FOUNDATION INVESTIGATION AND DESIGN REPORT
GABION WALLS AND HIGH FILL
NEW UPPER ACCESS ROAD
HIGHWAY 17/417 TWINNING
ARNPRIOR, ONTARIO
G.W.P. 647-92-00, SITE NO. 29-201/2
GEOCRES Number: 31F-154

Report to

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation carried out along the alignment of the New Upper Access Road. The new alignment will follow a winding pattern to connect the Upper Access Road at the Arnprior Dam with the Lower Access Road, at a location to the north of the existing Highway 17 Madawaska River bridge East Approach. Previous investigations had been carried out by, or under the direction of, the Ministry of Transportation Ontario (MTO) for the culvert that carries the existing Upper Access Road under Highway 17, and for the East Abutment of the existing Madawaska River bridge. Factual data from those investigations has been used as reference during the preparation of this report.

The purpose of this investigation was to determine the subsurface conditions at locations close to the proposed gabion wall alignments and high fill embankment footprint, to provide a borehole location plan and soil strata drawing with a stratigraphic profile and cross-sections, records of boreholes, laboratory test results and a written description of the subsurface conditions. A model of the subsurface conditions was developed primarily based on the data obtained from this investigation, and with reference to the data obtained from the previous MTO investigation.

Thurber carried out this investigation as a sub-consultant to McCormick Rankin Corporation (MRC), under the MTO Purchase Order Number 4005-A-000349.

The following documents were referenced during the preparation of this report:

- MTO Report titled "Foundation Investigation and Design Report, Hwy. 17N, Hydro Dam Upper Access Road Overpass", W.P. 78-76-01, Dist.9, GEOCRES No. 31F-93, dated Nov.23, 1976 (Reference 1).
- MTO report titled "Foundation Investigation and Design Report, Madawaska River Bridge (Arnprior Diversion)", Hwy. 17N, Dist. 9, W.P. 198-62-00, Str. Site 29-191, GEOCRES No. 31F-94, dated July 19th, 1977 (Reference 2).
- Thurber report titled "Foundation Investigation and Design Report, Madawaska River Bridge, Highway 17 Twinning, G.W.P. 647-92-00, Site No. 29-191/1, GEOCRES No. 31F-130, dated August 13, 2004 (Reference 3).

2 SITE DESCRIPTION

The site is located adjacent to the existing Highway 17 Madawaska River bridge east approach and the Arnprior Generating Station, just south of the Town of Arnprior, Township of McNab, County of Renfrew. The Arnprior Generating Station and associated dam are situated on the south side of the highway at this location. The general site location is shown on the Borehole Locations and Soil Strata drawing in Appendix E. The existing Upper Access Road crosses under the existing Highway 17 inside a culvert and provides access to the crest of the dam. The existing Lower Access Road runs along the east bank of the Madawaska River and provides access to the station buildings.

The site is located in an area characterized by bedrock outcrops and shallow glacial drift. The ground is typically sloping in a westerly direction towards the Madawaska River. Vegetation is light around the site area and mainly consists of grass and some small trees. The south-facing slope of the east approach runs perpendicular to the two access roads. Drainage in the area is largely governed by the nearby Madawaska River and the dam.

The project area is located within a physiographic region known as the Ottawa Valley Clay Plains. This area is located between the Laurentian upland to the north and west, and the Ottawa lowland to the south and east. Native soil deposits typically consist of glacio-lacustrine clayey silts to silty clays that were deposited when the Champlain Sea inundated the Ottawa – St. Lawrence lowland. In Renfrew County, there are prominent east-west trending scarps (fault zones), including a major depression geologically known as the "Ottawa-Bonnechere" graben. Bedrock in the site area consists of crystalline limestone of the Ordovician Age that has been subjected to faulting, weathering and erosion.

3 SITE INVESTIGATION AND FIELD TESTING

The site investigation program was carried out on December 12, 13 and 15, 2006 and on January 24, 2006. The borehole investigation program consisted of seven (7) boreholes, numbered UAR 05-1 to UAR 05-7 (including 05-6A which was augered without sampling in close proximity to 05-6 to about 3 m depth before a Shelby tube sample was taken. Boreholes UAR 05-1, 05-2, 05-3 and 05-4 were located close to the two proposed gabion wall alignments. Boreholes UAR 05-5 to UAR 05-7 were drilled along the south toe of the existing Madawaska River Bridge east approach embankment. The depths of termination of the boreholes range from 2.6 m to 19.3 m below existing ground surface. The approximate locations of all relevant boreholes are shown on the attached Borehole Locations and Soil Strata drawing in Appendix E.

The borehole locations were initially marked and/or staked in the field by surveyors from J.D. Barnes Ltd (Ottawa) who also provided the survey data to Thurber. Several of the boreholes were relocated from the surveyed locations to avoid interference with high tensioned overhead cables or working on sloping ground. Clearance of buried utilities at the borehole locations was obtained by Thurber prior to any drilling being carried out.

The drilling, sampling and in-situ testing operation was carried out using truck mounted CME 55 or CME 75 drill rigs that were supplied and operated by George Downing Estate Drilling Ltd. (Downing). Auger drilling techniques were used to advance most of the boreholes through soils and to obtain soil samples using a split spoon sampler in conjunction with the Standard Penetration Testing (SPT). Field vane shear tests were carried out within the silty clay deposit where appropriate. A continuous sample of the silty clay was recovered using a 73 mm inside diameter thin-walled Shelby tube in Borehole UAR 05-6A. Boreholes UAR 05-1 to 05-4 were further advanced some 3 m into bedrock by NQ size rotary core drilling techniques.

The drilling, coring and sampling operations were supervised on a full time basis by a member of Thurber's technical staff. The supervisor logged the boreholes, and the recovered soil and rock samples were placed in labelled containers and core boxes, and transported to Thurber's laboratory for further examination and testing.

All rock cores were logged, and the Total Core Recovery (TCR), the Rock Quality Designation (RQD) and the Fracture Indices (FI) were determined.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Standpipe piezometers were installed in Boreholes UAR 05-6 and 05-7 to permit longer term groundwater level monitoring. The installation details are presented on the Records of Boreholes in Appendix A and summarized as follows. At this site, 19 mm diameter Schedule 40 PVC pipes with 0.6 m long slotted screens were installed at the bottom of the open boreholes. The sand screens surrounding the pipe were in the order of 1 m in length. The remaining space in the borehole was grouted with a bentonite-based grout after a bentonite holeplug seal was placed on top of the sand screen.

On completion of drilling and sampling, the boreholes without piezometer installations were grouted to the ground surface using a bentonite grout.

4 LABORATORY TESTING

All recovered soil samples were subjected to visual identification and to natural moisture content determination. Selected soil samples were subjected to grain size distribution analysis and Atterberg Limits tests. The results of this testing program are shown on the Record of Borehole sheets in Appendix A and on the figures in Appendix B. The Shelby tube sample contained frequent sand and silt seams, rendering it unsuitable for laboratory oedometer testing.

Point load testing was carried out on selected rock cores from Boreholes UAR 05-1 to 05-4, and the results are presented in Table 1 attached immediately following the text.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Reference is made to the Record of Borehole sheets in Appendix A. Details of the encountered soil and rock stratigraphy are also presented on the Borehole Locations and Soil Strata drawing in Appendix E. A summary description of the stratigraphy is given in the following paragraphs. The factual information at the borehole locations governs any interpretation of site conditions.

In general, the subsurface conditions encountered in the boreholes consist of fill, topsoil and a deposit of native silty clay overlying crystalline limestone bedrock.

5.1 Fill

Fill of variable composition and thicknesses was encountered in all seven boreholes drilled during the present investigation. Sand fill with some gravel and occasional cobbles and boulders was present at ground surface in Boreholes UAR 05-1, 05-2 and 05-5. Rock fill was encountered at the ground surface in Borehole UAR 05-3, immediately below topsoil in Borehole UAR 05-7, and interlayered with topsoil in Borehole UAR 05-6. Silty clay fill was encountered at ground surface in Borehole UAR 05-4. The approximate depth of fill at each borehole location is as shown in the table below.

Borehole	Fill Thickness (m)
UAR 05-1	1.0
UAR 05-2	2.1
UAR 05-3	0.6
UAR 05-4	1.5
UAR 05-5	0.7
UAR 05-6	0.1
UAR 05-7	1.1

The sand fill was largely frozen at the time of the investigation with measured SPT 'N' values greater than 50 blows for less than 0.3 m penetration. These values do not represent the actual compactness of the sand fill. Figure B1 shows the grain size distribution of a sample of sand fill. Measured moisture contents of this fill varied between 5% and 15%. No SPT was conducted in the rock fill.

In Borehole UAR 05-4, an 'N' value of 13 blows per 0.3 m penetration was measured indicating that the silty clay fill had a stiff consistency. A measured moisture content of this fill was at 40%.

5.2 Topsoil

Topsoil was encountered at ground surface in Borehole UAR 05-7, interlayered with rock fill in Borehole UAR 05-6, and present immediately below the fill in Borehole UAR 05-4. The thickness of topsoil at each location is shown in the table below.

Borehole	Topsoil Thickness (mm)
UAR 05-4	100
UAR 05-6	50 (upper)
	75 (lower)
UAR 05-7	200

Although not encountered in the other boreholes, the presence of topsoil should be expected elsewhere along the new Upper Access Road alignment.

5.3 Silty Clay

A deposit of native, cohesive silty clay was encountered below the fill and/or topsoil in Boreholes UAR 05-2 to 05-7. The silty clay was absent in Borehole UAR 05-1. In Boreholes UAR 05-2, 05-5, 05-6 and 05-7 located near the south toe of the east approach embankment of the existing Highway 17, the thickness of this deposit ranged between 1.3m and 4.7 m. In Boreholes UAR 05-3 and 05-4 located near the east slope, along the existing Upper Access Road, the thickness of the silty clay increases to the range of 10.5 m to 14.4 m. The silty clay deposit was found extending from approximate Elevations 100.6m to 75.8 m.

Cobbles were inferred within the lower portion of the silty clay in Borehole UAR 05-2. The presence of cobbles and boulders at other locations should also be expected.

The colour of the silty clay was brown at the upper portion (crust) changing to grey with depth. The measured SPT 'N' values typically varied from 16 blows per 0.3 m penetration at the crust to the range of 2 to 4 blows per 0.3 m penetration at depth. Field vane shear strengths varying from 50 kPa to greater than 100 kPa were measured at several elevations in Boreholes UAR 05-3 and 05-4. Some of the higher values may be attributed to the presence of sand seams. The above data, in conjunction with results of pocket penetrometer testing, indicates that the silty clay is generally stiff to very stiff in consistency becoming firm with depth. Occasional 'N' values greater than 50 blows for 0.3 m penetration infer the presence of cobbles.

Figures B2 and B3 show the grain size distributions of silty clay samples. These analyses indicate that the clay content of this soil ranges between 30% and 65%. Figure B4 shows a grain size distribution of a sample of silty clay with sand seams. Figures B5 and B6 present plasticity charts to show that the silty clay samples had measured plasticity indices ranging between 18% and 34%, and a medium to high plasticity (group symbol of CI-CH). The plasticity is a function of the clay content and is anticipated to be higher as the clay content increases.

Measured moisture contents of this silty clay typically ranged from 30% to 50%.

5.4 Bedrock

Bedrock was encountered and proven by coring in Boreholes UAR 05-1, 05-2, 05-3 and 05-4. In Boreholes UAR 05-5, 05-6 and 05-7, probable bedrock was inferred by refusal to auger and/or split spoon sampler penetration. Bedrock surface depths and elevations at the borehole locations are summarized in the following table.

Borehole	Ground Surface Elevation (m)	Bedrock Surface	
		Depth (m)	Elevation (m)
UAR 05-1	82.7	1.0	81.7
UAR 05-2	82.5	6.8	75.7
UAR 05-3	98.0	11.1	86.9
UAR 05-4	102.2	16.0	86.2
UAR 05-5	82.5	3.5*	79.0*
UAR 05-6	83.6	4.3*	79.3*
UAR 05-7	86.2	2.6*	83.6*

Note: * Bedrock surface inferred from auger or spoon sampler refusal.

Based on the core samples recovered, the bedrock is described as a very thinly to thinly bedded, grey, crystalline limestone with black sub-vertical bandings. The cores are generally fresh to slightly weathered.

The measured Total Core Recovery (TCR) values for the bedrock core runs vary between 80% and 100%. The measured Rock Quality Designation (RQD) values typically range from 79% to 100% indicating good to excellent rock quality.

The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, was generally low with values of 0 and 2, except in Borehole UAR 05-3 where FI values up to 3 and 4 were measured. Sub-vertical joints were present at some locations. The condition of the joints was typically uneven and rough. Quartzite infilling was evident in Borehole UAR 05-2.

The inferred Unconfined Compressive Strengths (UCS) of intact rock cores (expressed as average value per run) typically range between 67 MPa and 98 MPa, indicating that the intact rock is generally strong. Some values of greater than 100 MPa were recorded in Boreholes UAR 05-3 and 05-4 indicating the presence of very strong to extremely strong zones within the bedrock. A summary of the Point Load Test Results is presented in Table 1 attached immediately following the text.

5.5 Water Levels

Free groundwater was not observed in any of the boreholes on completion of drilling. A standpipe piezometer was installed near the bottom of the silty clay in each of Boreholes UAR 05-6 and 05-7. On January 27, 2006, both piezometers were determined to be dry.

It should be noted that the above groundwater conditions are short term observations and the water levels are subject to seasonal fluctuations and severe climatic events. It is also anticipated that the local groundwater conditions at this site are largely governed by the nearby Madawaska River.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

6 GENERAL

This report presents interpretation of the geotechnical data in the factual report and presents foundation design recommendations to assist the design team to select and design a suitable foundation system for the two proposed gabion walls and the high fill associated with the new Upper Access Road alignment.

It is understood that the current design plan includes the construction of two gabion walls (1 and 2) and a section of high fill (≥ 6 m) associated with the new Upper Access Road alignment (see drawing in Appendix E).

Based on the available drawings provided by MRC, these walls are anticipated to be in the order of 20 m in length and 2 to 3 m in height. Along the new access road alignment, the section of high fill is in the order of 100 m in length. It is understood that the existing east approach for the Madawaska River Bridge is up to the order of 17 m in height, and consists of a rock fill shell and possibly an earth fill core. There is a mid-height berm of approximately 4 m in width, with both the upper and lower slopes at an inclination of 1.5 H : 1 V. New rock fill up to 9 m above toe level will be placed against the lower slope and beyond the existing toe to form a new road embankment with an inclination of 1.25 H : 1 V.

The discussions and recommendations presented in this report are based on our understanding of the project and on the factual data obtained during the course of this investigation.

7 FOUNDATION ALTERNATIVES

7.1 General

The current design involves a new alignment for Upper Access Road. Foundation input is required for the design of two gabion walls and one section of high fill. The approximate locations of these proposed works are as follows:

- Gabion Wall 1 - approximate Stations 45+135 to 45+155 referenced to centreline.
- Gabion Wall 2 - approximate Stations 45+310 to 45+330 referenced to centreline.
- High Fill - approximate Stations 45+180 to 45+280 referenced to centreline.

The subsurface stratigraphy encountered in the vicinity of the proposed works generally consists of fill overlying a stiff to firm silty clay deposit overlying bedrock. The top of bedrock elevations generally rise from the area of Gabion Wall 1, through the high fill section, to the area of Gabion Wall 2. The thickness of the silty clay deposit increases in the same orientation as the increase in ground surface elevations.

The elevations at which bedrock was encountered are as follows:

Location	Borehole Number	Elevations (m)	
		Existing Ground Surface	Top of Bedrock Surface
Gabion Wall 1			
Northerly Section	UAR 05-1	82.7*	81.7*
Southerly Section	UAR 05-2	82.5*	75.7*
High Fill			
Westerly Section	UAR 05-5	82.5**	79.0**
Central Section	UAR 05-6	83.6**	79.3**
Easterly Section	UAR 05-7	86.2**	83.6**
Gabion Wall 2			
Northerly Section	UAR 05-3	98.0*	86.9*
Southerly Section	UAR 05-4	102.2*	86.2*

Note : * rock cored.
** inferred from auger and/or spoon sampler refusal.

7.1 Available Alternatives

This section presents discussions on available foundation alternatives, provides recommendations and foundation design parameters for feasible and/or preferred foundation option(s) for this site.

7.1.1 Retaining Walls

During initial assessment, consideration was given to the following foundation types:

- Gabion walls
- Concrete toe walls
- Retained Soils System (RSS) walls

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

It is understood that retaining walls are required at approximate Stations 45+135 to 45+155 and Stations 45+310 to 45+330 to retain the embankment or cut slopes. Wall 1 is to support new rock fill of the proposed road embankment and Wall 2 is to support a cut into an existing slope. For these purposes, gabion walls, concrete toe walls or RSS walls may be considered for use at this site.

Based on the borehole information, the retaining wall subgrade consists predominantly of the stiff to firm silty clay. Although the majority of the foundation settlement is anticipated to consist of recompression of the over-consolidated clays, some time-dependent settlement may still occur within the "firm" zones of the clays. As such, the gabion wall option is preferable for this site, considering its inherent capability to sustain some differential settlement and from a cost effectiveness standpoint. An RSS wall is the least appropriate for this site due to potential distress along the wall panel alignments resulting from post construction differential settlements, and the proprietary construction procedures.

7.1.2 Road Embankment

As part of the new road construction, new fill will be placed against the lower, south-facing, slope of the existing Madawaska River Bridge east approach. In view of the space restriction on site and in order to maintain uniformity with the existing rock fill embankment, it is recommended that rock fill be used for the new construction.

Native foundation soils at this site consist of cohesive silty clay which will undergo settlement under the weight of the new fill. Settlement and slope stability assessments are reported later in this report.

7.2 Gabion Walls

7.2.1 General

Boreholes UAR 05-1 to UAR 05-4 drilled near the approximate alignments of the gabion walls indicate that the existing fill is variable in nature and composition, and that the native silty clay is variable in thickness. The depth to bedrock also varies across the site.

It is recommended that the gabion walls be founded on a pad of engineered fill that is itself resting on the existing fill underlain by native stiff to firm silty clay, or directly on the native silty clay. A layer of buried topsoil was encountered below the fill in Borehole UAR 05-4. Within the footprint of the pad, consideration should be given to sub-excavating, removing the topsoil and backfilling prior to constructing the pad. The engineered fill pad is required to provide subgrade uniformity along the gabion wall alignments. This pad should consist of compacted Granular A materials and have a minimum thickness of 0.5 m. Local sub-excavation may be required to accommodate the design grades or to remove unsuitable subgrade materials (section 7.2.4).

Stepped footings may be required at some locations, and should be designed in accordance with the requirements in the Canadian Highway Bridge Design Code (CHBDC, 2000).

