

Geocres No:  
31E-244

**DRAFT**  
**LIQUEFACTION ASSESSMENT**  
**SOUTH RIVER, SBL**  
**HIGHWAY 11 FOUR LANING**  
**BURK'S FALLS TO SOUTH RIVER, ONTARIO**  
**G.W.P. 759-93-00**

Geocres Number:

31E-244

**Report to**  
**Marshall Macklin Monaghan**

Thurber Engineering Ltd.  
2010 Winston Park Drive, Suite 103  
Oakville, Ontario  
L6H 5R7  
Phone: (905) 829 8666  
Fax: (905) 829 1166

January 4, 2006  
File: 19-1423-28

SMS/ C:\DATA\PROJECT FILES\19-3745-0Hwy17\Bonnechere.DRAFT.doc

## TABLE OF CONTENTS

|       |  |   |
|-------|--|---|
| 1     | GENERAL.....                                 | 1 |
| 2     | ENGINEERING ANALYSIS METHODOLOGY.....        | 2 |
| 2.1   | General .....                                | 2 |
| 2.2   | Screening For Liquefaction Potential.....    | 2 |
| 2.3   | Seismic Loading.....                         | 3 |
| 2.4   | Evaluation of Liquefaction Resistance.....   | 4 |
| 2.5   | Embankment Construction .....                | 5 |
| 2.6   | Post-Liquefaction Behaviour.....             | 5 |
| 2.6.1 | Stability .....                              | 5 |
| 2.6.2 | Settlement.....                              | 5 |
| 2.6.3 | Lateral Spreading.....                       | 5 |
| 3     | EMBANKMENT DESIGN .....                      | 6 |
| 3.1   | General .....                                | 6 |
| 3.2   | Liquefaction Potential .....                 | 6 |
| 3.3   | EMBANKMENT RESPONSE.....                     | 7 |
| 3.3.1 | Embankment Stability .....                   | 7 |
| 3.3.2 | Embankment Foundation Settlement.....        | 7 |
| 3.3.3 | Lateral Spreading.....                       | 8 |
| 3.4   | DESIGN OPTIONS .....                         | 8 |
| 3.4.1 | Design Foundation to Accommodate Loads ..... | 8 |
| 3.4.2 | Install Supplementary Pile System .....      | 9 |
| 3.4.3 | Foundation Treatment .....                   | 9 |
| 3.4.4 | Comparison of Options.....                   | 9 |
| 4     | CLOSURE .....                                | 9 |

## References

### Appendix A Liquefaction Analysis Data

- Figure A1 Plot of CRR versus Depth at boreholes – South Approach
- Figure A2 Plot of CRR versus Depth at boreholes – North Approach
- Figure A3 Stratigraphic Information & SPT results – South Approach
- Figure A4 Stratigraphic Information & SPT results – Hwy 11, SBL Bridge

### Appendix B Seismic Design Assessment

- Figure B1 Embankment Stability Analysis – South Approach
- Figure B2 Embankment Stability Analysis – Central Bridge Pier
- Figure B3 Analysis of Lateral Spreading

DRAFT COPY

**DRAFT**  
**LIQUEFACTION ASSESSMENT**  
**SOUTH RIVER, SBL**  
**HIGHWAY 11 FOUR LANING**  
**BURK'S FALLS TO SOUTH RIVER, ONTARIO**  
**G.W.P. 759-93-00**

**Geocres Number:**

**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 structure carrying the HWY 11 south bound lanes, SBL, over the South River. This assessment has been updated based on the design alignment provided by MMM dated November 22, 2005.

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  
South River Bridge, SBL  
Highway 11 Burk's Falls to south River  
GWP 759-93-00, W.P. 5037-03-01, Site 44-373

The surface topography at the proposed South River, SBL bridge is comprised of the South River which occupies a 25 m wide meandering thalweg and has a bank height of about 1.5 to 2 m. South of the river channel, a level flood plain is present at approximately Elevation 329 to 331 m. North of the river, the terrain rises to approximately Elevation 340 m at the north bridge abutment location.

The stratigraphy encountered in the boreholes near the South River, SBL structure is described in detail in the above noted report, and generally consists of very loose to compact layers of sand, sandy silt and silty sand extending to 28 m depth below ground surface near the south abutment

location. The sand and silty sand deposits overlie a dense to very dense deposit of sandy gravel with cobbles and boulders which overlays sloping bedrock at the north abutment. At the south abutment, the investigation extended 4.6 to 7.6 m into the gravelly sand before meeting refusal conditions.

## 2 ENGINEERING ANALYSIS METHODOLOGY

### 2.1 General

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

- 2.2
- Screening of subsurface information to delineate strata that is considered susceptible to liquefaction and requires detailed analysis.
  - Estimate of 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 SPT 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.

## 2.3 Seismic Loading

The following seismic parameters have been used in the liquefaction assessment

- |                     |                                     |      |
|---------------------|-------------------------------------|------|
| <i>Velocity</i>     | ▪ Velocity Related Seismic Zone     | 1    |
|                     | ▪ Zonal Velocity Ratio              | 0.05 |
| <i>Acceleration</i> | ▪ 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. By convention, this uniform CSR is scaled to a standardized duration of shaking representing a standard magnitude 7.5 earthquake to allow comparison between different seismic events. The cyclic stress ratio is calculated using the following formula:

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

*more detail  
parameters*

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 mass and varies with the soil type

and 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 corrected 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 of the corrected field data with an envelope that has been derived from a database 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 and are not included in the database of case histories. Correction for overburden stresses are therefore typically not carried out as part of conventional liquefaction assessment at depths of less than 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 corrected SPT N-value for each 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 Corp.

### **2.6.2 Settlement**

Liquefaction of sediments will result in the densification of both saturated and unsaturated materials. Post-liquefaction 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).



### 3 EMBANKMENT DESIGN

#### 3.1 General

The proposed south approach embankment for SBL of Highway 11 mainline crosses a floodplain and will vary in height from approximately 14.9 m near Sta. 14+900 to a maximum height of 15.7 m near Sta. 14+950. At the south bridge abutment, the proposed height of fill is 15.6 m. The height of fill at the north bridge abutment is 10.0 m and the height of the approach embankment decreases to 3 m about 70 m north of the abutment.

The soil conditions encountered within the floodplain area south of the river consist of surficial peat or topsoil overlying very loose to loose sand. The sand deposit is in turn underlain at some borehole locations by a deposit of silty sand to sandy silt. A dense to very dense gravelly sand with cobbles and boulders was encountered beneath the upper deposits at the boreholes advanced for the bridge foundations. The water-table within the floodplain area is approximately at the existing ground surface.

about  
north  
of  
stream

#### 3.2 Liquefaction Potential

The screening for liquefaction potential of the deposits relative to the Chinese Criteria indicates that the sandy silt to silt is considered non-plastic. This stratum was therefore included in the more detailed liquefaction 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 the entire soil profile on both the north and south sides of the river. The calculated CRR values with depth and the liquefaction envelope for each borehole are summarized in Figure A1 and Figure A2 in Appendix A. The locations of the areas susceptible to liquefaction are shown in Figures A3 and A4 in Appendix A. The results show several zones that are considered susceptible to liquefaction, in particular zones adjacent to the river such as near the toe of the SBL south approach embankment and a zone at 3 to 5 m depth near the central pier of the SBL bridge.

