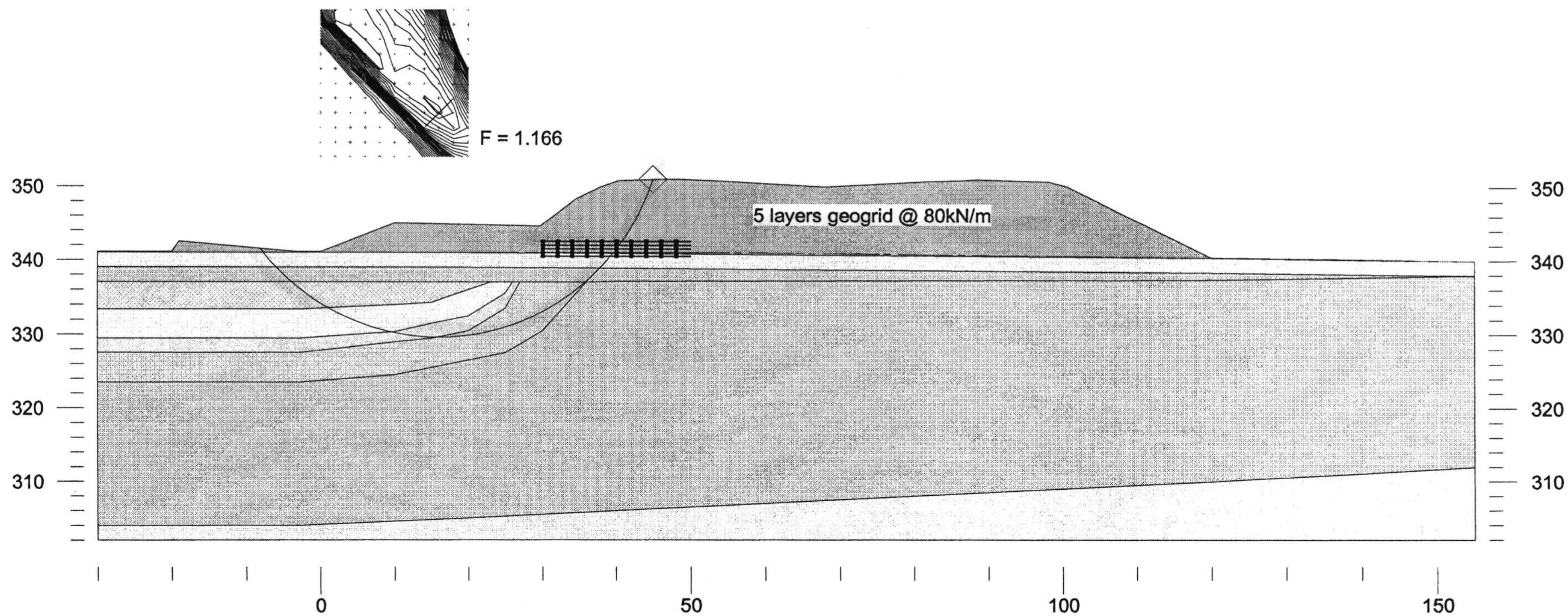

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FIGURE B5