Based on the existing borehole information, the gabion walls should be founded at the following elevations:

- Gabion Wall 1 – approximate Elevation 82 m (or below).

- Gabion Wall 2 – from approximate Elevation 97.5 m (or below) at the northerly section to approximate Elevation 100.5 m (or below) at the southerly section (after the topsoil is removed).

7.2.2 Bearing Resistance

It is recommended that gabion walls founded on an engineered granular pad resting on the stiff silty clay at this site may be designed for a factored geotechnical resistance of 250 kPa at Ultimate Limit States (ULS) and a geotechnical resistance at Serviceability Limit State (SLS) of 150 kPa.

The above values are for vertical, concentric loads only. Effects of load inclination and eccentricity should be taken into account as illustrated in the CHBDC, 2000 Clause 6.7.3 and Clause 6.7.4. The design of this gravity wall should involve checking for internal stability, overturning stability and sliding resistance.

Global stability for both gabion walls has been assessed based on typical sections at approximate Stations 45+150 and 45+320 provided by MRC (see Section 11.3).

7.2.3 Horizontal Resistance

Resistance to lateral forces / sliding resistance between the gabion wall and the engineered fill pad hould be evaluated in accordance with the CHBDC, 2000 assuming an ultimate coefficient of friction of 0.7.

7.2.4 Subgrade Preparation

Prior to constructing the pad, the subgrade should be proof-rolled, and all surficial and buried topsoil, loose, soft soils or otherwise disturbed materials should be removed. Local sub-excavation that may be required should be backfilled with compacted granular fill or mass concrete. The engineered fill should consist of OPSS Granular "A" compacted to 100% of its Standard Proctor maximum dry density (SPMDD) at ±2% of optimum moisture content (OPSS 501, Section 501.08.02 Method A).

Once the desired founding subgrade is exposed, careful inspection should be carried out in conjunction with the proof-rolling outlined above. Upon approval by qualified geotechnical personnel, all subgrade should then be covered with engineered fill and/or mass concrete. The silty clay is prone to softening upon exposure to water and should therefore not be left uncovered for extended periods.

7.3 Frost Cover

From a foundation perspective, frost protection is not required for gabion walls. However, it is recommended that adequate frost protection be provided to toe wall foundations. Frost protection may take the form of 1.9 m of earth cover, or equivalent insulation, over the founding elevation.

8 LATERAL EARTH PRESSURES

For cases where backfill to the retaining walls is placed in accordance with OPSD 3101.150 or OPSD 3101.200 (Rev. Nov. 2005), the lateral earth pressure will be governed by the properties of the material within the backfill limits shown in the respective OPSD, i.e. a line projected up at 1.5H: 1V for granular backfill and 1.25H : 1V for rock backfill.

Based on information from MRC, it is understood that Gabion Wall 1 will directly retain the rock fill embankment, and that Gabion Wall 2 will have a Granular B Type II backfill bound by a line projecting up at 1H : 1V from the heel of the wall.

If the support system allows yielding of the wall (unrestrained system), active horizontal earth pressure may be used in the geotechnical design of the structure. If the support system does not allow yielding (restrained system), at-rest horizontal earth pressures should be used. The amount of wall movement required for the development of active, passive and at-rest earth pressures may be interpreted using Figure C6.9.1(a) in the Commentary to the CHBDC, 2000.

Earth pressures acting on the structure for a fully drained backfill should be computed in accordance with Clause 6.9 of the CHBDC but generally are given by the expression:

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

where

P_h	=	horizontal pressure on the wall at depth h (kPa)
K	=	earth pressure coefficient (see below)
γ	=	unit weight of retained soil (see below)
h	=	depth below top of fill where pressure is computed (m)
q	=	value of any surcharge (kPa)

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 2.0 m for Granular B Type I, or at a depth of 1.7 m for Granular A or Granular B Type II.

Earth pressure coefficients for backfill to the retaining walls are dependent on the material used as backfill. Typical unfactored values are shown in the following table.

Wall Conditions	Earth Pressure Coefficient (K)					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35°$; $\gamma = 22.8$ kN/m^3		Rock Fill $\phi = 42°$; $\gamma = 19$ kN/m^3			
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H : 1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (1.25 H : 1V)	Sloping Surface Behind Wall (1.5 H : 1 V)	Sloping Surface Behind Wall (2 H : 1 V)
Active (Unrestrained Wall)	0.27	0.40	0.2	0.37	0.31	.28
At rest (Restrained Wall)	0.43	-	0.33	-	-	-
Passive (Movement Towards Soil Mass)	3.7	-	5.0	-	-	-

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) might be preferred as it results in lower earth pressures acting on the wall.

The factors in the table above are "ultimate" values and require certain movements for the respective conditions to be mobilized. The values to be used in design can be estimated from Figure C6.9.1 (a) in the Commentary to the CHBDC, 2000.

9 EXCAVATION AND BACKFILL

9.1 General

All excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the native silty clay at this site are classified as Type 2 soils above the water table. All these soils below the water table and existing fills are classified as Type 3 soils.

9.2 Foundations

The excavation and backfilling for foundations must be carried out in accordance with SP 902S01.

9.3 Excavations

Where required, excavations for gabion wall construction will extend through the existing fill and the native silty clay. At locations where there is space restriction or where a slope has to be retained, the excavations will need to be carried out in conjunction with a temporary shoring system.

An item titled "Roadway Protection" as per SSP 105S19 will have to be included in the contract documents. It is recommended that Performance Level 2 as per Clause 539.04.02.01 be specified for this site.

A braced soldier pile and lagging wall is considered to be suitable for use as temporary shoring at this site. It is anticipated that the shoring system may be stiffened by struts or cross bracings, where applicable.

For a temporary braced soldier pile and lagging wall, the lateral pressure diagram as shown on Figure D1 may be used for design in conjunction with the following parameter values.

γ = 20 kN/m^3
γ_w = 10 kN/m^3
K_a = 0.4 (silty clay)
h_w = 0
(assuming that there is no hydrostatic pressure build-up behind a presumably permeable wall)
H = depth to base of excavation, m

Below the excavation base, lateral earth pressures are applied over a width of 3B, where B is the diameter of the socket, to take into account three dimensional effects. The ultimate passive force that can be mobilized by the embedded portion of a socket within rock is constant with depth and is given by :

$$P_p = 6\,c\,B\,L$$

where

c = 2,000 kPa (equivalent Mohr-Coulomb cohesion based on Hoek and Brown rock mass classification)
L = depth of socket in rock, m

10 GROUNDWATER CONTROL

Groundwater perched in the fill and sand seams within the silty clay deposit will seep into local excavations. Surface runoff may also tend to pond in the excavations. The design of the gabion wall foundations will not be influenced by the groundwater, but the Contractor must make provision to control the water seepage and ponding by using sump pumps to remove any accumulated water from the footing base prior to compacting granular fill or placing concrete.

11 HIGH FILL (STATIONS 45+180 TO 45+280)

11.1 Methods of Stability and Settlement Analyses

For the purpose of embankment stability analyses in this report, the commercially available slope stability program GSLOPE developed by Mitre Software Inc. was used.

The global stability of the new road embankment depends on its slope geometry, foundation conditions, and the nature of the existing and new fill materials. Based on consideration of the risks involved, past experience of road embankment performance and site specific conditions, a minimum Factor of Safety (F.S.) of 1.3 is considered appropriate to achieve global embankment stability at this site.

The global stability of embankment fills due to seismic loading has been assessed based on a pseudo-static approach. In this approach, the horizontal acceleration associated with a F.S. of 1.0, referred to as the "yield acceleration", is compared with 67% of the peak horizontal acceleration (0.67 x PHA) based on seismic parameters applicable to the site location. The potential for embankment instability and lateral spreading is considered low if the yield acceleration is higher than 0.67 x PHA.

The magnitudes of consolidation settlements were calculated using an in-house spreadsheet program based on one-dimensional consolidation theory. The Shelby tube sample obtained from Borehole 05-6A contained frequent sand and silt seams, and was therefore unsuitable for laboratory oedometer testing. Consolidation parameters were estimated based on soil index properties and SPT 'N' values.

Seismic Considerations

The following seismic parameters have been used for design:

- Velocity Related Seismic Zone: 2
- Zonal Velocity Ratio: 0.1
- Acceleration Related Seismic Zone: 4
- Zonal Acceleration Ratio: 0.2

The Soil Profile Type at this site is classified as Type 1, which, according to Table 4.4.6.1 of the CHBDC, is associated with a Site Coefficient (also referred to as the ground motion amplification factor) of 1.0. A value of 67% of the Peak Horizontal Ground Acceleration (0.67 x PHA) at ground surface of 0.134g (0.67 x 0.2 g), where g is the acceleration due to gravity, has been used in the analysis. This PHA value corresponds to a probability of exceedance of 10% in 50 years.

Anticipated foundation settlements due to over-consolidation and primary consolidation, where applicable, of the foundation silty clay have been estimated based on the methods described in the CHBDC, 2000 Commentary Section C6.6.3.6.

11.2 Embankment Design and Construction

The new high fill will straddle the lower rock fill slope of the existing east approach of the Madawaska River bridge and the shallow native silty clay beyond the existing slope toe. The new fill will be up to 9 m above the existing grade of the toe, or about 0.5 m above the grade of the mid-height berm. The crest width of the new road embankment is approximately 8 m.

It is understood, with the exception of the granular core, that the east approach embankment of the existing Madawaska River bridge consists of rock fill. From an existing preliminary design drawing, the slope of the granular core is inclined at approximately 1H : 1V.

For blast rock fill, the exposed side slopes of the new road embankment should be formed at an inclination of 1.25H : 1V.

If the magnitudes and/or timing of the settlements discussed in the following sub-section 11.3.3 are unacceptable, conventional means of ground improvement such as preloading/surcharging may be considered to minimize post construction, time-dependent, settlements. Ground improvement works may be carried out in advance of the main construction. Moreover, global embankment stability can be enhanced by means of slope geometry modification such as the provision of berms.

11.3 Stability Analysis

For the purpose of embankment stability analysis, the granular core is assumed to be engineered Granular A or Granular B Type II material with a side slope of 1H : 1V. It is assumed that the new rock fill will have a side slope of 1.25H : 1V.

For analysing global stability of Gabion Wall 1 (Station 45+150), it is assumed that the wall is to directly retain a rock fill embankment with a slope inclination of 1.25H : 1V.

For analysing global stability of Gabion Wall 2 (Station 45+320), it is assumed that the wall is to retain an earth fill slope with an inclination of 3H : 1V.

11.3.1 Static Analysis

Results of static stability analyses carried out for the proposed embankments and gabion wall locations at this site indicate that adequate Factors of Safety (F.S.) for global stability can be maintained. Figures FU1 to FU7 and FWA1 to FWA4 present selected stability analysis results.

The range of F.S. for selected locations are summarized as follows:

Stations 45+180 to 45+280 - High Fills Up to 9 m

Location	Type of Analysis	Factor of Safety (minimum)
Existing East Approach	Drained	1.45 (overall slope)
	Undrained	1.20 (overall slope)
Existing East Approach with New Road Embankment	Drained	1.75 (overall slope) 1.35 (new embankment)
	Undrained	1.45 (overall slope) 1.75 (new embankment)

Although the undrained F.S. (short term condition that had occurred during original construction) for the existing overall east approach slope is less than 1.3, it should not have adverse effects on the new embankment since placement of new rock fill against the lower slope would increase the F.S.

Gabion Walls 1 and 2

Location	Type of Analysis	Factor of Safety (minimum)
Gabion Wall 1	Drained	1.50
	Undrained	> 2
Gabion Wall 2	Drained	1.30
	Undrained	> 2

11.3.2 Seismic Analysis

Liquefaction Potential

In general, the new road embankment will consist of rock fill placed on existing rock fill and native silty clay overlying bedrock. The groundwater level is below the base of the embankments. Based on the CHBDC, the foundation materials have negligible potential for liquefaction.

Consequently, the new road embankments and cuts, including the sections with gabion walls, will be stable against seismic activities at this site. Some toe failure may occur due to seismic loading, but this is expected to be minor in nature and readily repairable.

Limit Equilibrium

Results of pseudo-static analysis indicate that the yield acceleration is higher than 67% of the peak horizontal ground acceleration (0.67 x PHA) in the critical cases.

Figures FS1 and FS2 in Appendix F present the results of pseudo-static analyses carried out to estimate the dynamic stability of a rock embankment with a granular core subject to seismic loading. Figures FS3 and FS4 show results of similar analyses for the gabion wall locations. Based on these results, it is considered that there is negligible potential for embankment foundation failure to occur upon seismic loading at this site.

11.3.3 Settlement Analysis

Within the new road section where up to 9 m of fill will be placed, a combination of the applied vertical stress, depth to bedrock and silty clay consistency could result in consolidation settlement.

Total settlement at the top of the embankment is a result of settlement of the foundation soils and settlement due to compression of the embankment material.

At this site, the new embankment is to be constructed directly on the existing rock fill slope and on the over-consolidated silty clay overlying bedrock. The foundation silty clay deposit at this site is stiff within the upper portion becoming firm with depth until bedrock is encountered.

Settlement of the cohesive silty clay foundation soils consists mainly of recompression of the upper, heavily over-consolidated crust with some primary consolidation of the lower, lightly over-consolidated zone. Settlement due to recompression of the clay is anticipated to occur during construction.

Stations 45+180 to 45+280

The total foundation settlement within this section consists of mostly elastic recompression and some primary consolidation of the clay.

The compression index for primary consolidation, C_c, was estimated from conventional correlation with liquid limits as well as a graphical correlation with water contents (based on a database developed in-house). The recompression index, C_r, is a function of C_c. The coefficient of consolidation, C_v, was obtained from typical correlations with liquid limits. The preconsolidation pressure, p'_c, was estimated based on correlation with the undrained shear strength, C_u, and plasticity indices. The values used in the settlement calculations are as follows.

- Compression index, C_c = 0.1 (stiff clay); 0.45 (firm clay)
- Recompression index, C_r = 0.01 (stiff clay); 0.045 (firm clay)
- Coefficient of consolidation, C_v = 3 to 9 m^2 / yr.
- Preconsolidation pressure, p'_c = 300 kPa (stiff clay); 140 kPa (firm clay)

The magnitude of elastic recompression of the foundation silty clay is estimated to be up to the order of 50 mm to 75 mm, which is anticipated to take place during fill placement. Primary consolidation settlement of the silty clay is estimated to be up to the order of 25mm. This settlement is time-dependent and is expected to occur within 3 months after the fill reaches its highest elevation.

11.3.4 Embankment Compression

Embankments constructed with rock fill are anticipated to settle due to compression of the fill materials. This settlement is anticipated to occur due to rock particle reorientation and particle deterioration due to high contact stresses. Such settlement is time dependent and is expected to take place throughout the years after construction. For planning and design purposes, it is assumed that the magnitude of rock fill settlement will be up to 0.5%, or about 45 mm, of the new embankment height.

Consideration should be given to over-building the new embankment, i.e. wider platform, in accordance with the latest MTO practice in order to accommodate the effects of foundation clay settlement and embankment compression.

11.3.5 Embankment Construction

Embankment construction should be carried out in accordance with OPSS 206, as amended by Special Provision "Amendment to OPSS 206, December 1993", dated November 2002. It is anticipated that rock fill will be used to construct the new road embankment.

If earth fill is to be used in some localized areas, vegetation cover should be established on all exposed earth slopes to protect against surficial erosion. Reference may be made to SP 572S01 (supersedes OPSS 572) for more detailed requirements.

Berms are only required for rock fill heights of 10 m or higher to address surficial stability and to provide access for post construction maintenance. Such berms are therefore not required for the new road embankment at this site.

12 DYNAMIC EARTH PRESSURE COEFFICIENTS

In accordance with Clause 4.6.4 of the CHBDC, retaining structures such as gabion walls should be designed using active (K_{AE}) and passive (K_{PE}) earth pressure coefficients that incorporate the effects of earthquake loading.

In calculating the values of (K_{AE}) and (K_{PE}), the following geotechnical parameters were used:

ϕ = angle of internal friction of backfill

δ = angle of friction between the wall and the backfill

The seismic earth pressure coefficients to be used in design at this site are shown in the table below.

Wall Condition	Earth Pressure Coefficient (K) for Earthquake Loading						
	Height of Application From Base as Percentage of Wall Height	Granular A or Granular B Type II $\phi = 35°$, $\delta = 17.5°$		Rock Fill $\phi = 42°$, $\delta = 21°$			
		Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H : 1V)	Sloping Surface Behind Wall (1.5H : 1V)	Sloping Surface Behind Wall (1.25H : 1V)
Active (K_{AE})	40%	0.33	0.70	0.26	0.40	0.63	> 0.73*
Passive (K_{PE})	33%	3.5	-	4.8	-	-	-
At Rest (K_{OE})**	45%	0.67	-	0.58	-	-	-

* Slope may undergo movement for short durations during seismic activities
** After Woods

13 CONSTRUCTION CONCERNS

During construction, the Contract Administrator should employ experienced geotechnical staff to observe construction activities related to foundation construction.

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

- disturbance of the silty clay or existing fill subgrade under the gabion wall foundations,
- maintaining stability of the existing east slope during construction of Gabion Wall 2 and the adjacent roadway.

LICENSED PROFESSIONAL ENGINEER
Sydney Pang
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Aug.14/06
PROVINCE OF ONTARIO

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Review Principal, Designated MTO Contact

TABLE 1
Upper Access Road
Point Load Test Results

Depth feet	Depth Inches	m	Is50	UCS (MPa)
UAR 05-1				
4	6	1.37	2.808	67.39
6	3	1.91	4.320	103.68
7	0	2.13	3.888	93.31
9	11	3.02	2.160	51.84
11	4	3.45	3.672	88.13
13	0	3.96	2.160	51.84
14	0	4.27	3.240	77.76

Total Rock Core

Average	Minimum	Maximum	
76	52	104	MPa

Run #	Average
1	67.39
2	82.94
3	72.57

Depth feet	Depth Inches	m	Is50	UCS (MPa)
UAR 05-2				
22	8	6.91	4.320	103.68
23	11	7.29	3.024	72.57
25	11	7.90	4.320	103.68
26	6	8.08	3.456	82.94
27	8	8.43	3.240	77.76
29	0	8.84	4.752	114.05
30	4	9.25	3.888	93.31
31	10	9.70	3.024	72.57

Total Rock Core

Average	Minimum	Maximum	
90	73	114	MPa

Run #	Average
1	93.31
2	91.58
3	82.94

Depth feet	Depth Inches	m	Is50	UCS (MPa)
UAR 05-3				
36	5	11.10	9.936	238.46
38	4	11.68	8.208	196.99
40	1	12.22	12.528	300.67
41	3	12.57	3.024	72.57
43	0	13.11	3.024	72.57
44	6	13.56	3.024	72.57

Total Rock Core

Average	Minimum	Maximum	
159	73	301	MPa

Run #	Average
1	245.37
2	72.57

Depth feet	Depth Inches	m	Is50	UCS (MPa)
UAR 05-4				
54	0	16.46	4.752	114.05
55	7	16.94	4.752	114.05
57	0	17.37	3.672	88.13
58	7	17.86	3.456	82.94
59	4	18.08	6.480	155.52
61	5	18.72	3.888	93.31
62	9	19.13	4.320	103.68

Total Rock Core

Average	Minimum	Maximum	
107	83	156	MPa

Run #	Average
1	114.05
2	108.86
3	98.49

Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

NOTE: Hierarchy of Soil Strength Prediction

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

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE

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

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

$\underline{\nabla}$ 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.