Based on the proposed embankment geometry and the subsurface conditions encountered in the boreholes, the south approach embankment is considered the most susceptible to liquefaction induced instability or settlements. A critical section near Sta. 14+935 was therefore selected for detailed stability analysis using foundation shear strength adjusted for embankment loads and short term (undrained) post-liquefaction conditions.

Sporadic zones with low CRR values were encountered in the boreholes drilled within the footprint of the proposed south approach embankment. However, the expected increase in effective stress and relative density of the foundation associated with construction of the

proposed 16 m high embankment indicate that the foundation of the embankment will not be susceptible to liquefaction.

No significant zones that may be susceptible to liquefaction were noted on the north side of the river.

### 3.3 EMBANKMENT RESPONSE

#### 3.3.1 Embankment Stability

##### *Embankment Sideslopes*

The stability analysis for embankment sideslopes indicate a Factor of Safety,  $F = 1.2$  for the south approach embankment considering reduced shear strength for liquefied foundation soils near the toe of the embankment. The results of the embankment stability analysis are shown in Figure B1 in Appendix B.

##### *Embankment Headslope*

Review of the soil conditions at the proposed headslope location indicates that liquefaction of foundation soils is not likely to occur. Stability analysis of the headslopes for post-liquefaction conditions was therefore not carried out.

##### *Hwy 11, SBL Bridge - Central Pier*

A stability analysis was carried out for the terrain adjacent to South River at the location of the proposed central bridge pier for Highway 11, SBL, where a zone with potential for liquefaction was encountered at 3 to 5 m depth below the existing ground surface. The results of the analysis are summarized in Figure B2 in Appendix B. The analysis indicates that  $F < 1$  for the terrain extending about 20 m inland of the river bank following the design seismic event. Drainage of excess pore pressures generated during the earthquake is expected to take up to several hours, so that the deformation associated with the instability may be in the up to several metres.

#### 3.3.2 Embankment Foundation Settlement

There is also 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%. The settlements are expected to influence the ground surface above zones where liquefaction occurs. The location of the maximum settlement is therefore predicted to be outside of the toe of the south approach embankment. No significant settlement is expected beneath the central portion of the embankment.

Settlement will also occur near the central pier location of the South River bridge, SBL. where liquefaction susceptible soils have been identified at 3 to 5 m depth below ground

surface. The maximum calculated surface settlement for a 2 m thick liquefied layer at 2 m depth is about 100 mm. This will result in an additional negative skin friction load to the upper portion of the piles supporting the central pier. The estimated vertical load is 15 kPa, applied over the perimeter of each pile in the upper 3 m.

### 3.3.3 Lateral Spreading

The results of the analysis for lateral spreading of the floodplain adjacent to the South River indicate a horizontal displacement of about 140 mm at the edge of the river bank decreasing to 60 mm at a distance of 20 m from the free-face (river bank). The results of the analysis are summarized in Figure B3 in Appendix B.

The proximity of the central pier to the river bank indicates that there is potential for the pier foundation to be subject to lateral loads following the design seismic event. The abutment foundations are set-back a sufficient distance from the river bank, such that no significant lateral movements at the abutments are expected.

## 3.4 DESIGN OPTIONS

Based on the analyses described above, it is expected that the approach embankments will not be subject to instability or excess settlement following the design earthquake. Additional seismic design measures are therefore not required for the approach embankments.

The central pier of the Highway 11 SBL bridge over South River may be impacted by slope instability associated with liquefaction of a zone at 3 to 5 m depth below ground surface. The analysis for lateral spreading indicates lateral movements of 60 mm to 140 mm towards the river in the vicinity of the proposed pier location.

The following design options were considered to address the potential instability associated with liquefaction at the central pier location:

- Design foundation of central pier to withstand the lateral loads
- Install supplementary piles to withstand the lateral loads
- Foundation Treatment

### 3.4.1 Design Foundation to Accommodate Loads

Based on the unbalanced lateral force calculated from the stability analysis and the ultimate passive soil resistance, a lateral soil load of approximately 100 kN/m (ULS, unfactored) will be applied by the soil mass moving against the pier post-liquefaction conditions. This force will be applied against the pier foundation at between 1 to 4 m depth below ground

surface. The projected width of the pile group that will be subjected to this load is expected to be about 10 m.

*3.4.2* Install Supplementary Pile System *provide robust*

A second design option is to support the loading associated with potential seismic related instability on a separate pile structure, such as a sheet-pile enclosure or individual steel H-piles set at regular intervals along the edge of the river bank. The pile support system could be incorporated into erosion control measures, if required to protect the pier foundation. The minimum recommended penetration for individual piles or sheet piles is 12 m (Elev 317 m). A maximum horizontal spacing of 1.5 m for individual piles installed parallel to the river bank is recommended. The structure should be designed for the loading described in Section 3.4.1.

3.4.3 Foundation Treatment

Densification of the loose zone in the vicinity of the pier foundation is an effective method to reduce the potential for liquefaction induced instability following a major earthquake. Methods of densification of saturated loose, cohesion-less soil include: vibro-compaction, dynamic compaction or installation of 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.

3.4.4 Comparison of Options

A detailed cost comparison of the options listed above is not possible without detailed structural analysis of the suitability of each option. Based on an initial assessment, modification of the existing pile group to support the lateral loading is considered the preferred option.

4 CLOSURE

The estimated liquefaction susceptibility is very sensitive to the SPT N-values recorded in the field. Conventional auger drilling methods may yield low N-values that will result in an overestimation of the amount of liquefaction. Accordingly, specific drilling and sampling methods, and the use of such as Seismic Cone Penetration Test to confirm SPT energy 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.

Depending on the estimated cost associated with design for the seismic loads described above, there may be requirement for further more detailed investigation.

Engineering analysis and report preparation by:

S.M. Sather, P.Eng.,  
Senior Geotechnical Engineer



Report reviewed by:  
P.J. BRanco, P.Eng.,  
Review Engineer



DRAFT COPY

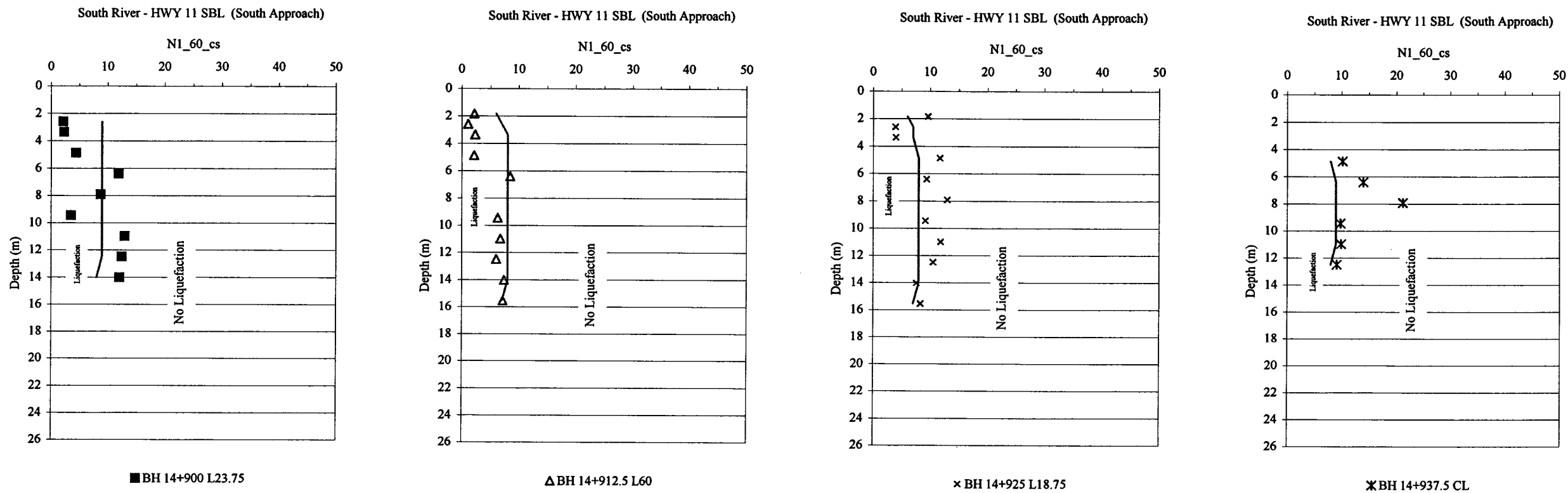
## REFERENCES

- Bartlett, S.F. and Youd, T.L. (1995). "Empirical Prediction of Liquefaction –induced lateral spread". *J. Geotech. Eng.*, 121(4), 316-329.
- Seed, H.B., and Idriss, I.M. (1971). "Simplified procedure for evaluating soil liquefaction potential". *J. Geotechnical Engrg. Div.*, ASCE, 97(9), 1249-1273.
- Seed, H.B. and Idriss, I.M. (1983). "Evaluation of Liquefaction Potential Using Field Performance Data" *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 109, No. 3 March.
- Cetin, K.O, Seed, R.B., Der Kiurghian, Tokimatsu, K., Harder Jr., L.F.Kayen, R.E., and Moss, R.E.S. (2004). "Standard penetration test-based probabilistic and deterministic assessment of seismic soil liquefaction potential" *J. Geotech. Geoenviron. Eng.*, 130(12), 1314-1340.
- 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.