UNIFIED SOILS CLASSIFICATION

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

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

Fresh (FR)	No visible signs of weathering.
Fresh Jointed (FJ)	Weathering limited to the surface of major discontinuities.
Slightly Weathered (SW)	Penetrative weathering developed on open discontinuity surfaces, but only slight weathering of rock material.
Moderately Weathered (MW)	Weathering extends throughout the rock mass, but the rock material is not friable.
Highly Weathered (HW)	Weathering extends throughout the rock mass and the rock is partly friable.
Completely Weathered (CW)	Rock is wholly decomposed and in a friable condition, but the rock texture and structure are preserved.

SYMBOLS

CLAYSTONE

SILTSTONE

SANDSTONE

COAL

Bedrock (general)

DISCONTINUITY SPACING

Bedding	Bedding Plane Spacing
Very thickly bedded	Greater than 2m
Thickly bedded	0.6 to 2m
Medium bedded	0.2 to 0.6m
Thinly bedded	60mm to 0.2m
Very thinly bedded	20 to 60mm
Laminated	6 to 20mm
Thinly Laminated	Less than 6mm

TERMS

Total Core Recovery: (TCR)	Core recovered as a percentage of total core run length.
Solid Core Recovery: (SCR)	Percent Ratio of solid core of full cylindrical shape recovered. Expressed with respect to the total length of core run.
Rock Quality Designation: (RQD)	Total length of sound core recovered in pieces 0.1m in length or larger as a percentage of total core run length.
Uniaxial Compressive Strength (UCS)	Axial stress required to break the specimen
Fracture Index: (FI)	Frequency of natural fractures per 0.3m of core run.

STRENGTH CLASSIFICATION

Rock Strength	Approximate Uniaxial Compressive Strength (MPa)	(psi)	Field Estimation of Hardness*
Extremely Strong	Greater than 250	Greater than 36,000	Specimen can only be chipped with a geological hammer
Very Strong	100-250	15,000 to 36,000	Requires many blows of geological hammer to break
Strong	50-100	7,500 to 15,000	Requires more than one blow of geological hammer to break
Medium Strong	25.0 to 50.0	3,500 to 7,500	Breaks under single blow of geological hammer.
Weak	5.0 to 25.0	750 to 3,500	Can be peeled by a pocket knife with difficulty
Very Weak	1.0 to 5.0	150 to 750	Can be peeled by a pocket knife, crumbles under firm blows of geological pick.
Extremely Weak (Rock)	0.25 to 1.0	35 to 150	Indented by thumbnail

THURBER

Ministry of Transportation
Ontario

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RECORD OF BOREHOLE No UAR 05-1

METRIC

W.P. 647-92-00 LOCATION N 5 031 123.7 E 316 762.1 (Upper Access Road) ORIGINATED BY SLL

HWY 17 BOREHOLE TYPE Hollow Stem Augers/NQ Coring COMPILED BY WM

DATUM Geodetic DATE 12.12.05 - 12.12.05 CHECKED BY SP

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
82.7											
0.0	**SAND**, some gravel, occasional cobbles, trace silt Very Dense Brown (frozen)		1	SS	89/.275		82		o		
81.7 1.0	AUGER REFUSAL AT 1.04 m. CORING STARTED AT 1.04 m. **CRYSTALLINE LIMESTONE (BEDROCK),** Fresh to slightly weathered, very thinly to thinly bedded, whitish grey with black subvertical banding, strong		1	RUN			81			FI 0 0 1	RUN 1# TCR=80%, SCR=80%, RQD=80%, UCS=67.4MPa
			2	RUN			80			0 0 0	RUN 2# TCR=100%, SCR=100%, RQD=100%, UCS=82.9MPa
			3	RUN			79			2 0 0 3	RUN 3# TCR=100%, SCR=100%, RQD=79%, UCS=72.6MPa
78.4 4.3	Subvertical joints between 4.17 and 4.27 m. END OF BOREHOLE AT 4.32 m. BOREHOLE GROUTED WITH BENTONITE TO GROUND SURFACE.										

Legend: PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No UAR 05-2

METRIC

W.P. 647-92-00 LOCATION N 5 031 106.7 E 316 766.2 (Upper Access Road) ORIGINATED BY SLL

HWY 17 BOREHOLE TYPE Hollow Stem Augers/Mud Rotary Triconing/NQ Coring COMPILED BY WM

DATUM Geodetic DATE 12.12.05 - 13.12.05 CHECKED BY SP

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
82.5								
0.0	**SAND,** some gravel, occasional cobbles and boulders, trace silt Very Dense Brown (frozen)							Could not recover grab sample due to frozen ground
					82			
		1	SS	50/ .150		o		
		2	SS	50/ .075	81			No sample recovery for SS2 due to frozen ground
80.4 / 2.1	Silty **CLAY,** trace sand seams Very Stiff Grey							
					80			
		3	SS	16				No sample recovery for SS3
					79	o		
	becoming firm				78			
		4	SS	6		wP —o— wL		0 6 42 52
					77			
	some gravel, occasional cobbles	5	SS	76/.275		o		Vane could not be pushed to 5.6m depth due to obstruction (possibly cobble)
75.8 / 6.8	AUGER REFUSAL AT 6.76 m. CORING STARTED AT 6.76 m. **CRYSTALLINE LIMESTONE (BEDROCK),** Fresh to slightly weathered, very thinly to thinly bedded, whitish grey with black subvertical banding, strong Vertical joints at 7.75 and 7.85 m	1	RUN		76 / 75		FI 0 1 1	RUN 1# TCR=100%, SCR=100%, RQD=95%, UCS=93.3MPa
	Some quartzite infillings	2	RUN		74		3 1 1 1 1	RUN 2# TCR=98%, SCR=90%, RQD=87%, UCS=91.6MPa
72.7 / 9.9		2	RUN		73		1 0	RUN 2# TCR=100%, SCR=100%, RQD=96%, UCS=82.9MPa

Continued Next Page

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 20 40 60 80 100

PLASTIC LIMIT w_P — NATURAL MOISTURE CONTENT w — LIQUID LIMIT w_L

$+^3, \times^3$: Numbers refer to Sensitivity   20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No UAR 05-2

METRIC

W.P. 647-92-00 LOCATION N 5 031 106.7 E 316 766.2 (Upper Access Road) ORIGINATED BY SLL

HWY 17 BOREHOLE TYPE Hollow Stem Augers/Mud Rotary Triconing/NQ Coring COMPILED BY WM

DATUM Geodetic DATE 12.12.05 - 13.12.05 CHECKED BY SP

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	END OF BOREHOLE AT 9.88 m. BOREHOLE GROUTED WITH BENTONITE TO GROUND SURFACE.										

+ ³, × ³: Numbers refer to Sensitivity

15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No UAR 05-3

METRIC

W.P. 647-92-00	LOCATION N 5 031 058.9 E 316 905.7 (Upper Access Road)		ORIGINATED BY SLL
HWY 17	BOREHOLE TYPE Hollow Stem Augers/NQ Coring		COMPILED BY WM
DATUM Geodetic	DATE 13.12.05 - 14.12.05		CHECKED BY SP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa (FIELD VANE, sensitivity)	WATER CONTENT (%)	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
98.0 / 0.0	**ROCK FILL**							
97.4 / 0.6	Silty **CLAY**, occasional sand seams Stiff to Firm Grey				97			
		1	SS	8	96			
		2	SS	5	95			0 1 67 32
					94	>100		
		3	SS	6	93			
						2.8		
		4	SS	3	92			
					91	2.7		
		5	SS	4	90			
		6	SS	4	89			0 7 61 32

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL; STRAT PLOT; GROUND WATER CONDITIONS

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity   20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No UAR 05-3

METRIC

W.P. 647-92-00 | LOCATION N 5 031 058.9 E 316 905.7 (Upper Access Road) | ORIGINATED BY SLL

HWY 17 | BOREHOLE TYPE Hollow Stem Augers/NQ Coring | COMPILED BY WM

DATUM Geodetic | DATE 13.12.05 - 14.12.05 | CHECKED BY SP

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
								4.4			
86.9	trace sand, trace gravel		7	SS	52/.225		87		o	FI	
11.1	AUGER REFUSAL AT 11.07 m. CORING STARTED AT 11.07 m. **CRYSTALLINE LIMESTONE (BEDROCK)**, Fresh to slightly weathered, very thinly to thinly bedded, whitish grey with black subvertical banding, very strong to strong		1	RUN			86			1 2 2 1	RUN 1# TCR=100%, SCR=100%, RQD=92%, UCS=245.4MPa
84.2			2	RUN			85			3 3 4 0 3	RUN 2# TCR=100%, SCR=100%, RQD=83%, UCS=72.6MPa
13.8	END OF BOREHOLE AT 13.79 m. BOREHOLE GROUTED WITH BENTONITE TO GROUND SURFACE.										

Legend: O UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × LAB VANE. Plastic limit w_P, natural moisture content w, liquid limit w_L.

$+^3, \times^3$: Numbers refer to Sensitivity

15 ⊕ 5 (20 above, 10 below) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No UAR 05-4

METRIC

W.P.	647-92-00	LOCATION	N 5 031 038.6 E 316 909.5 (Upper Access Road)	ORIGINATED BY	SLL
HWY	17	BOREHOLE TYPE	Hollow Stem Augers/NQ Coring	COMPILED BY	WM
DATUM	Geodetic	DATE	15.12.05 - 15.12.05	CHECKED BY	SP

SOIL PROFILE: ELEV/DEPTH, DESCRIPTION, STRAT PLOT. SAMPLES: NUMBER, TYPE, "N" VALUES. GROUND WATER CONDITIONS. ELEVATION SCALE. DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100). SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20, 40, 60, 80, 100). PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (20, 40, 60). UNIT WEIGHT γ kN/m³. REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
102.2 / 0.0	Silty **CLAY**, some topsoil Stiff Dark Brown (FILL)	1	SS	13				
100.7 / 1.5	**TOPSOIL** (100mm), trace rootlets Dark Brown							
100.6 / 1.6	Silty **CLAY**, occasional topsoil inclusions, occasional silt pockets Hard to Very Stiff Brown	2	SS	35				
		3	SS	16	0	3	32	65
		4	SS	11				
	Becoming Firm, Grey	5	SS	6				
		6	SS	7				
		7	SS	4	0	0	50	50

Elevation scale: 102, 101, 100, 99, 98, 97, 96, 95, 94, 93

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 ⊕ 5 / 10 (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No UAR 05-4

METRIC

W.P. 647-92-00	LOCATION N 5 031 038.6 E 316 909.5 (Upper Access Road)	ORIGINATED BY SLL	
HWY 17	BOREHOLE TYPE Hollow Stem Augers/NQ Coring	COMPILED BY WM	
DATUM Geodetic	DATE 15.12.05 - 15.12.05	CHECKED BY SP	

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	DYNAMIC CONE / SHEAR STRENGTH	WATER CONTENT (%)	FI	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	Silty **CLAY**, some silt and sand seams Firm to Soft Grey				92				
		8	SS	2			○		
					91	>100			
					90				
		9	SS	2			○		
					89				
		10	SS	3			wP–w		0 4 64 32
					88	>100			
					87				
		11	SS	6			○		
86.2 16.0	AUGER REFUSAL AT 16.03 m. CORING STARTED AT 16.03 m. **CRYSTALLINE LIMESTONE (BEDROCK)**, Fresh to slightly weathered, very thinly to thinly bedded, whitish grey with black subvertical banding, strong to very strong	1	RUN		86			1, 0, 0, 0	RUN 1# TCR=100%, SCR=100%, RQD=100%, UCS=114.1MPa
		2	RUN		85, 84			0, 0, 0, 0	RUN 2# TCR=100%, SCR=100%, RQD=100%, UCS=108.9MPa
82.9		3	RUN		83			0, 0, 0	RUN 3# TCR=96%, SCR=96%, RQD=96%, UCS=98.5MPa
19.3	END OF BOREHOLE AT 19.25 m. BOREHOLE GROUTED WITH BENTONITE TO GROUND SURFACE.								

$+^3, \times^3$: Numbers refer to Sensitivity   15 ⊕ 5 (20/10) (%) STRAIN AT FAILURE

ONTMT4S 5182-PHASE II.GPJ 01/05/06

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No UAR 05-5

METRIC

W.P. 647-92-00 LOCATION N 5 031 090.3 E 316 782.0 (Upper Access Road) ORIGINATED BY SLL

HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY WM

DATUM Geodetic DATE 13.12.05 - 13.12.05 CHECKED BY SP

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
82.5 / 0.0	**SAND**, some gravel, some silt Compact Brown (frozen) (FILL)	1	GS		○	13 71 16 (SI+CL)
81.8 / 0.7	Silty **CLAY**, some sand, occasional topsoil inclusions Very Stiff to Stiff Brown	1	SS	21	○	
	Becoming Grey	2	SS	9	├○─┤	0 6 63 30
		3	SS	56/.225	○	
79.1 / 3.5	END OF BOREHOLE AT 3.45 m. AUGER REFUSAL AT 3.45 m ON PROBABLE BEDROCK OR BOULDER. BOREHOLE OPEN TO 3.45 m AND DRY UPON COMPLETION. BOREHOLE GROUTED WITH BENTONITE TO GROUND SURFACE.					

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (82, 81, 80); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3

$+^3, \times^3$: Numbers refer to Sensitivity   15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No UAR 05-6

METRIC

W.P. 647-92-00 LOCATION N 5 031 074.5 E 316 818.7 (Upper Access Road) ORIGINATED BY SLL

HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY WM

DATUM Geodetic DATE 13.12.05 - 13.12.05 CHECKED BY SP

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL
83.6									
0.0	TOPSOIL (50mm)								
83.3 0.1	ROCK FILL								
0.2	TOPSOIL (75mm)	1	SS	18					
0.3	Silty CLAY, some silt pockets, trace gravel Very Stiff to Stiff Grey				83				
		2	SS	15	82	0	6	51	43
					81				
	some sand seams	3	SS	6		0	4	61	35
					80	0	39	40	22
79.4 4.3	END OF BOREHOLE AT 4.27 m. AUGER REFUSAL AT 4.27 m ON PROBABLE BEDROCK OR BOULDER. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 0.61 m slotted screen.								

WATER LEVEL READINGS:

DATE	DEPTH (m)
01/27/06	Dry

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

$+^3, \times^3$: Numbers refer to Sensitivity   15-⊕-5 (20/10) (%) STRAIN AT FAILURE

Ministry of Transportation
Ontario

THURBER

RECORD OF BOREHOLE No UAR 05-6A

METRIC

W.P. 647-92-00 LOCATION N 5 031 074.5 E 316 818.7 (Upper Access Road) ORIGINATED BY SLL

HWY 17 BOREHOLE TYPE Hollow Stem Augers COMPILED BY JHL

DATUM Geodetic DATE 24.01.06 - 24.01.06 CHECKED BY SP

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE)	PLASTIC LIMIT w_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT w_L — WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
83.6											
0.0	Augered without sampling to 3.05m depth						83				
							82				
							81				
80.6 / 3.0	Silty **CLAY**, some thin sand lenses Stiff Brown		1	TW	PH						
80.0 / 3.7	END OF BOREHOLE AT 3.66m. BOREHOLE BACKFILLED UPON COMPLETION.						80				

$+^3, \times^3$: Numbers refer to Sensitivity

15-⊕-5 (20 above, 10 below) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No UAR 05-7

METRIC

W.P. 647-92-00	LOCATION N 5 031 073.2 E 316 854.8 (Upper Access Road)	ORIGINATED BY SLL
HWY 17	BOREHOLE TYPE Hollow Stem Augers	COMPILED BY WM
DATUM Geodetic	DATE 13.12.05 - 13.12.05	CHECKED BY SP

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
86.2								
0.0	**TOPSOIL** (200mm)							
0.2	**ROCK FILL**, occasional cobbles and boulders				86			
84.9 / 1.3	Silty **CLAY**, some topsoil inclusions and wood fibres Stiff Dark Brown				85			
		1	SS	10		o (approx. 13)		
					84			
83.6 / 2.6	END OF BOREHOLE AT 2.59 m. AUGER REFUSAL AT 2.59 m ON PROBABLE BEDROCK OR BOULDER. Piezometer installation consists of 19mm diameter Schedule 40 PVC pipe with a 0.61 m slotted screen. WATER LEVEL READINGS: DATE DEPTH (m) 01/27/06 Dry							

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60.

$+^3, \times^3$: Numbers refer to Sensitivity

20 / 15 5 / 10 (%) STRAIN AT FAILURE

Appendix B

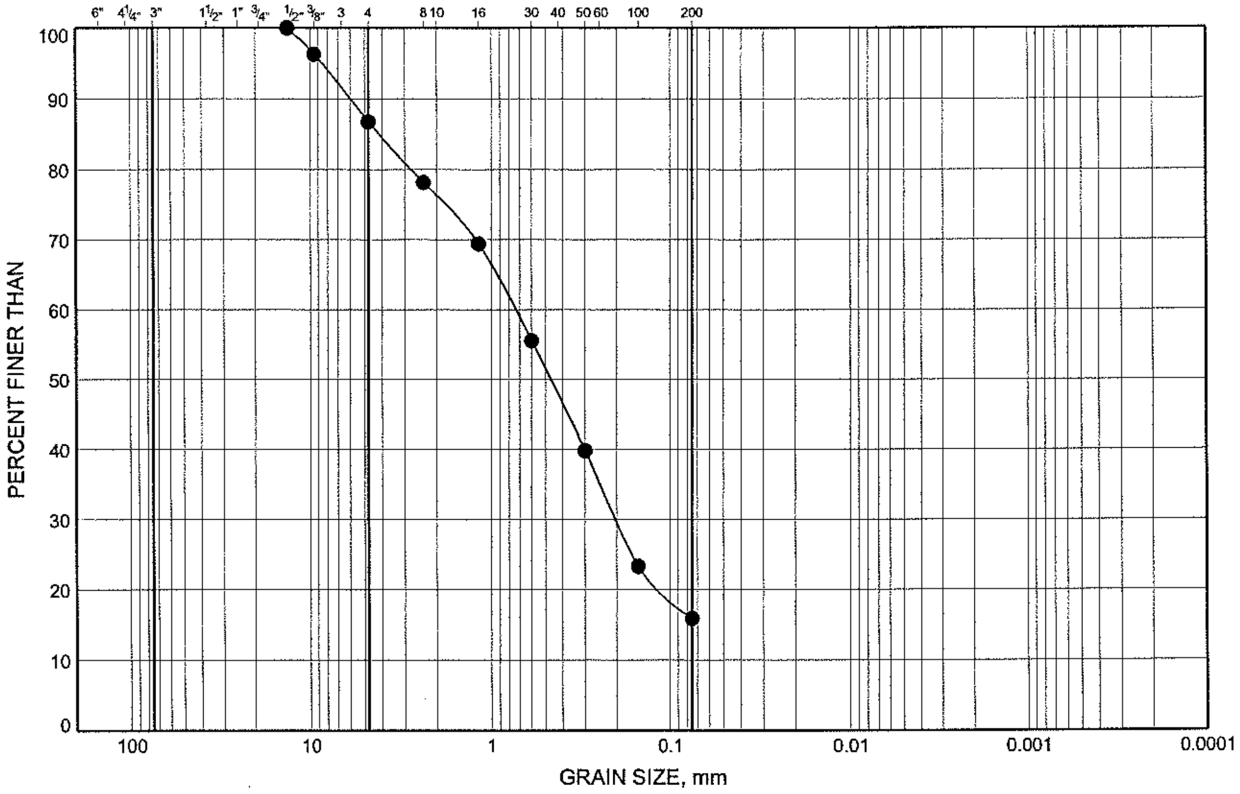
Laboratory Test Results

HWY 17-417

GRAIN SIZE DISTRIBUTION

FIGURE B1

SAND FILL



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	UAR 05-5	0.23	82.28

Date May 2006

Project 647-92-00

THURBER

Prep'd SLL

Chkd. SP

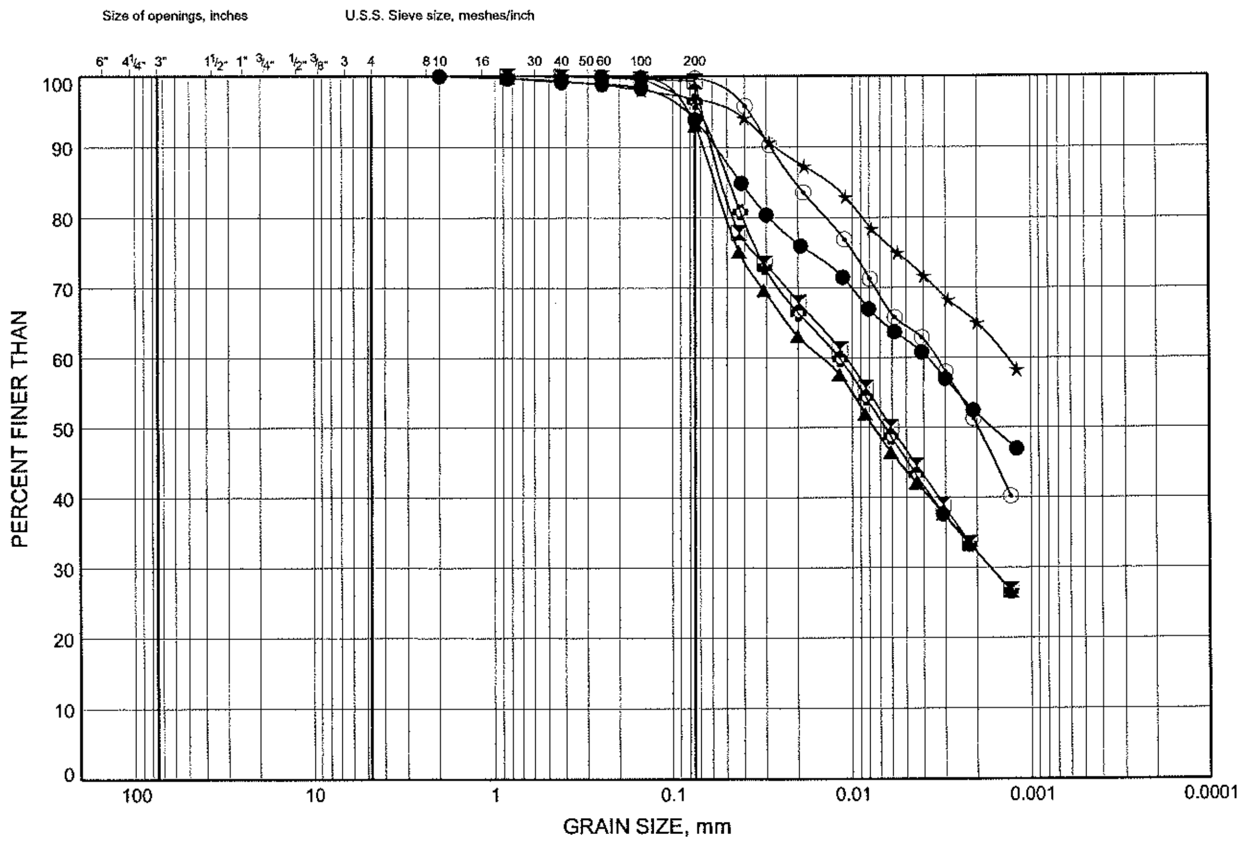
THURBGSD 5182-PHASE II.GPJ 01/05/06

HWY 17-417

GRAIN SIZE DISTRIBUTION

FIGURE B2

SILTY CLAY



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	UAR 05-2	4.88	77.66
⧗	UAR 05-3	3.35	94.61
▲	UAR 05-3	9.45	88.51
★	UAR 05-4	3.35	98.83
⊙	UAR 05-4	9.45	92.73
✪	UAR 05-4	14.02	88.16

Date May 2006

Project 647-92-00

THURBER

Prep'd SLL

Chkd. SP

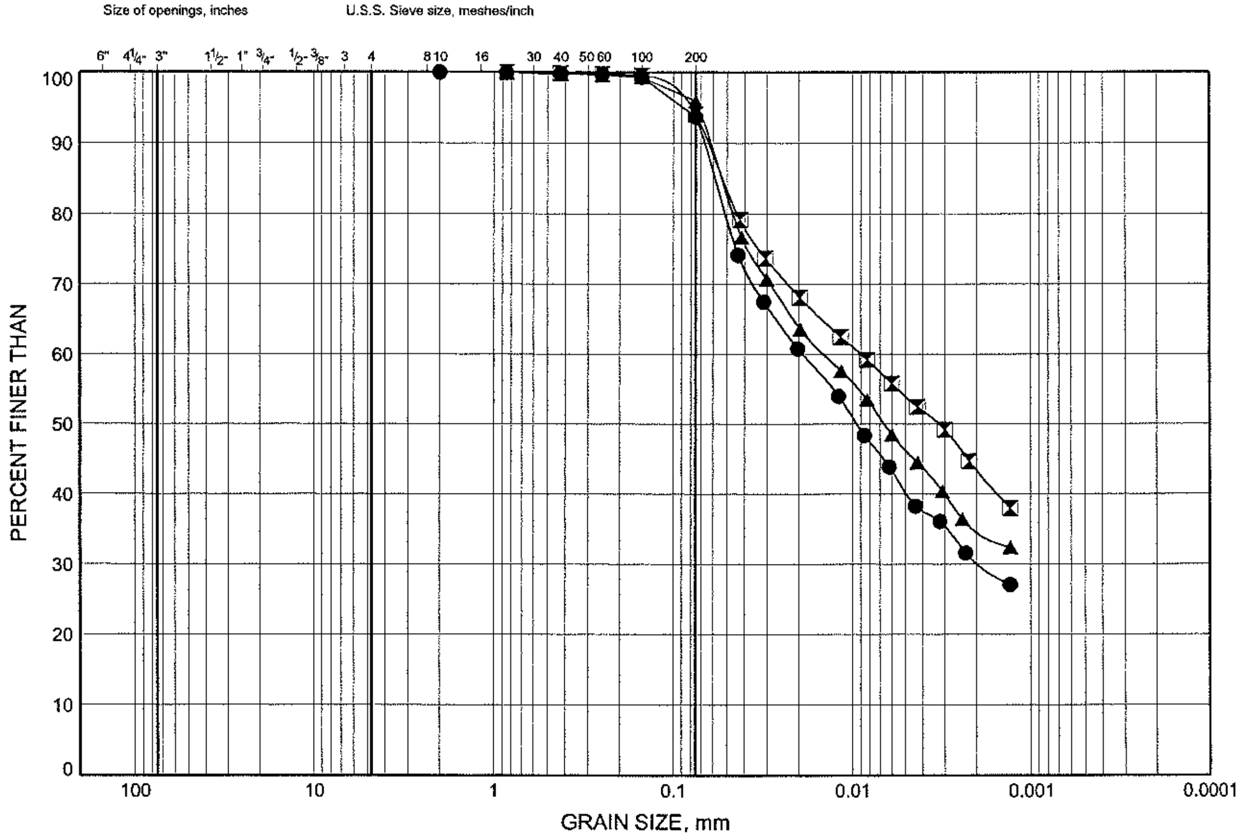
THURBGSD 5182-PHASE II.GPJ 01/05/06

HWY 17-417

GRAIN SIZE DISTRIBUTION

FIGURE B3

SILTY CLAY



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	UAR 05-5	1.83	80.68
⧗	UAR 05-6	1.83	81.79
▲	UAR 05-6	3.28	80.35

Date May 2006

Project 647-92-00

THURBER

Prep'd SLL

Chkd. SP

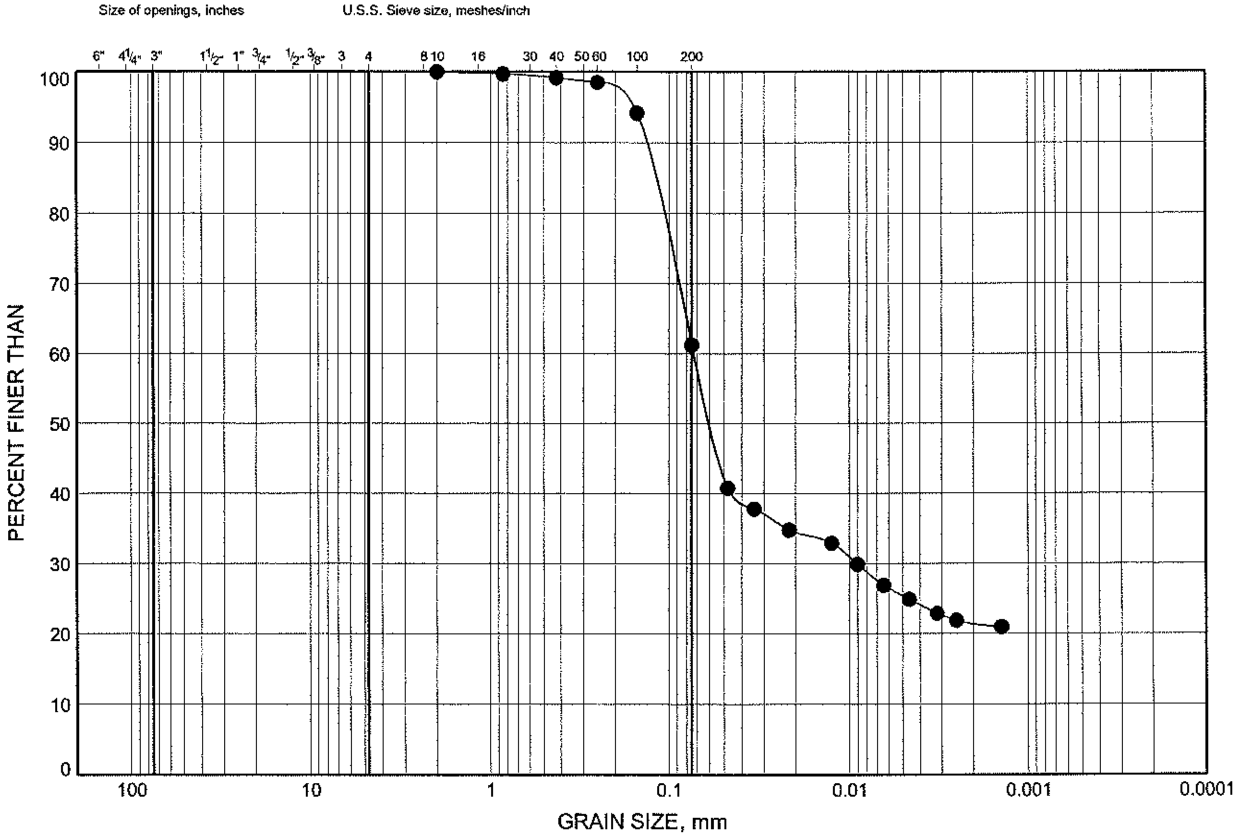
THURBGSD 5182-PHASE II.GPJ 01/05/06

HWY 17-417

GRAIN SIZE DISTRIBUTION

FIGURE B4

SILTY CLAY with sand seams



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	UAR 05-6	3.61	80.01

THURBGSD 5182-PHASE II.GPJ 01/05/06

Date May 2006

Project 647-92-00

THURBER

Prep'd SLL

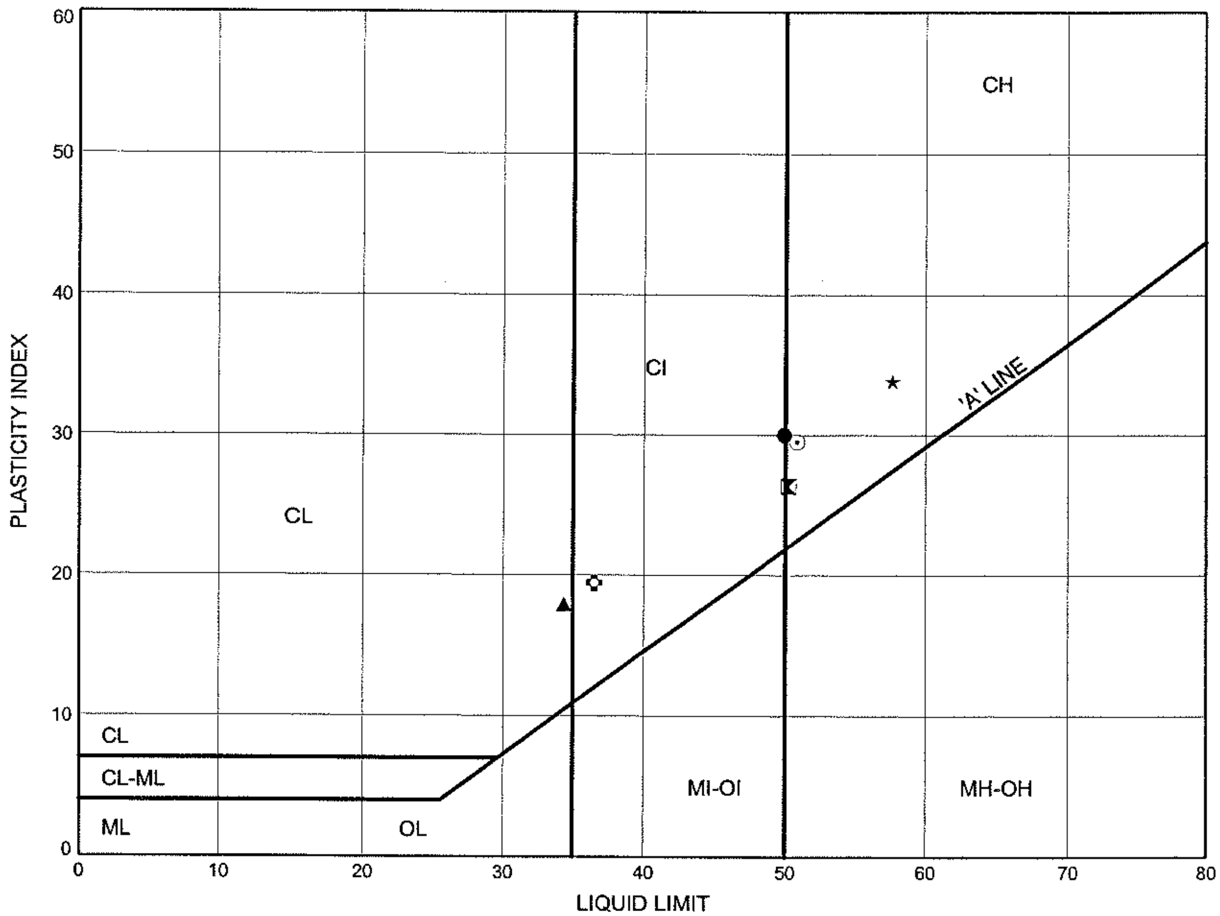
Chkd. SP

HWY 17-417

ATTERBERG LIMITS TEST RESULTS

FIGURE B5

SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	UAR 05-2	4.88	77.66
⧗	UAR 05-3	3.35	94.61
▲	UAR 05-3	9.45	88.51
★	UAR 05-4	3.35	98.83
⊙	UAR 05-4	9.45	92.73
✪	UAR 05-4	14.02	88.16