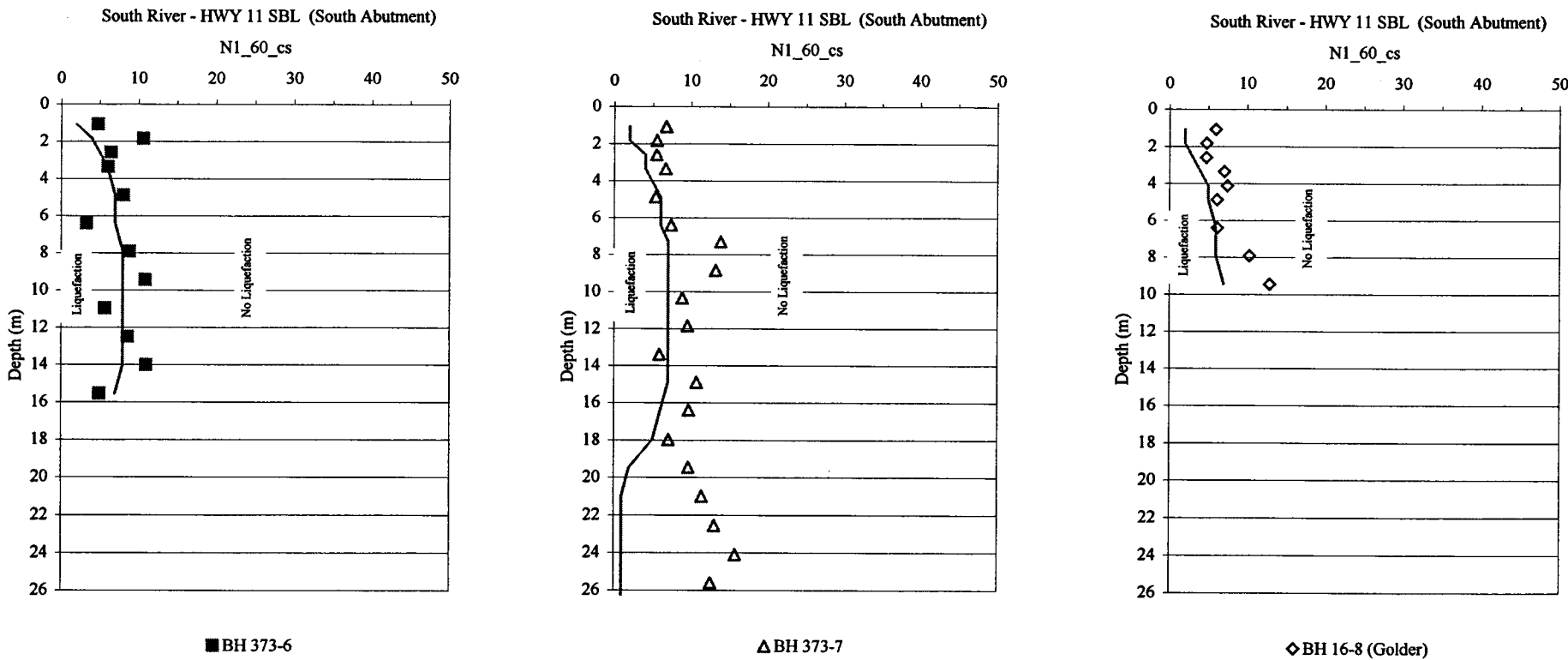
Appendix A  
Liquefaction Analysis

DRAFT COPY

HWY 11, SBL:South River Bridge - South Approach Embankment



HWY 11, SBL: South River Bridge - South Abutment

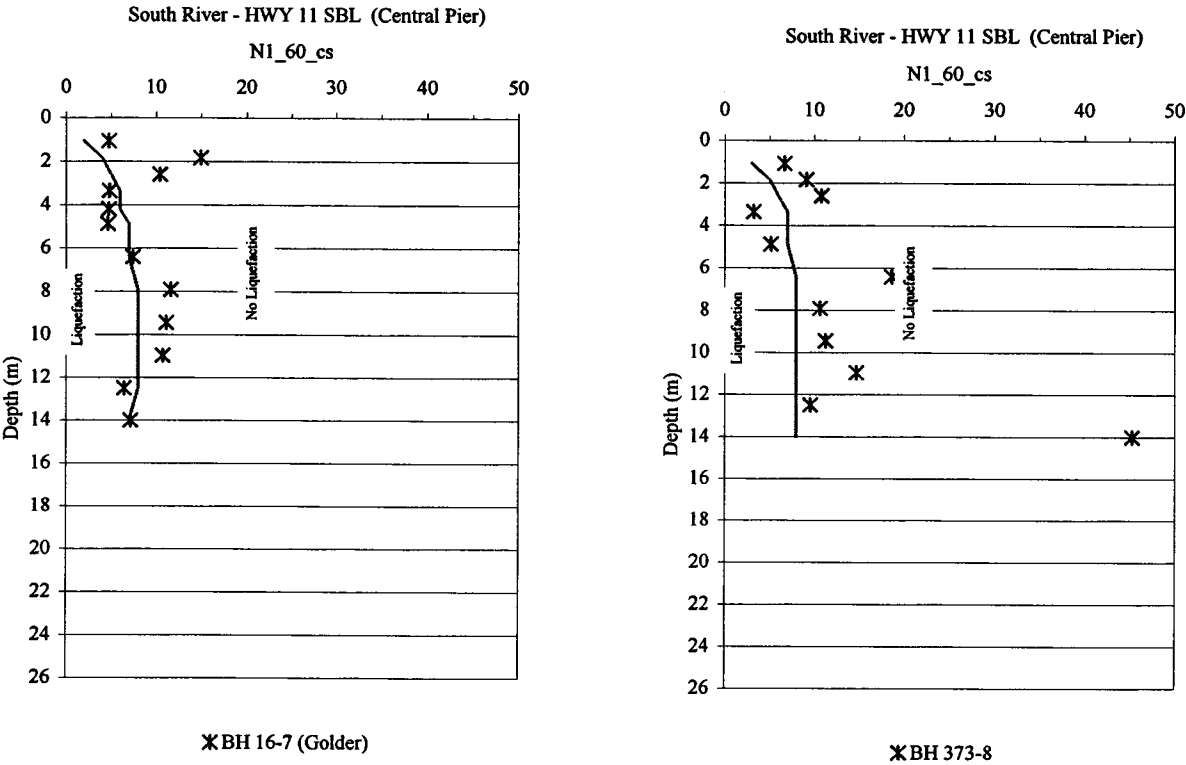


HWY 11 MAINLINE SBL  
SOUTH RIVER  
Existing Conditions

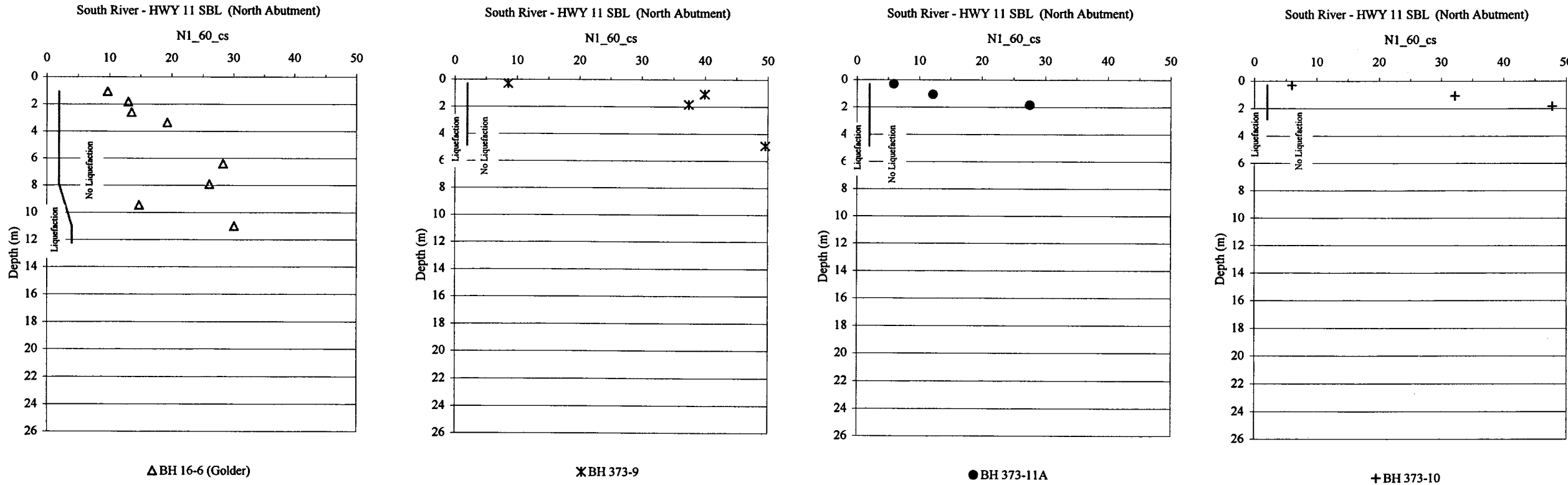
FIGURE A1



HWY 11, SBL:South River Bridge - Central Pier

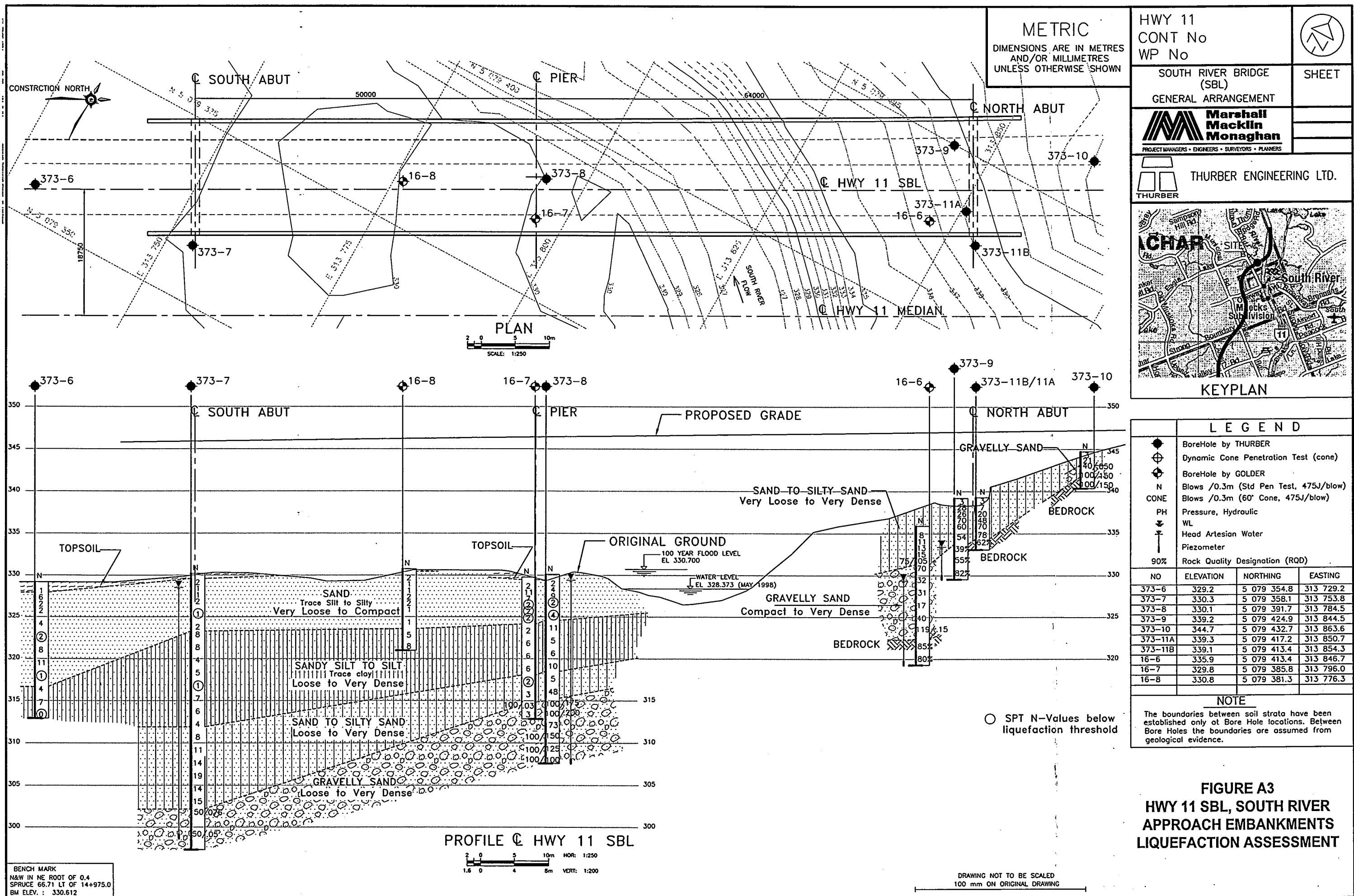


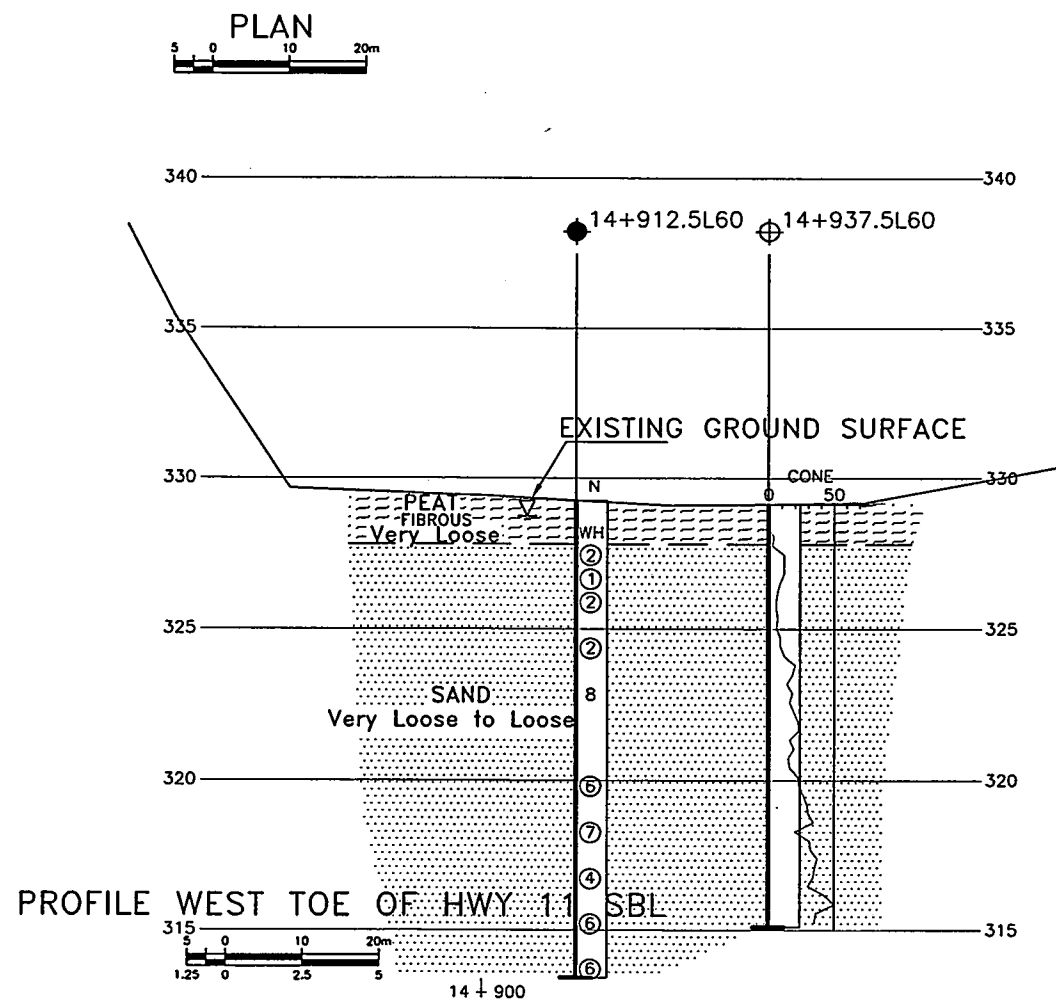
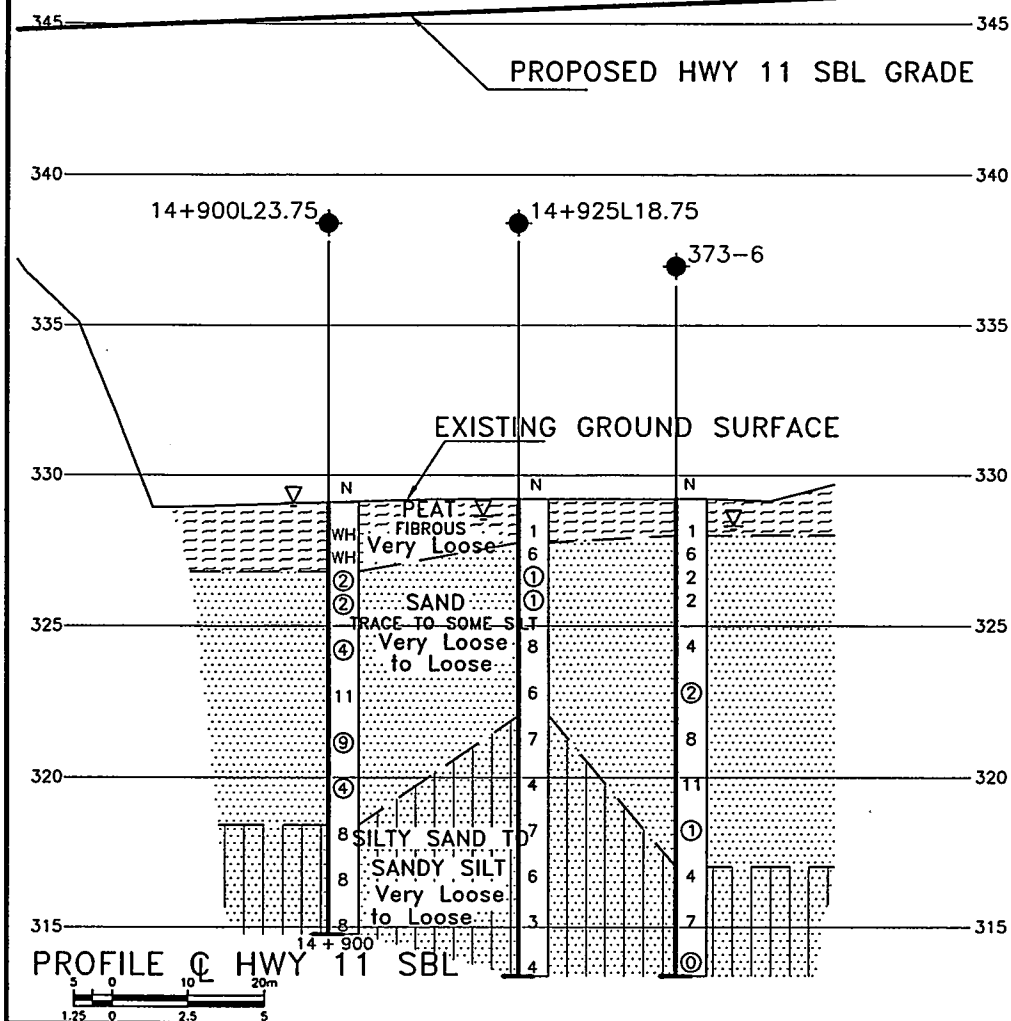
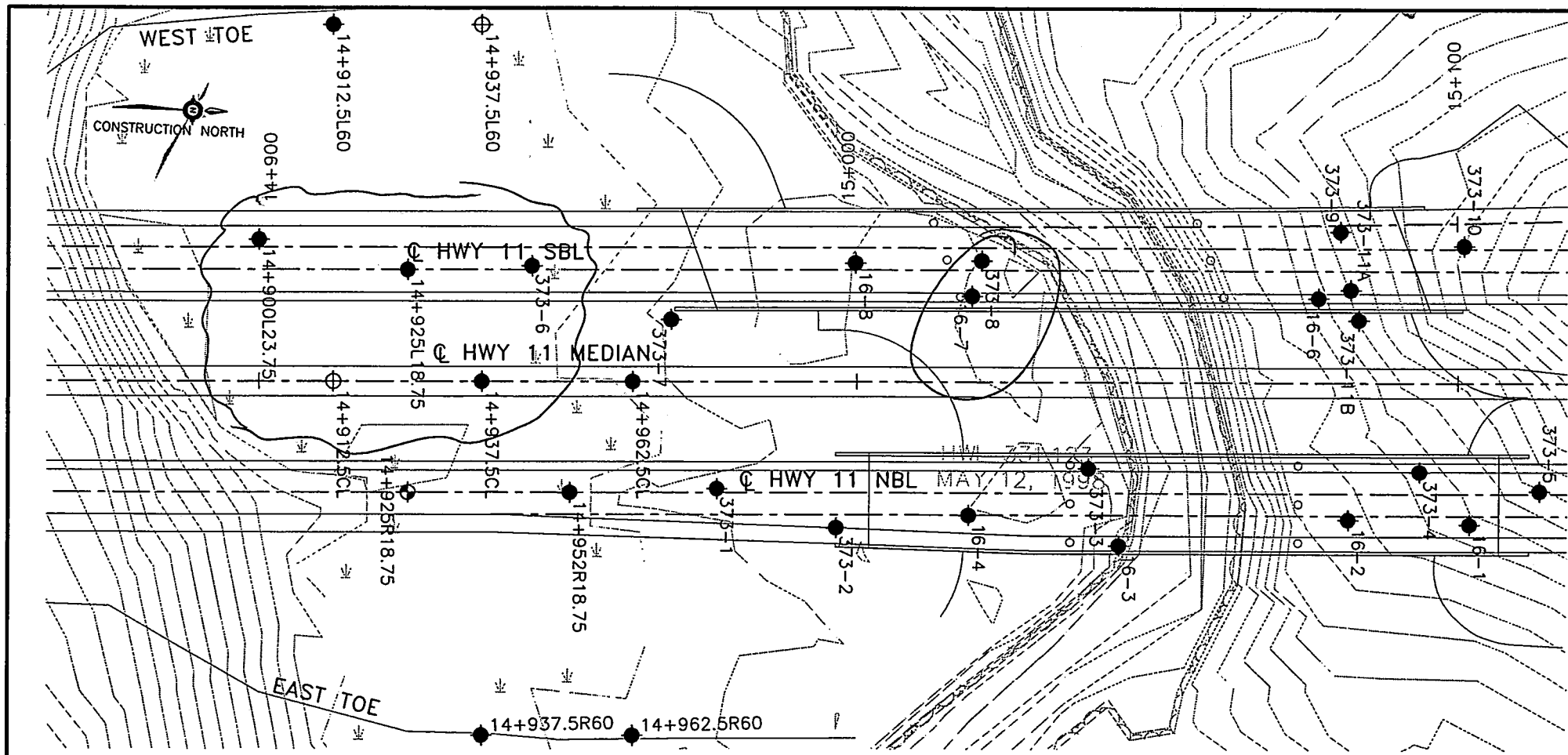
HWY 11, SBL: South River Bridge - North Abutment



HWY 11 MAINLINE SBL  
SOUTH RIVER  
Existing Conditions

FIGURE A2





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

HWY 11  
CONT No  
GWP No759-93-00

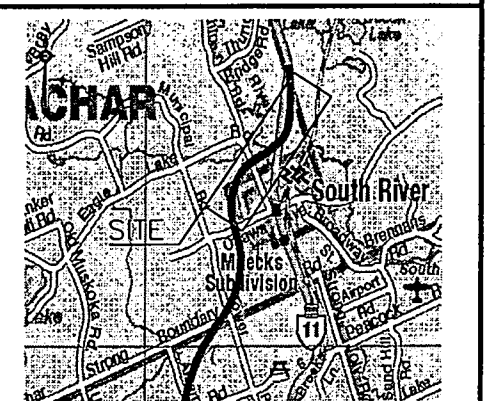


HIGHWAY 11 MAIN LINE  
MACHAR TOWNSHIP  
STATION 14+875 TO 15+120  
SBL CENTRELINE AND WEST TOE  
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