Date May 2006

Project 647-92-00

THURBER

Prep'd SLL

Chkd. SP

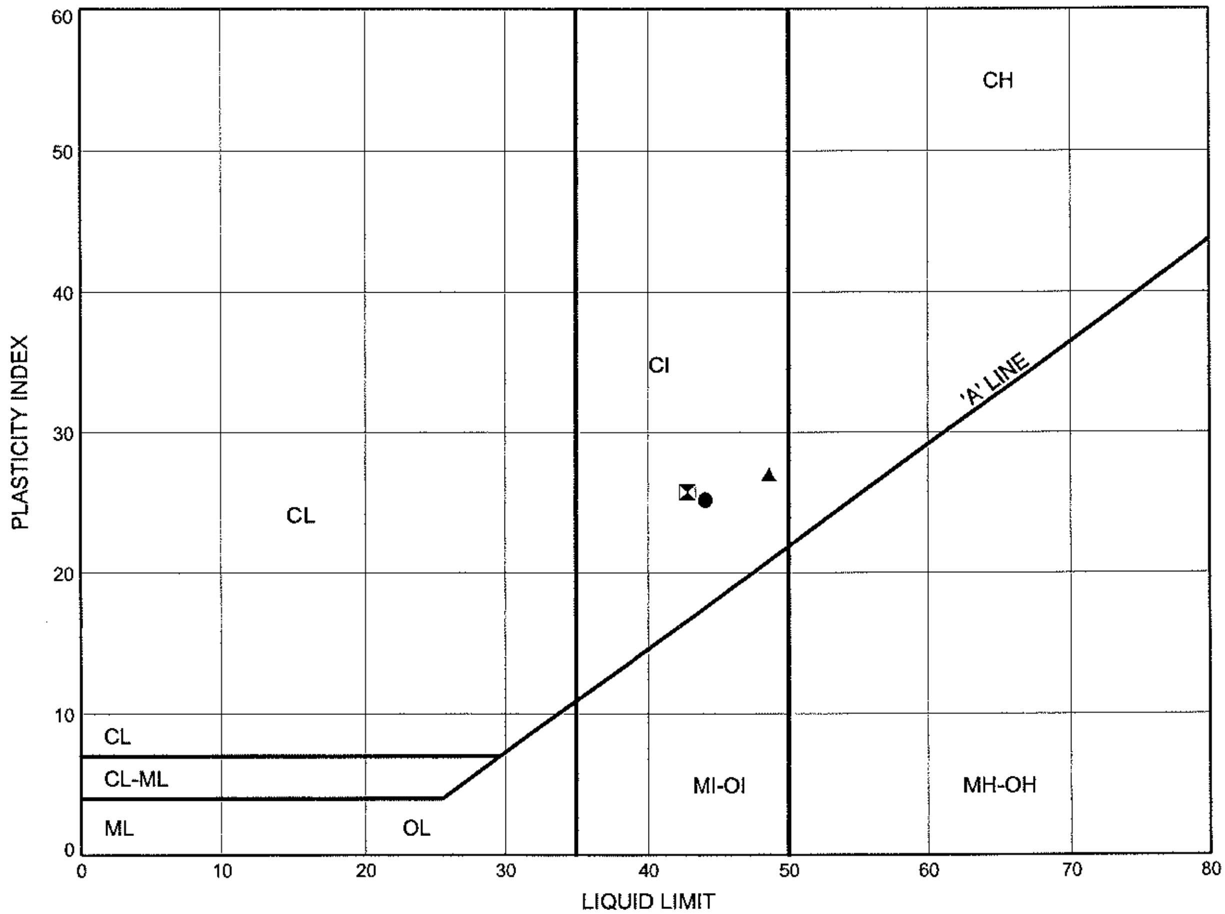
THURBALT 5182-PHASE II.GPJ 01/05/06

HWY 17-417

ATTERBERG LIMITS TEST RESULTS

FIGURE B6

SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	UAR 05-5	1.83	80.68
⊠	UAR 05-6	1.83	81.79
▲	UAR 05-6	3.28	80.35

THURBALT 5182-PHASE II.GPJ 01/05/06

Date May 2006

Project 647-92-00

THURBER

Prep'd SLL

Chkd. SP

Appendix C

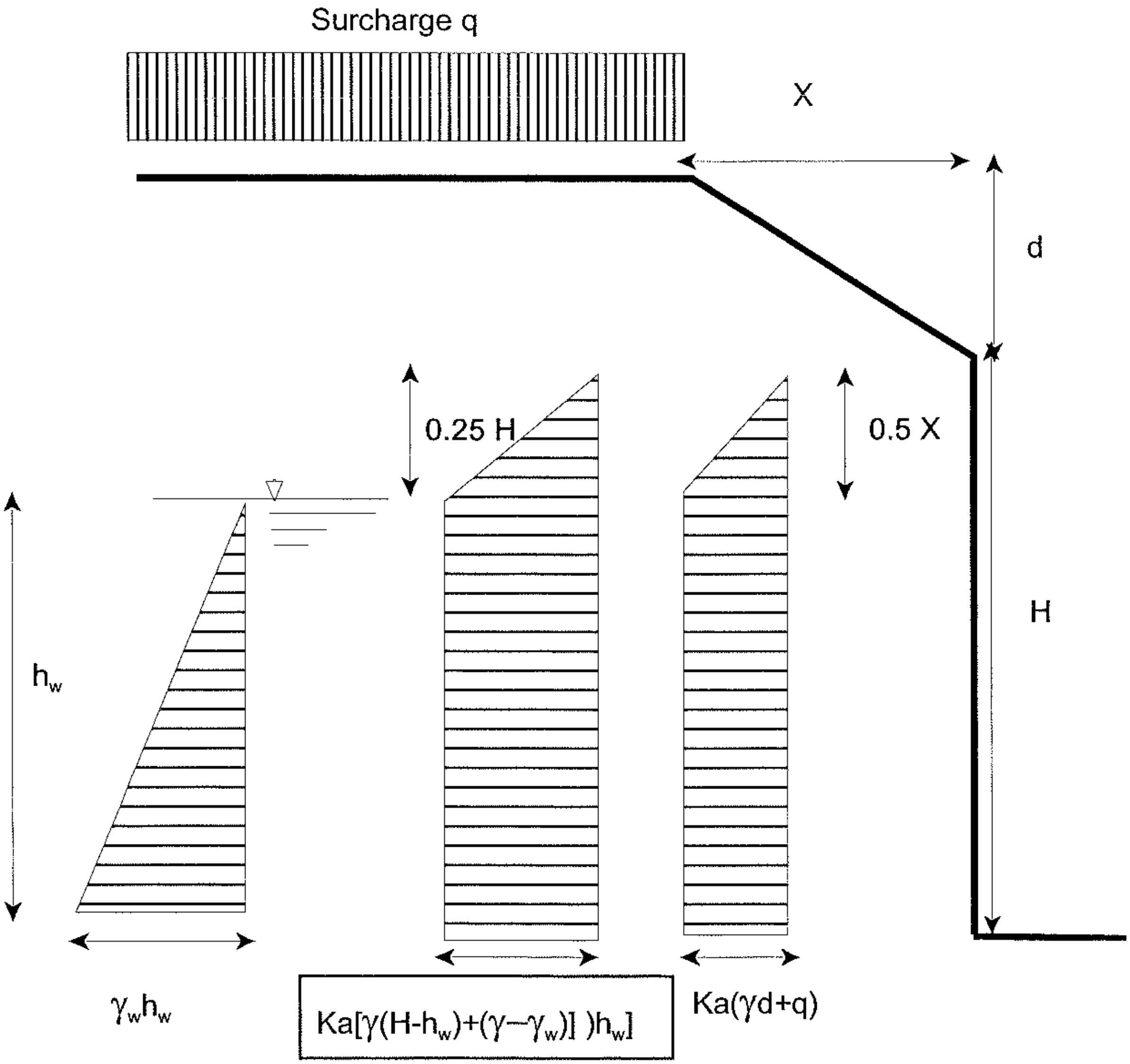
Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR RETAINING STRUCTURES

Foundation Element	Deep Foundations (Sheetpile or Caisson Walls)	Gabion Wall	Concrete Toe Wall	RSS Wall
Retaining Walls	*Advantages:* i. None identified. *Disadvantages:* i. The relatively small retained height and the shallow depths to competent founding stratum at this site does not warrant the use of these foundation types. ii. Relatively cost ineffective for this type of application.	*Advantages:* i. Relative ease of construction. ii. Cost effective for this type of application. iii. Less sensitive to differential settlements. *Disadvantages:* i. Aesthetic concerns with respect to general appearance. ii. Relatively flexible, potentially allowing more lateral ground movement.	*Advantages:* i. Better general appearance than gabions. ii. Relatively stiffer than gabion wall. *Disadvantages:* i. Requires more detailed subgrade preparation and construction procedures than gabions. ii Less cost effective than gabions. iii. More sensitive to differential settlements.	*Advantages:* i. Better general appearance than gabions. ii. More choices of wall types. *Disadvantages:* i. Requires more detailed subgrade preparation and construction procedures than both gabion and toe walls. ii Less readily constructible than both gabion and toe walls. iii. More sensitive to differential settlements along the wall panels.

Appendix D

Figures



γ = unit weight of clay

γ_w = unit weight of water

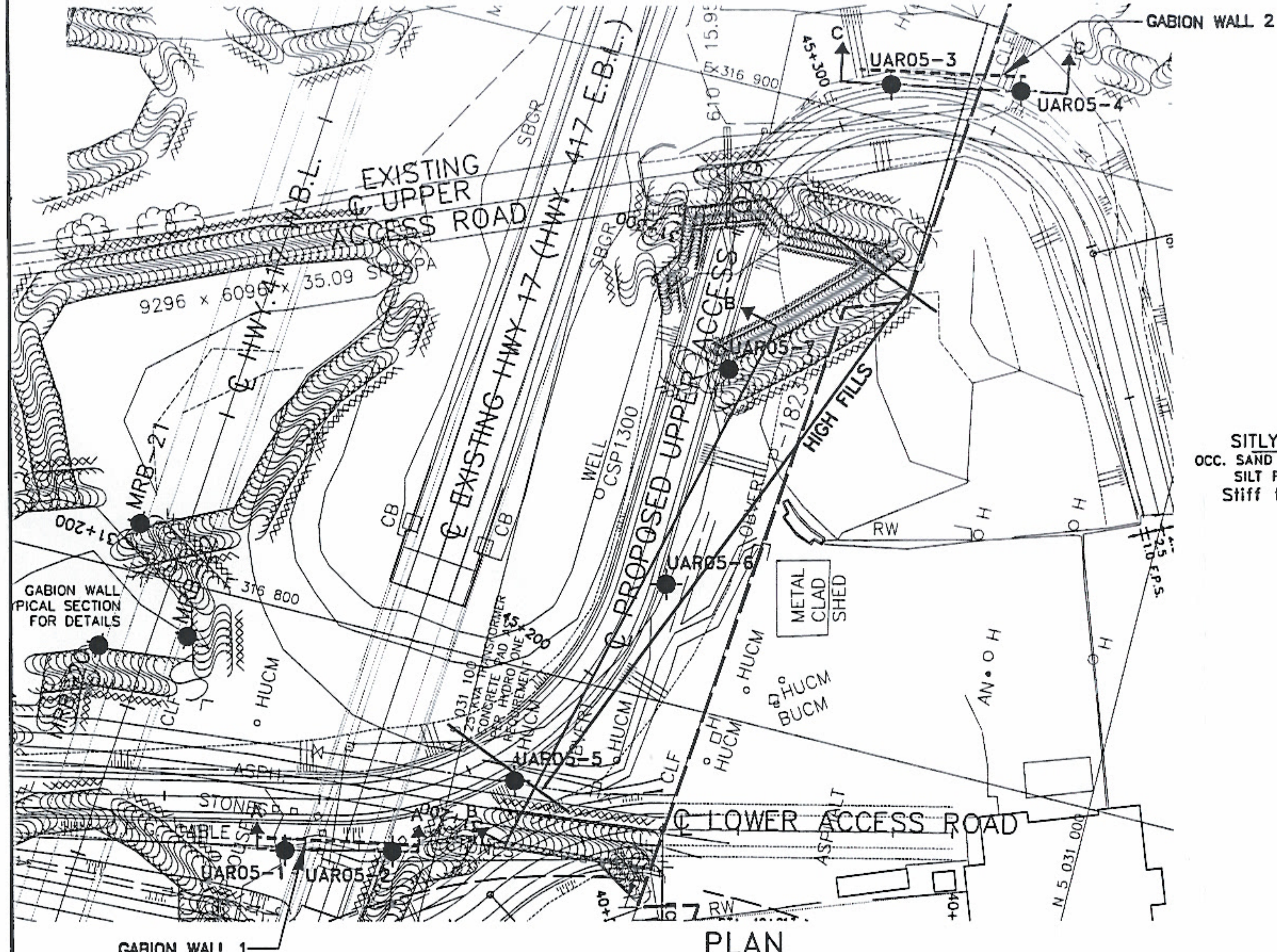
Ka = earth pressure coefficient

THURBER	**LATERAL PRESSURE DISTRIBUTION FOR BRACED SHORING DESIGN**	**FIGURE D1**

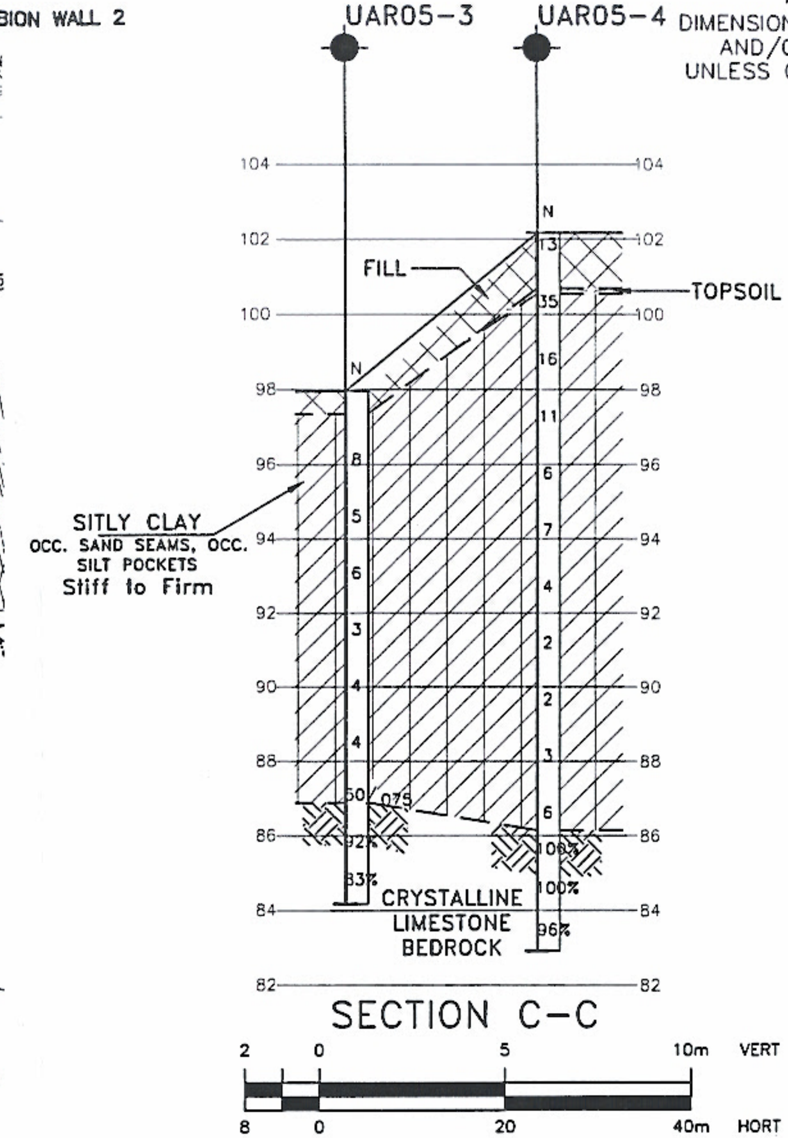
Appendix E

Drawing

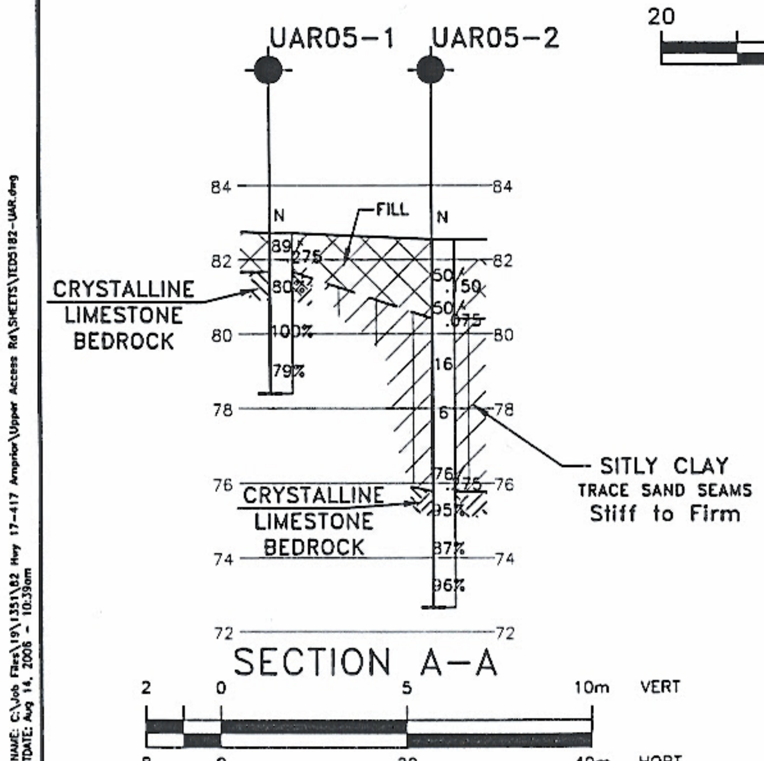
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN



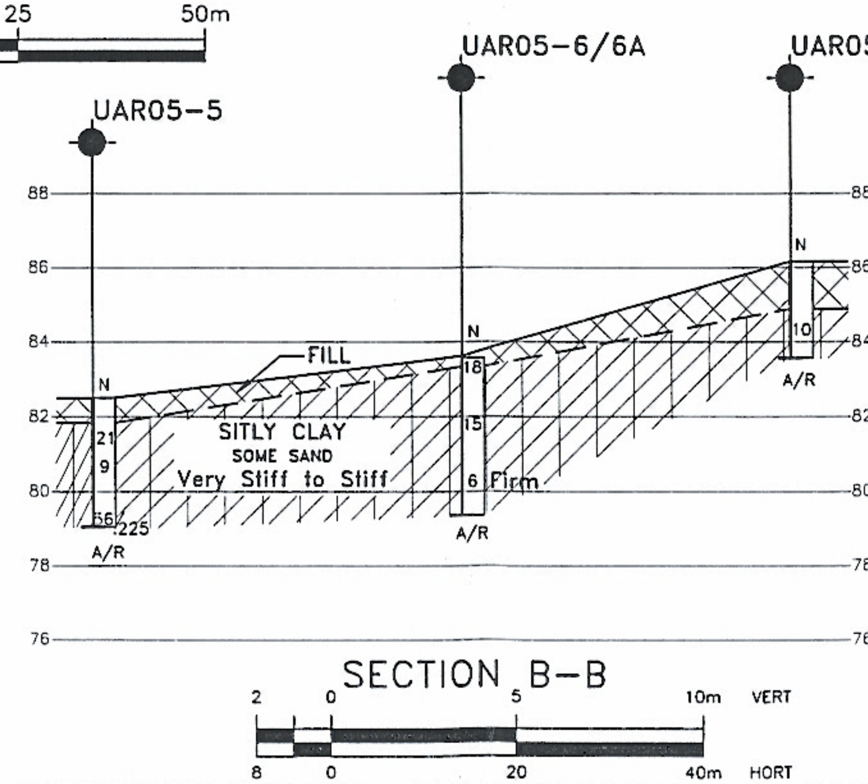
PLAN



SECTION C-C



SECTION A-A



SECTION B-B

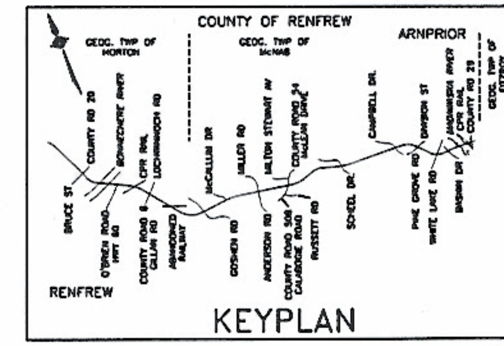
HWY.17
GWP NO. 647-92-00

HIGHWAY 17 TWINNING
UPPER ACCESS ROAD-PROPOSED
GABION WALLS AND HIGH FILLS
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

McCORMICK RANKIN CORPORATION

THURBER ENGINEERING LTD.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS



KEYPLAN

LEGEND

●	Bore Hole
◆	Dynamic Cone Penetration Test (cone)
⊕	Bore Hole & Cone
N	Blows/ 0.3m (Std Pen Test, 475 J/blow)
CONE	Blows/ 0.3m (60° Cone, 475 J/blow)
PH	Pressure, Hydraulic
	WL at Time of Investigation
	Head Artesian Water
	Piezometer
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal

NO	ELEVATION	NORTHING	EASTING
UAR05-1	82.7	5031123.7	316762.1
UAR05-2	82.5	5031106.7	316766.2
UAR05-3	98.0	5031058.9	316905.7
UAR05-4	102.2	5031038.6	316909.5
UAR05-5	82.5	5031090.3	316782.0
UAR05-6/6A	83.6	5031074.5	316818.7
UAR05-7	86.2	5031073.2	316854.8

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

LICENSED PROFESSIONAL ENGINEER
S. PANG
Aug. 14/06
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
P. K. CHATTERJI
Aug 14/06
PROVINCE OF ONTARIO

REVISIONS			
AUG 06	SP	FINAL	
MAY 06	SP	ISSUED AS DRAFT FOR REVIEW	
DATE	BY	DESCRIPTION	

DESIGN SP	CHK PKC	CHBDC 2000	LOAD	DATE MAY 2006
DRAWN SS	CHK SP	SITE 29-201/2	STRUCT	DWG. 19-1351-82-UAR1

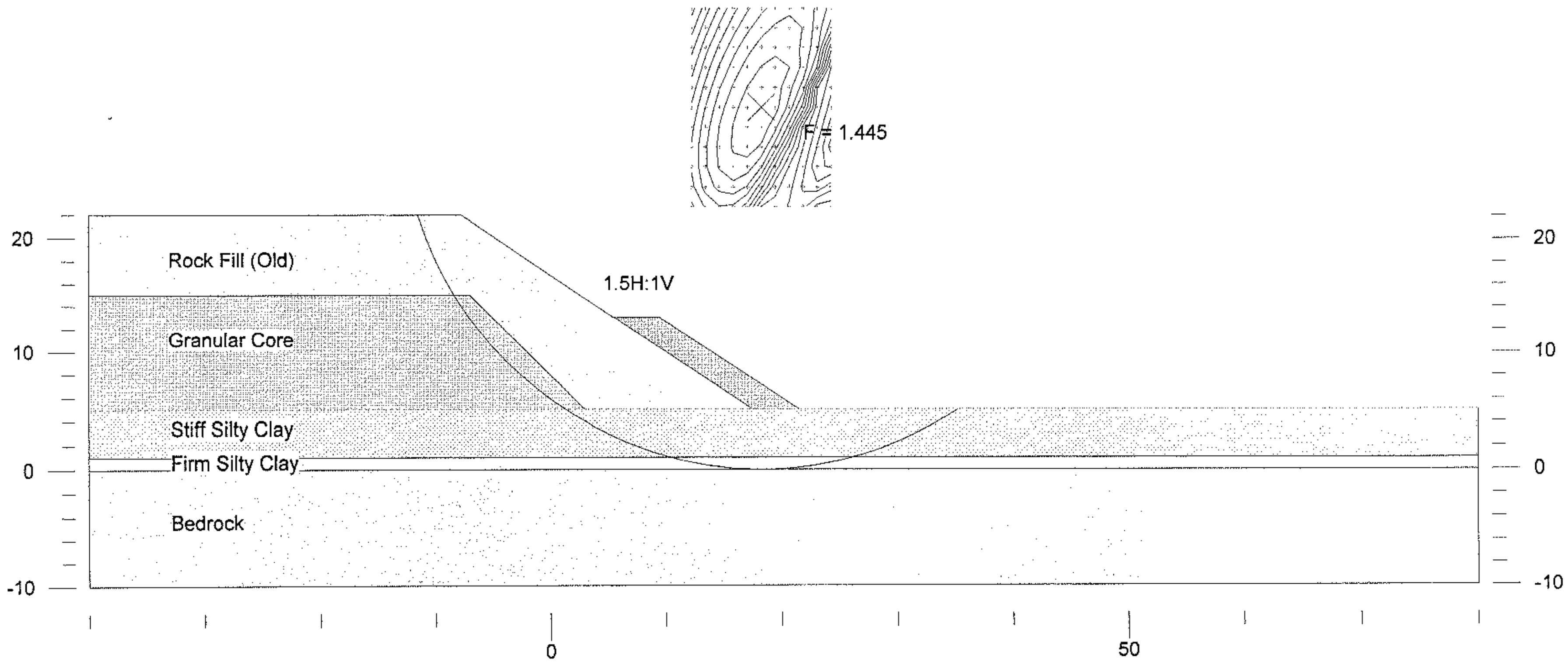
FILENAME: C:\Job Files\19\1351\82 Hwy 17-417 Arnprior\Upper Access Rd\SHEETS\TED0182-UAR.dwg
PLOTDATE: Aug 14, 2006 - 10:39am

Appendix F

Selected Stability Analyses Results

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Rock Berm	19	0	42	0	0
Rock Fill (old)	19	0	42	0	0
Granular Core	22	0	35	0	0
Stiff Silty Clay	19	0	29	0	1
Firm Silty Clay	18	0	27	0	1
Bedrock	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-1351-82
Hwy 17/417 Twinning, Arnprior
April 24, 2006
Stability of New Upper Access Road Embankment
FIGURE FU1 Drained Analysis - Existing East Approach Embankment

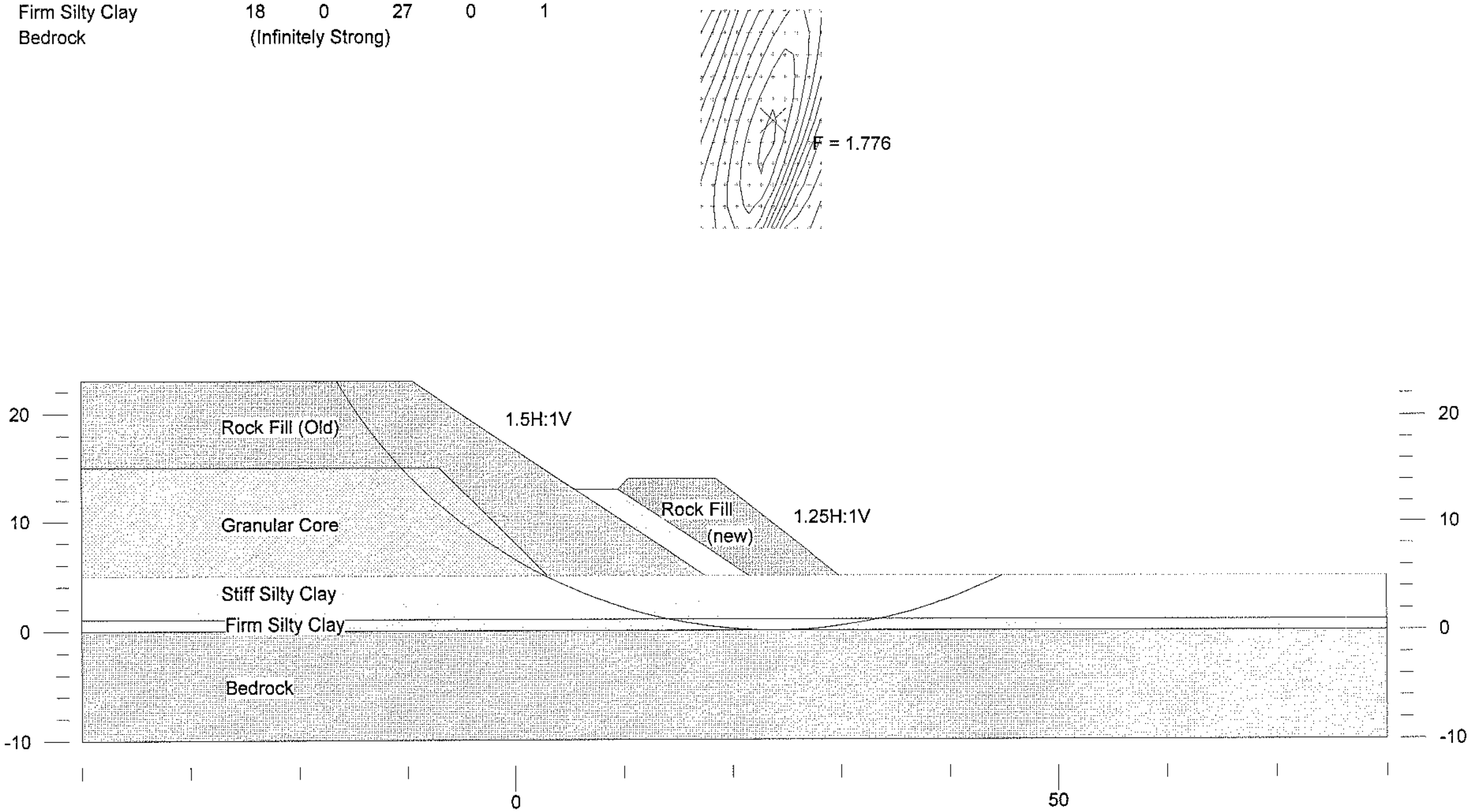


03/05/2006 11:09:30 AM C:\SPANG\THURB~12\19-135~4\FIGFU1.GSL Thurber Engineering Ltd. - Toronto F = 1.445

FIGURE FU1

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Rock Fill (new)	19	0	42	0	0
Rock Berm	19	0	42	0	0
Rock Fill (old)	19	0	42	0	0
Granular Core	22	0	35	0	0
Stiff Silty Clay	19	0	29	0	1
Firm Silty Clay	18	0	27	0	1
Bedrock	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-1351-82
Hwy 17/417 Twinning, Arnprior
April 24, 2006
Stability of New Upper Access Road Embankment
FIGURE FU2 Drained Analysis - 9 m high new road embankment



03/05/2006 11:29:08 AM C:\SPANG\THURB~12\19-135~4\FIGFU2.GSL Thurber Engineering Ltd. - Toronto F = 1.776

FIGURE FU2

Thurber Engineering Ltd. - Toronto
19-1351-82
Hwy 17/417 Twinning, Arnprior
April 24, 2006
Stability of New Upper Access Road Embankment
FIGURE FU3 Drained Analysis - 9 m high new road embankment

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Rock Fill (new)	19	0	42	0	0
Rock Berm	19	0	42	0	0
Rock Fill (old)	19	0	42	0	0
Granular Core	22	0	35	0	0
Stiff Silty Clay	19	0	29	0	1
Firm Silty Clay	18	0	27	0	1
Bedrock	(Infinitely Strong)				

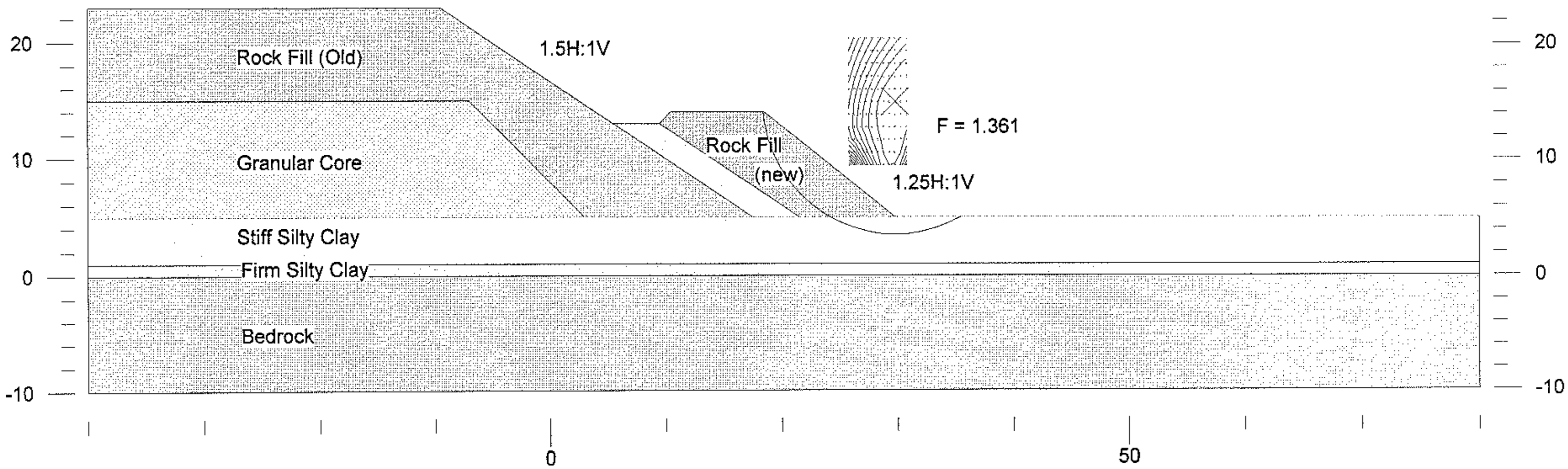


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FIGURE FU3

Thurber Engineering Ltd. - Toronto
19-1351-82
Hwy 17/417 Twinning, Arnprior
April 24, 2006
Stability of New Upper Access Road Embankment
FIGURE FU4 Drained Analysis - 9 m high new road embankment

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Rock Fill (new)	19	0	42	0	0
Rock Berm	19	0	42	0	0
Rock Fill (old)	19	0	42	0	0
Granular Core	22	0	35	0	0
Stiff Silty Clay	19	0	29	0	1
Firm Silty Clay	18	0	27	0	1
Bedrock	(Infinitely Strong)				

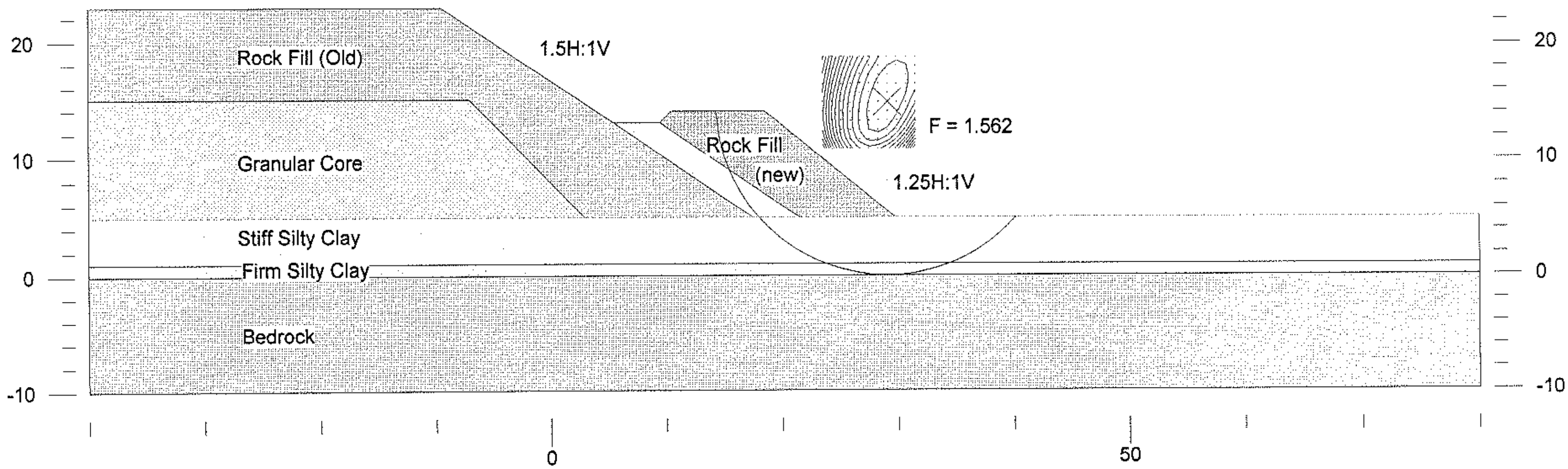

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FIGURE FU4

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Rock Berm	19	0	42	0	0
Rock Fill (old)	19	0	42	0	0
Granular Core	22	0	35	0	0
Stiff Silty Clay	19	70	0	0	1
Firm Silty Clay	18	35	0	0	1
Bedrock	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-1351-82
Hwy 17/417 Twinning, Arnprior
April 24, 2006
Stability of New Upper Access Road Embankment
FIGURE FU5 Undrained Analysis - Existing East Approach Embankment