THURBER ENGINEERING LTD.  
THURBER



KEYPLAN

LEGEND

|  |  |
|--|--|
|  | Bore Hole  |
|  | Dynamic Cone Penetration Test (cone) or Probe Hole |
|  | Bore Hole & Cone                                   |
|  | Blows/0.3m (Std pen Test, 475J/blow)               |
|  | Blows/0.3m (60' Cone, 475J/blow)                   |
|  | Pressure, Hydraulic                                |
|  | WL in Piezometer at Time of Investigation (Date)   |
|  | Head Artesian Water                                |
|  | Piezometer   |
|  | WL in Open Borehole Upon Completion of Drilling    |
|  | Rock Quality Designation (RQD)                     |
|  | Auger Refusal                                      |
|  | Cone Refusal                                       |

| NO           | STATION  | OFFSET FROM MEDIAN CL |
|--------------|----------|-----------------------|
| 14+900L18.75 | 14+900   | L18.75                |
| 14+912.5CL   | 14+912.5 | CL                    |
| 14+912.5L60  | 14+912.5 | L60                   |
| 14+925L18.75 | 14+925   | L18.75                |
| 14+925R18.75 | 14+925   | R18.75                |
| 14+937.5CL   | 14+937.5 | CL                    |
| 14+937.5L60  | 14+937.5 | L60                   |
| 14+937.5R60  | 14+937.5 | R60                   |
| 14+952R18.75 | 14+952   | R18.75                |
| 14+962.5CL   | 14+962.5 | CL                    |
| 14+962.5R60  | 14+962.5 | R60                   |

NOTE

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

○ SPT N-Values below liquefaction threshold

**FIGURE A4**  
**HWY 11 SBL, SOUTH RIVER**  
**APPROACH EMBANKMENTS**  
**LIQUEFACTION ASSESSMENT**

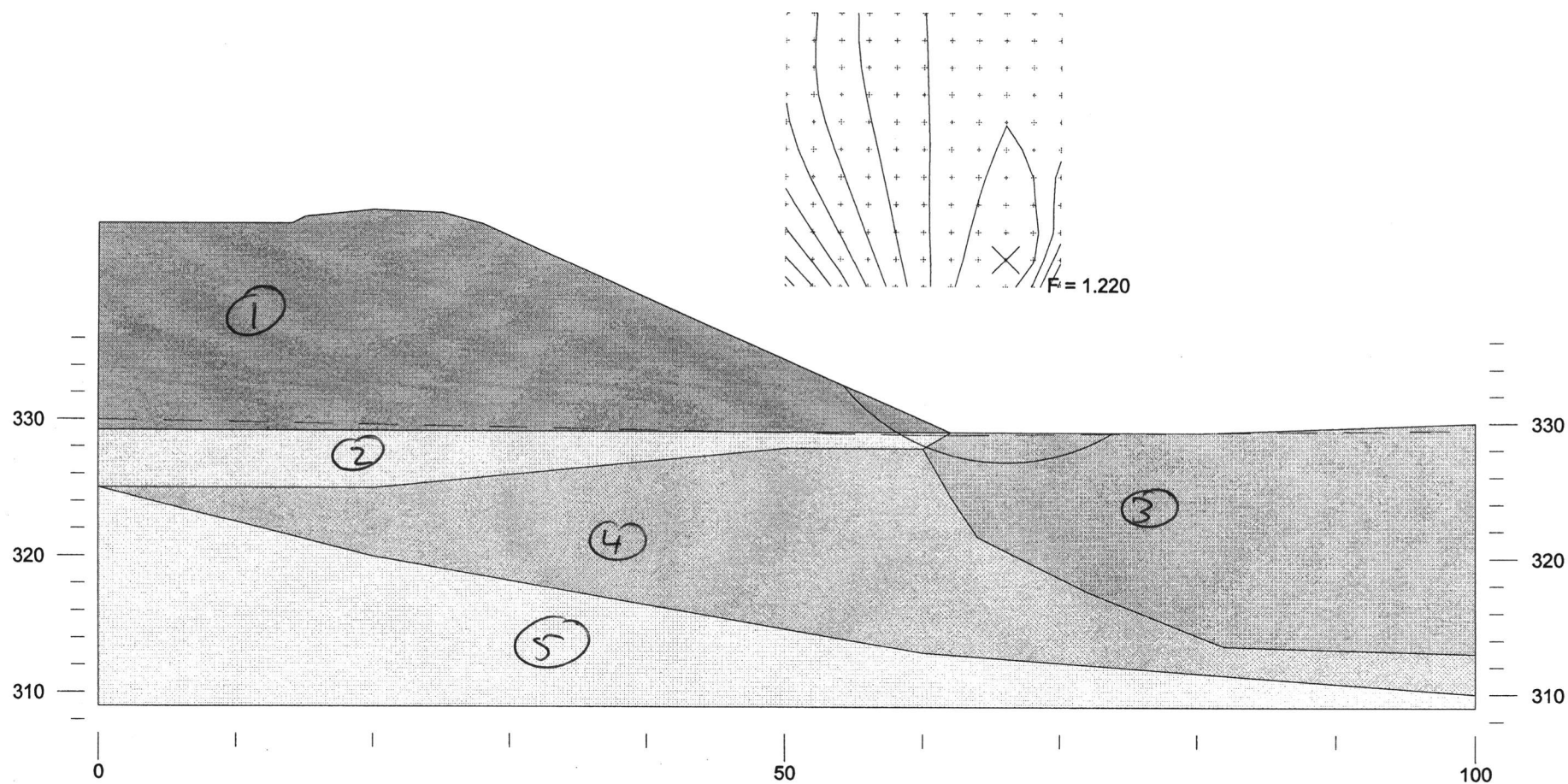
DRAWING NOT TO BE SCALED  
100 mm ON ORIGINAL DRAWING

Appendix B  
Seismic Design

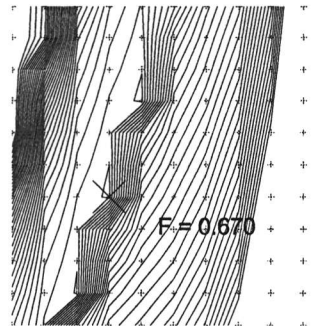
DRAFT COPY

|                      | Gamma<br>kN/m <sup>3</sup> | C<br>kPa | Phi<br>deg | Min<br>c/p | Piezo<br>Surf. |
|----------------------|----------------------------|----------|------------|------------|----------------|
| (1) Fill             | 21                         | 1        | 30         | 0          | 1              |
| (2) Subexcav backfil | 20                         | 0        | 32         | 0          | 1              |
| (3) Sand- liquefied  | 20                         | 0        | 0          | .05        | 1              |
| (4) Sand             | 20                         | 0        | 30         | 0          | 1              |
| (5) Silty Sand       | 20                         | 0        | 32         | 0          | 1              |