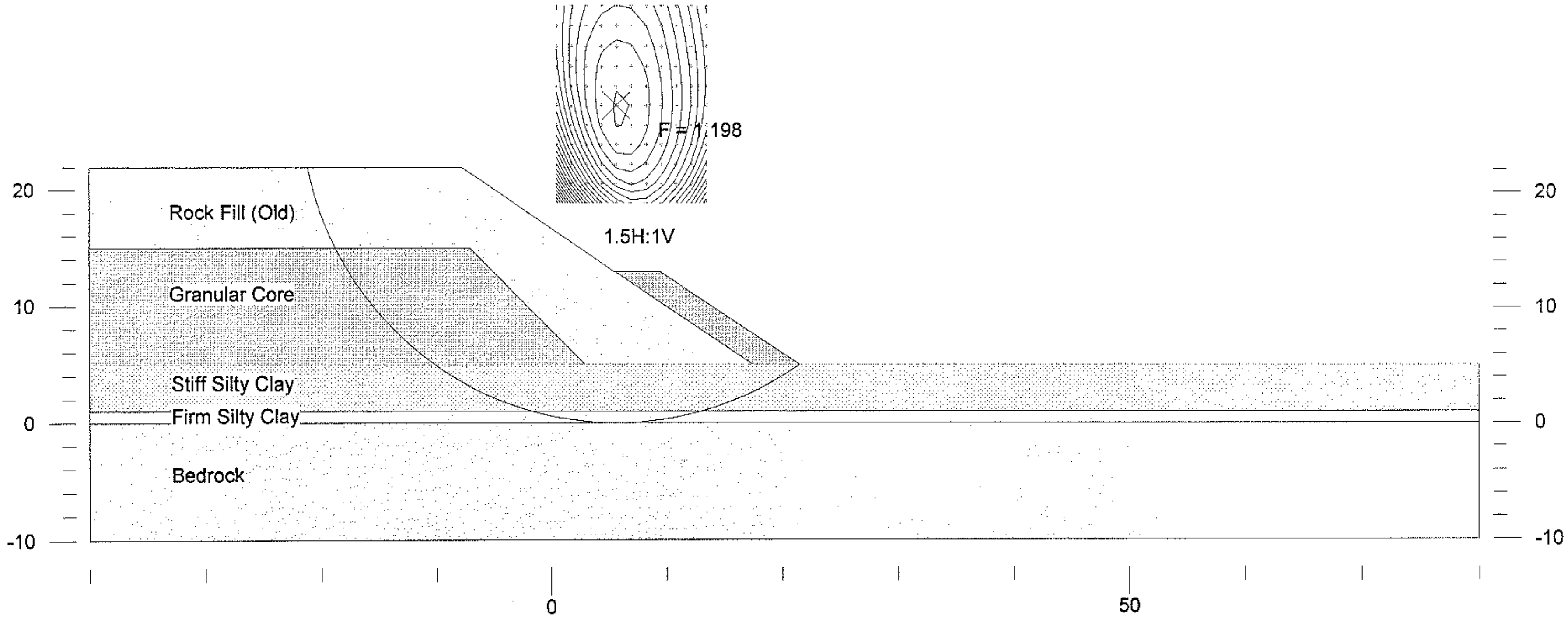

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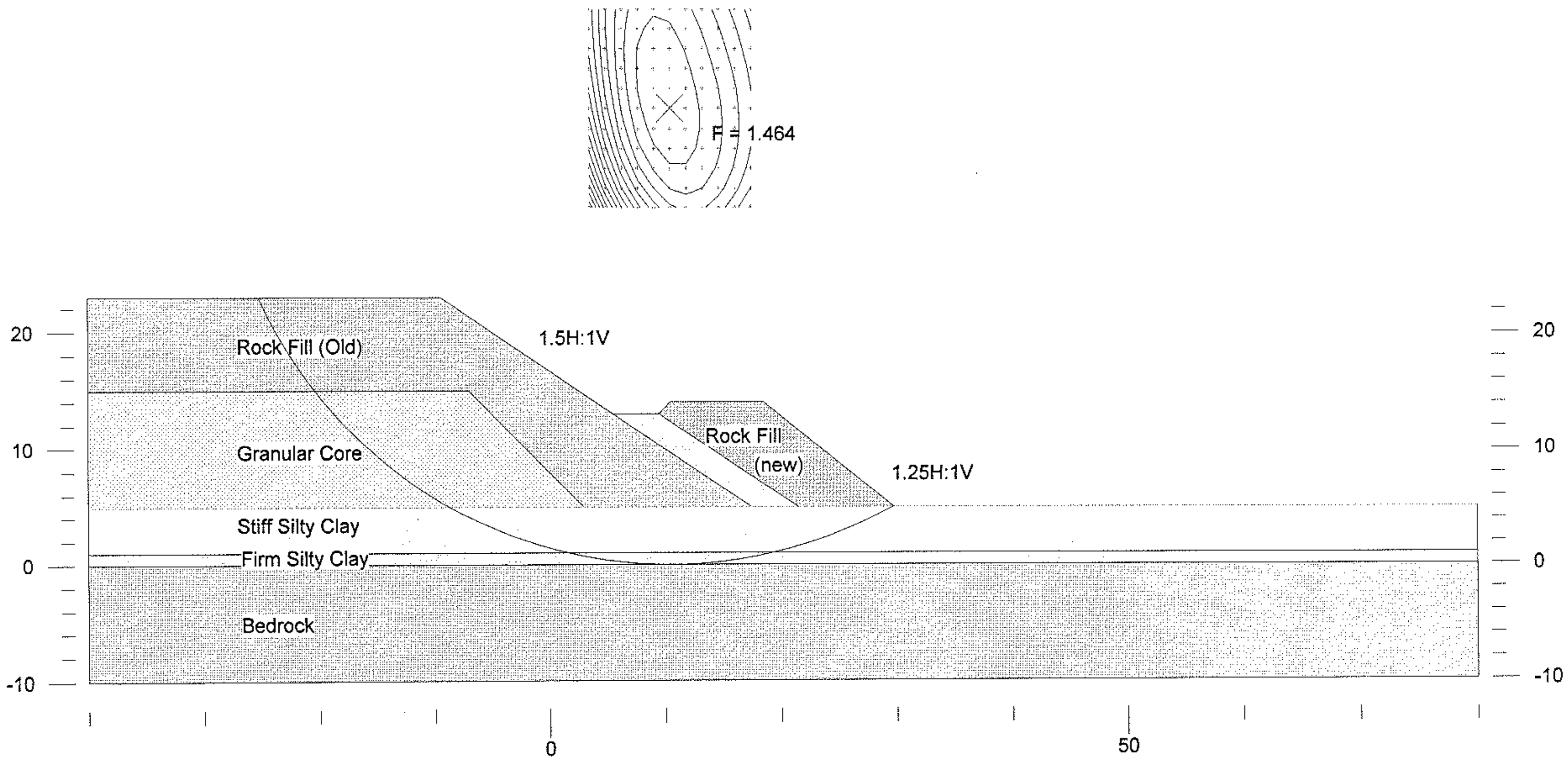
FIGURE FU5

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Rock Fill (new)	19	0	42	0	0
Rock Berm	19	0	42	0	0
Rock Fill (old)	19	0	42	0	0
Granular Core	22	0	35	0	0
Stiff Silty Clay	19	70	0	0	1
Firm Silty Clay	18	35	0	0	1
Bedrock	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-1351-82
Hwy 17/417 Twinning, Arnprior
April 24, 2006
Stability of New Upper Access Road Embankment
FIGURE FU6 Undrained Analysis - 9 m high new road embankment

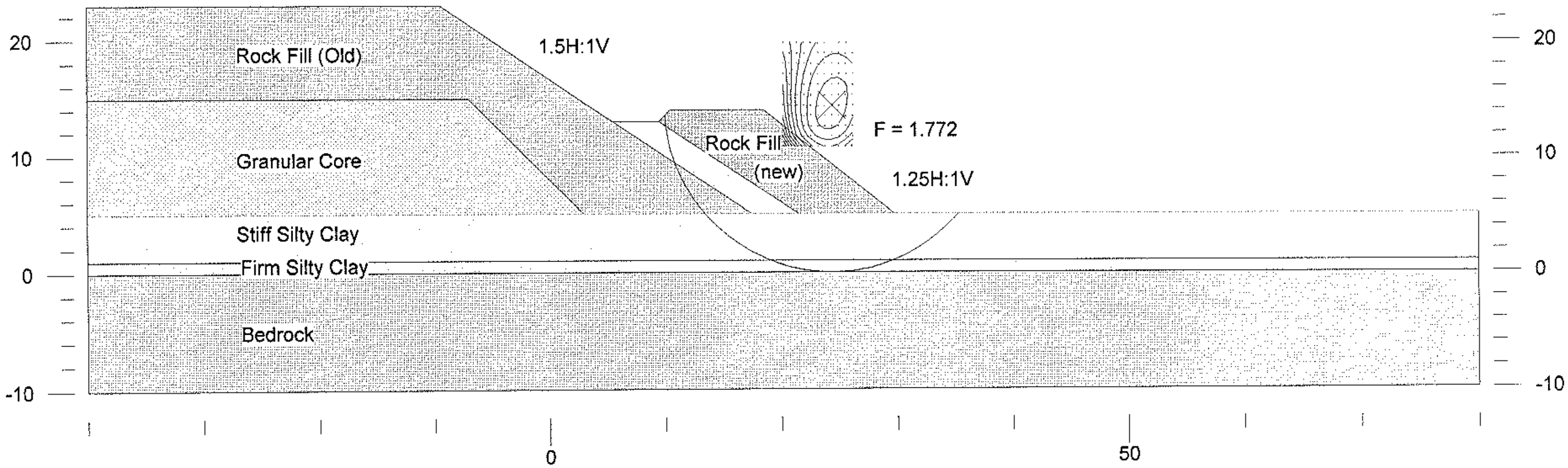


03/05/2006 11:43:08 AM C:\SPANG\THURB~12\19-135~4\FIGFU6.GSL Thurber Engineering Ltd. - Toronto F = 1.464

FIGURE FU6

Thurber Engineering Ltd. - Toronto
19-1351-82
Hwy 17/417 Twinning, Arnprior
April 24, 2006
Stability of New Upper Access Road Embankment
FIGURE FU7 Undrained Analysis - 9 m high new road embankment

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Rock Fill (new)	19	0	42	0	0
Rock Berm	19	0	42	0	0
Rock Fill (old)	19	0	42	0	0
Granular Core	22	0	35	0	0
Stiff Silty Clay	19	70	0	0	1
Firm Silty Clay	18	35	0	0	1
Bedrock	(Infinitely Strong)				



03/05/2006 11:41:52 AM C:\SPANG\THURB~12\19-135~4\FIGFU7.GSL Thurber Engineering Ltd. - Toronto F = 1.772

FIGURE FU7

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Gabion Wall 1	19	0	42	0	0
Rock Fill (new)	19	0	42	0	0
Stiff Silty Clay	19	0	29	0	1
Firm Silty Clay	18	0	27	0	1
Bedrock	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-1351-82
Hwy 17/417 Twinning, Arnprior
May 15, 2006
Stability of Gabion Wall 1 - Station 45+150
FIGURE FWA1 Drained Analysis - New Road Embankment

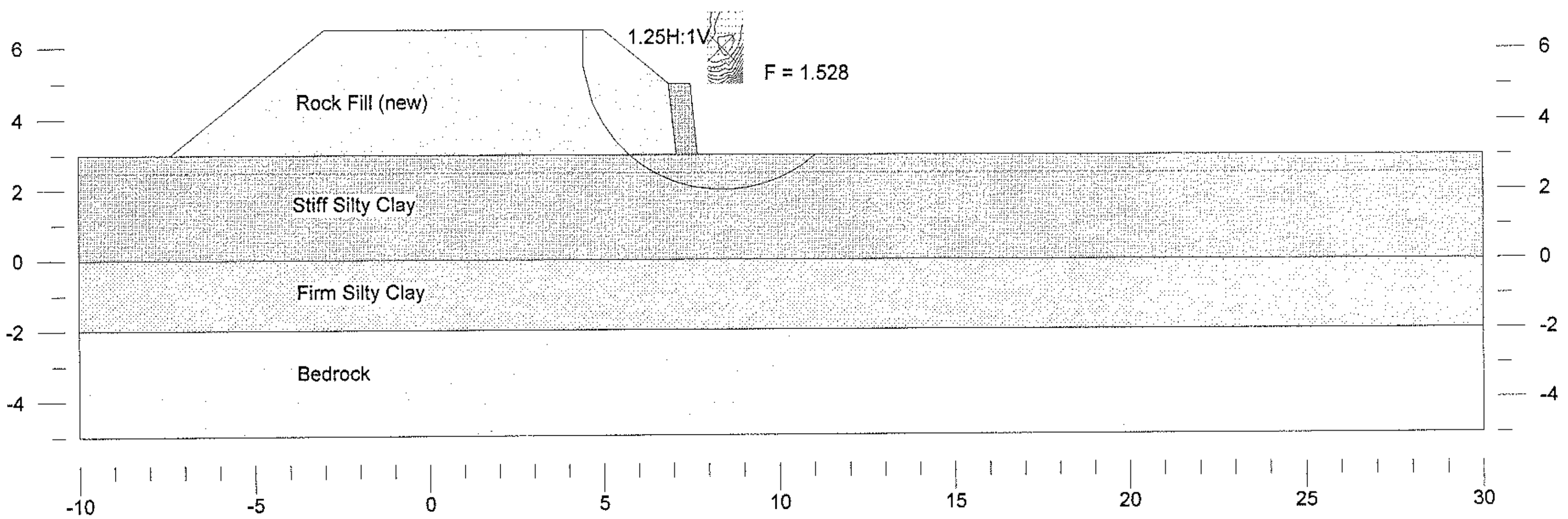


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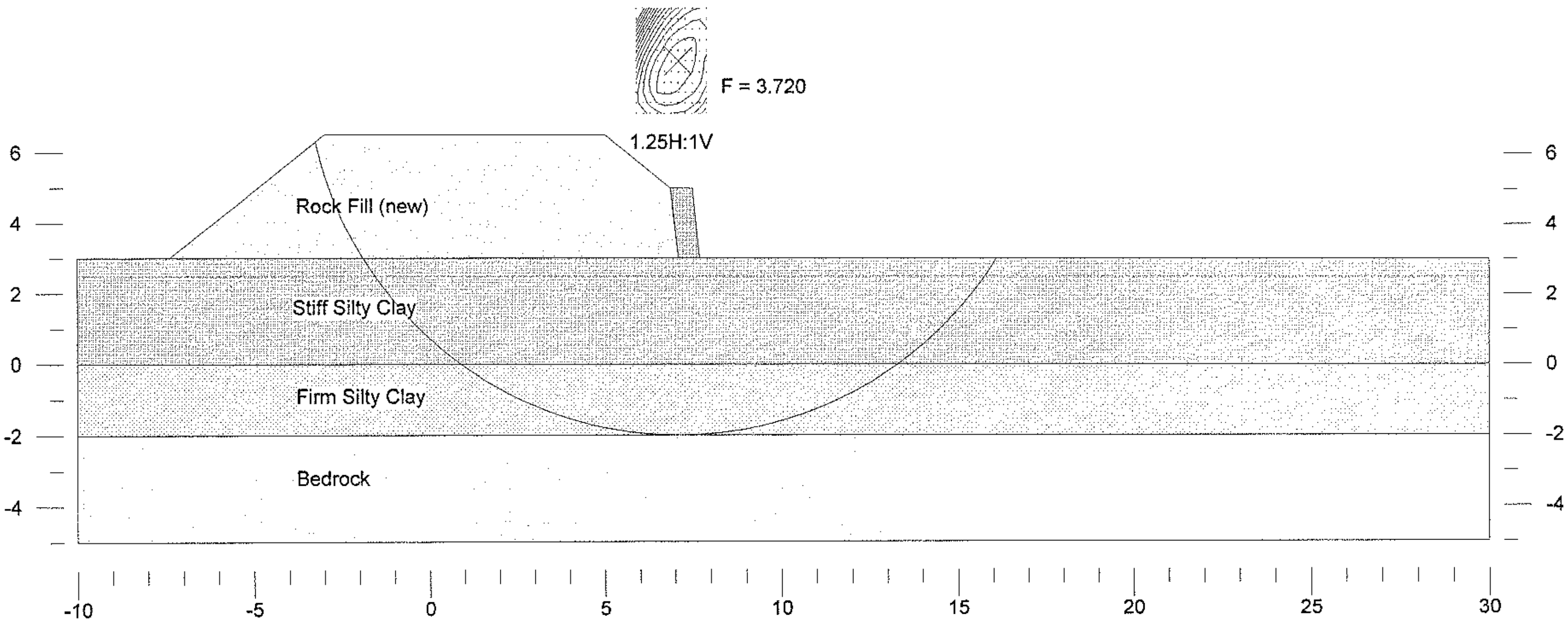
FIGURE FWA1

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Gabion Wall 1	19	0	42	0	0
Rock Fill (new)	19	0	42	0	0
Stiff Silty Clay	19	70	0	0	1
Firm Silty Clay	18	35	0	0	1
Bedrock	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-1351-82
Hwy 17/417 Twinning, Arnprior
May 15, 2006
Stability of Gabion Wall 1 - Station 45+150
FIGURE FWA2 Undrained Analysis - New Road Embankment



17/05/2006 11:21:26 AM C:\SPANG\THURB~12\19-135~4\FIGFWA2.GSL Thurber Engineering Ltd. - Toronto F = 3.720

FIGURE FWA2

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Gabion Wall 2	19	0	42	0	0
Gran. B Type II	22.8	0	35	0	0
Earth/Fill	20	0	30	0	0
Stiff Silty Clay	19	0	29	0	1
Firm Silty Clay	18	0	27	0	1

Thurber Engineering Ltd. - Toronto
19-1351-82
Hwy 17/417 Twinning, Arnprior
May 15, 2006
Stability of Gabion Wall 2 - Station 45+320
FIGURE FWA3 Drained Analysis - New Road Embankment

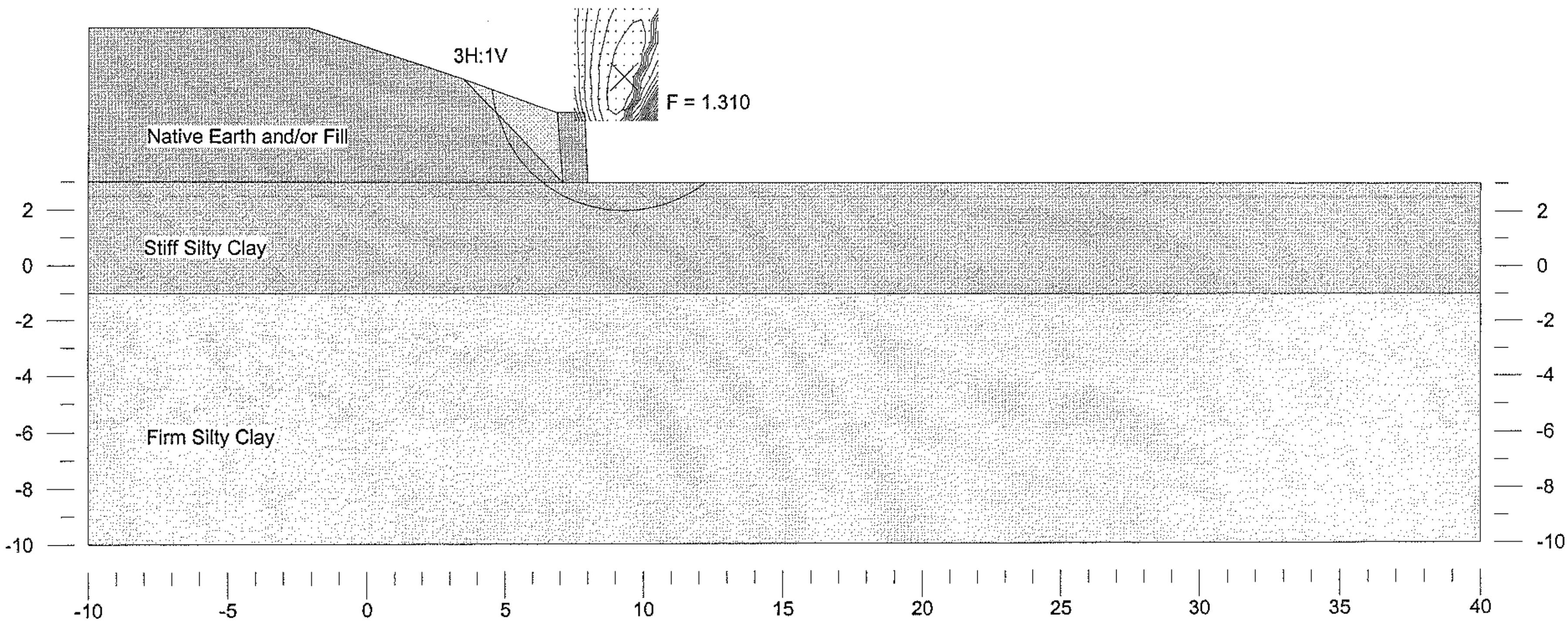


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FIGURE FWA3

Thurber Engineering Ltd. - Toronto
19-1351-82
Hwy 17/417 Twinning, Arnprior
May 15, 2006
Stability of Gabion Wall 2 - Station 45+320
FIGURE FWA4 Undrained Analysis - New Road Embankment

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Gabion Wall 2	19	0	42	0	0
Gran. B Type II	22.8	0	35	0	0
Earth/Fill	20	0	30	0	0
Stiff Silty Clay	19	70	0	0	1
Firm Silty Clay	18	35	0	0	1

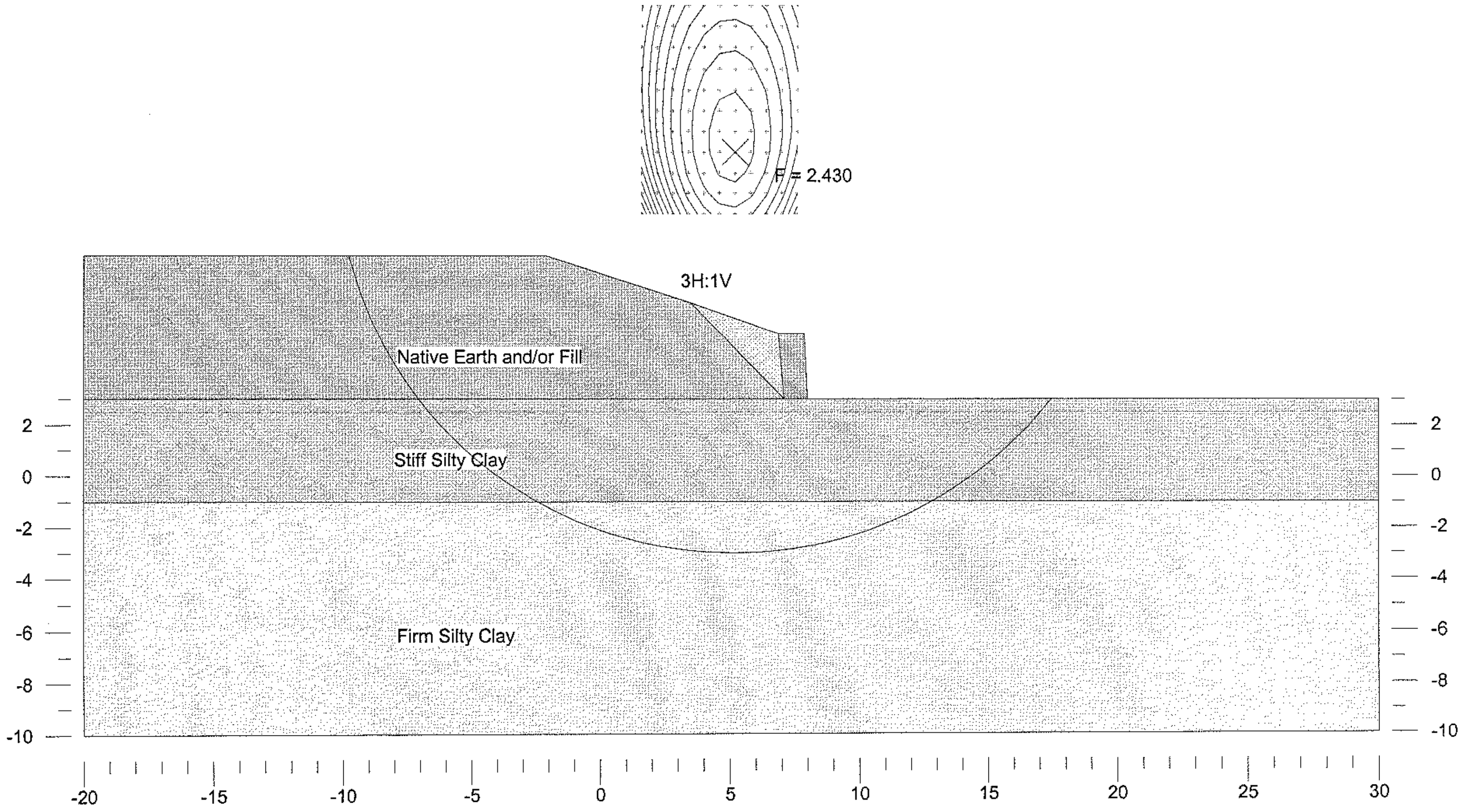

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FIGURE FWA4

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Rock Fill (new)	19	0	42	0	0
Rock Berm	19	0	42	0	0
Rock Fill (old)	19	0	42	0	0
Granular Core	22	0	35	0	0
Stiff Silty Clay	19	0	29	0	1
Firm Silty Clay	18	0	27	0	1
Bedrock	(Infinitely Strong)				

Seismic coefficient = 0.145

Thurber Engineering Ltd. - Toronto
19-1351-82
Hwy 17/417 Twinning, Arnprior
April 24, 2006
Seismic Stability of New Upper Access Road Embankment
FIGURE FS1 Drained Analysis - 9 m high new road embankment

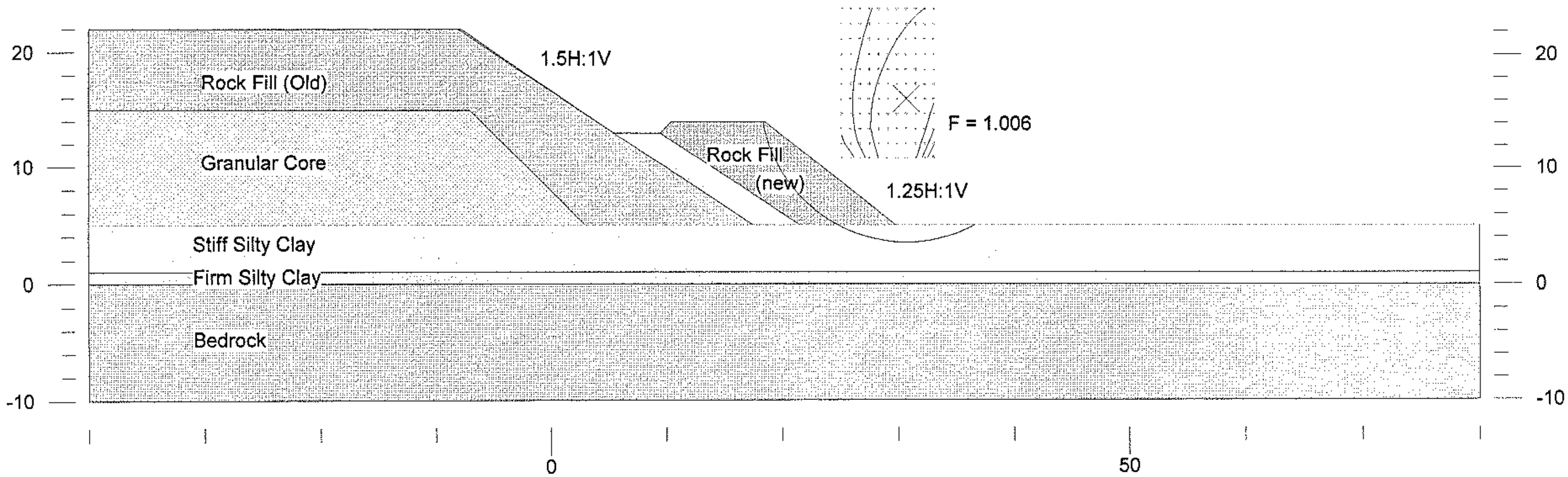

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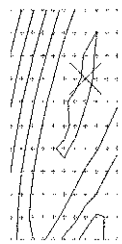
FIGURE FS1

Thurber Engineering Ltd. - Toronto
19-1351-82
Hwy 17/417 Twinning, Arnprior
April 24, 2006
Seismic Stability of New Upper Access Road Embankment
FIGURE FS2 Drained Analysis - 9 m high new road embankment

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Rock Fill (new)	19	0	42	0	0
Rock Berm	19	0	42	0	0
Rock Fill (old)	19	0	42	0	0
Granular Core	22	0	35	0	0
Stiff Silty Clay	19	0	29	0	1
Firm Silty Clay	18	0	27	0	1
Bedrock	(Infinitely Strong)				

Seismic coefficient = 0.215


F = 1.008

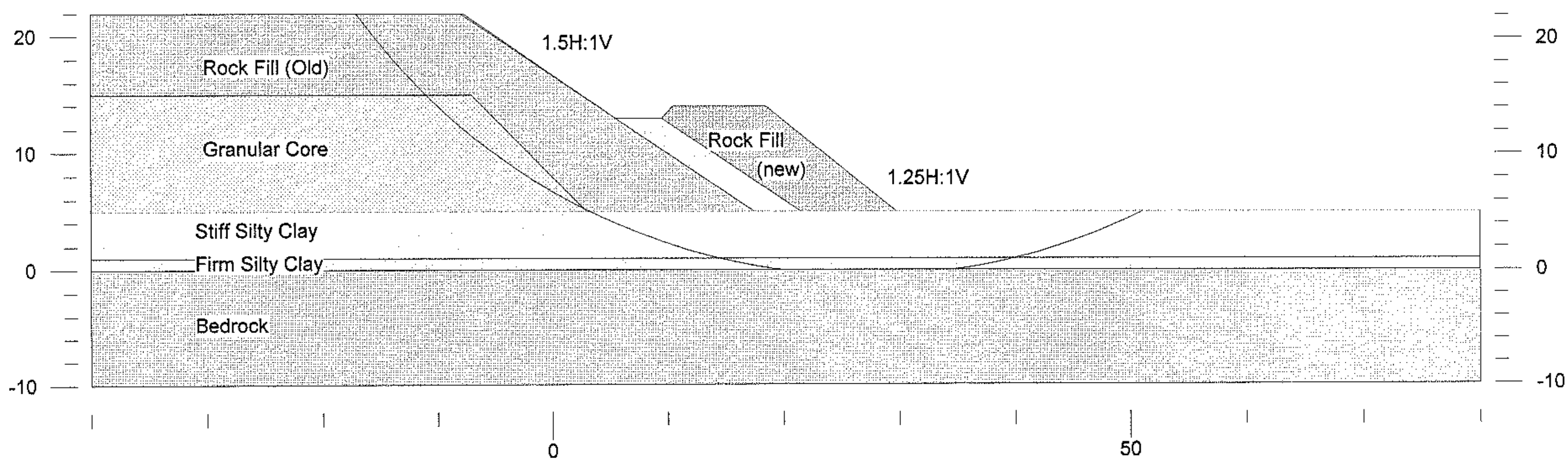


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FIGURE FS2

Thurber Engineering Ltd. - Toronto
19-1351-82
Hwy 17/417 Twinning, Arnprior
May 15, 2006
Seismic Stability of Gabion Wall 1 - Station 45+150
FIGURE FS3 Drained Analysis - New Road Embankment

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Gabion Wall 1	19	0	42	0	0
Rock Fill (new)	19	0	42	0	0
Stiff Silty Clay	19	0	29	0	1
Firm Silty Clay	18	0	27	0	1
Bedrock	(Infinitely Strong)				

Seismic coefficient = 0.245

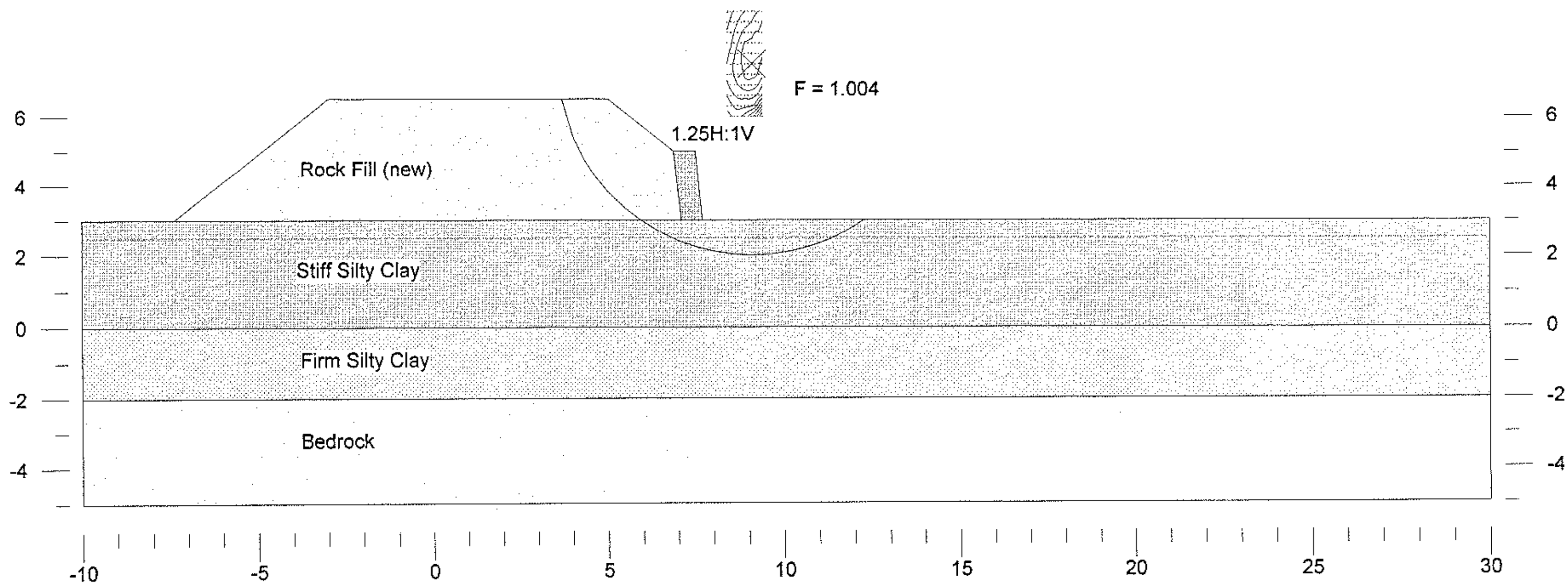

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FIGURE FS3

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Gabion Wall 2	19	0	42	0	0
Gran. B Type II	22.8	0	35	0	0
Earth/Fill	20	0	30	0	0
Stiff Silty Clay	19	0	29	0	1
Firm Silty Clay	18	0	27	0	1

Seismic coefficient = 0.155

Thurber Engineering Ltd. - Toronto
19-1351-82
Hwy 17/417 Twinning, Arnprior
May 15, 2006
Seismic Stability of Gabion Wall 2 - Station 45+320
FIGURE FS4 Drained Analysis - New Road Embankment

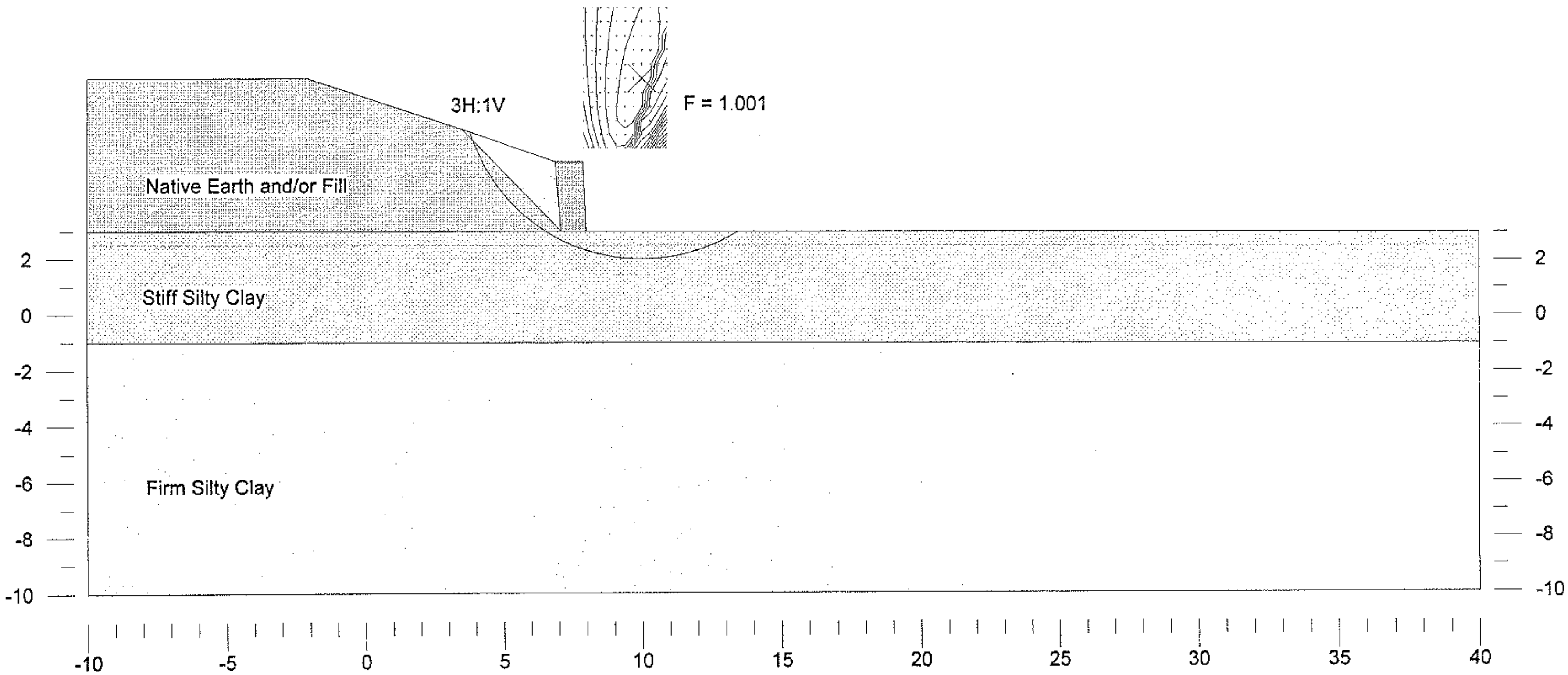

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FIGURE FS4