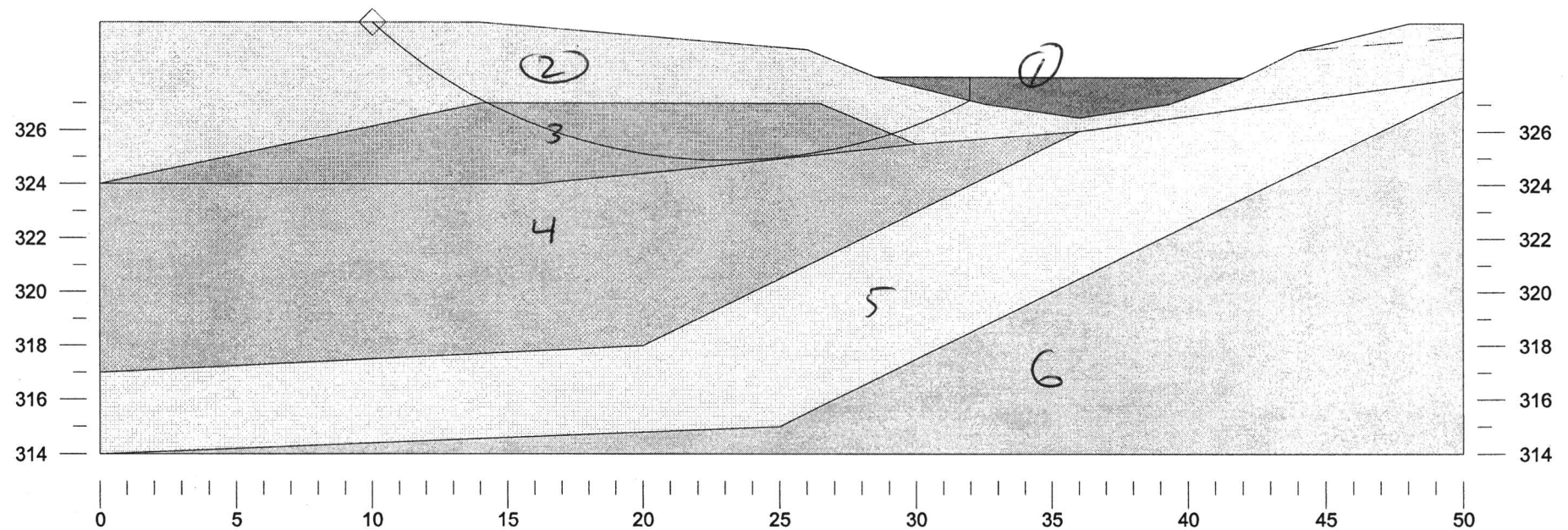
Thurber Engineering Ltd. - Toronto  
19-1423-28  
HWY 11 South River Bridge, SBL  
Dec 2005  
South Approach Embankment  
Post-Liquefaction Analysis

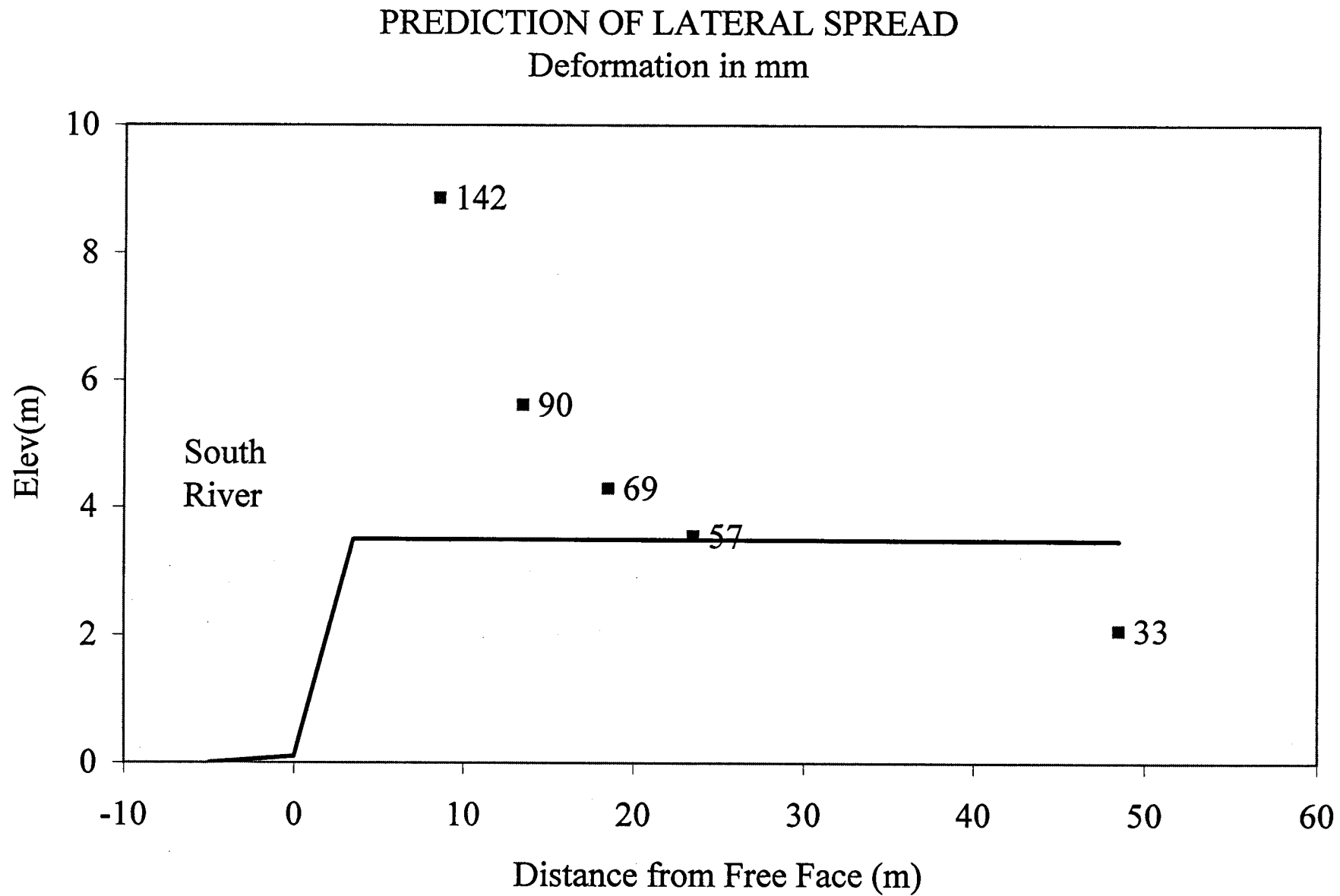


|   |                | Gamma<br>kN/m <sup>3</sup> | C<br>kPa | Phi<br>deg | Min<br>c/p | Piezo<br>Surf. |
|---|----------------|----------------------------|----------|------------|------------|----------------|
| 1 | Water          | 9.81                       | 0        | 0          | 0          | 1              |
| 2 | Upper Sand     | 20                         | 0        | 30         | 0          | 1              |
| 3 | Liquified Zone | 20                         | 0        | 0          | .05        | 1              |
| 4 | Sandy Silt     | 20                         | 0        | 32         | 0          | 1              |
| 5 | Silty Sand     | 20                         | 0        | 33         | 0          | 1              |
| 6 | Gravelly Sand  | 21                         | 0        | 35         | 0          | 1              |



Thurber Engineering Ltd. - Toronto  
 19-1423-28  
 HWY 11 South River Bridge, SBL  
 Dec 2005  
 Central Pier  
 Post-Liquefaction Analysis





**Highway 11, SBL**  
**South River**  
Seismic Assessment

**FIGURE B3**