FOUNDATION INVESTIGATION AND DESIGN REPORT
SWAMP CROSSINGS AND HIGH FILL EMBANKMENTS
HIGHWAY 69 FOUR-LANING
FROM THE SOUTH JUNCTION OF HIGHWAY 529 NORTHERLY 15 KM
G.W.P. 5076-06-00
SOUTH SECTION – HIGHWAY 529 TO NAISCOOT LAKE
VOLUME 1 OF 2

Geocres Number: 41H-96

Report to

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H:\19\5161\21 Hwy69 N of Hwy529\Reports & Memos\Embankments\Section 1 - HWY 529 to Naiscoot\Swamp Crossings & High Fills South Section Final FIDR.doc

TABLE OF CONTENTS

VOLUME 1 of 2

THURBER

Appendices

VOLUME 2 of 2

Appendices B to J include:

- Record of Borehole Sheets
- Laboratory Test Results
- Borehole Locations and Soil Strata Drawings

FOUNDATION INVESTIGATION AND DESIGN REPORT
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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a foundation investigation conducted for the proposed swamp crossings and high fill embankments required along a section of the Highway 69 four-laning project extending from Highway 529 northerly approximately 3.8 km to north of Naiscoot Lake.

The report is the first of two reports addressing a larger section of the four-laning project extending from the south junction of Highway 529 northerly for 15 km in the Townships of Harrison and Wallbridge, Ontario. The report deals with the south part of this section; the remaining north part of the section is dealt with in a separate report.

The purpose of the investigation was to explore the subsurface conditions at sites where embankments or swamp crossings are proposed and, based on the data obtained, to provide record of borehole sheets, borehole location plans, stratigraphic profiles, laboratory test results, and a generalized description of the subsurface conditions at each location. This information provides a model of the anticipated geotechnical conditions influencing design and construction of the swamp crossings and high fill embankments.

Thurber carried out the investigation as a sub-consultant to MMM Group Limited (MMM) under the Ministry of Transportation Ontario (MTO) Agreement Number 5006-E-0030.

2 SITE DESCRIPTION

Highway 69 in the study section is currently a two lane undivided roadway. The proposed four-lane alignment will run roughly parallel to the existing alignment, with a new median centreline approximately 130 m west of the current alignment at the south project limit, before crossing to the east side and running approximately 70 m to the east. Both northbound and southbound lanes will be on new alignment in this section.

The roadway corridor typically has a rolling topography with frequent bedrock outcrops of generally low relief, separated by low-lying swamp areas, water bodies, and small streams. In general, the area is heavily wooded except in swamp areas.

The site lies within the physiographic region known as the Georgian Bay Fringe, characterized by very shallow soils and bare rock knobs and ridges. Where present, the overburden materials consist of sand, silt and clay. Recent organic deposits of peat and muck occur in abundance in bedrock hollows and valleys. The area is underlain by strongly foliated and highly to intermediately deformed rocks of Precambrian age, primarily migmatitic rocks and gneisses.

The locations and existing conditions at each section of swamp and high fill embankment investigated during the current study are summarized below:

Highway 69 NBL & SBL, Sta. 17+425 to 17+560 – Undulating treed area bordered by rock outcrops and a small stream. Embankment heights of up to 9.4 m are planned.

Highway 69 NBL & SBL, Sta. 17+900 to 18+000 – Low-lying, partially treed swamp area bordered on the north and south by rock outcrops. Embankment fill heights will be less than 2.0 m.

Highway 69 NBL, Sta. 18+180 to 18+230 – Roadside clearing between existing Highway 69 and rock outcropping. Fill heights will be less than 1.5 m.

Naiscoot Access Road, Sta. 10+555 to 10+590 – Relatively small marsh. Fill heights of 2.0 to 2.5 m are planned.

Highway 69 NBL & SBL, Sta. 18+240 to 18+500 – Relatively large, partially treed swamp area through which the existing Highway 69 crosses. New embankment fill heights will be up to 3.0 m.

Highway 69 SBL, Sta. 18+890 to 18+990 – Small depression bordered by rock outcropping, on east side of existing Highway 69. Fill heights will be up to 9.1 m.

Highway 69 NBL & SBL, Sta. 19+285 to 19+350 – Elongated body of water bordered on the north and south sides by steeply sloping bedrock outcrops. Embankment heights will range from 5.0 to 8.6 m.

Highway 69 SBL, Sta. 19+455 to 19+555 - Marshy area adjacent to east side of existing Highway 69. Fill heights will be up to 2.3 m.

Highway 69 NBL & SBL, Sta. 19+680 to 19+870 – Broad marsh with areas of open water. Fill heights of 3.5 to 6.0 m are planned.

3 SITE INVESTIGATION AND FIELD TESTING

Thurber carried out site investigation and field testing at each swamp crossing and high fill location identified in the Terms of Reference during the period January 27 to March 28, 2009 and on August 25, 2009. The limits of the study sections were adjusted during the course of the study to accommodate alignment changes, revised stationing and field observations. Following the initial investigations, the collected subsurface data and detailed contour plans of the corridor were reviewed, a walkover survey of the proposed alignment was carried out, and engineering analyses were conducted. Supplementary field exploration was then conducted in November 2010, February 2011 and June 2012 to investigate additional or extended areas of swamp identified by the data review, visual observations and/or engineering assessment requirements.

The site investigation and field testing consisted of drilling and sampling boreholes, advancing dynamic cone penetration tests (DCPT), and visual observation or manual excavation at locations where the bedrock surface was either exposed or at very shallow depth. All boreholes and DCPTs were advanced to refusal on probable bedrock.

In general, the boreholes were positioned along the centreline of the embankments/swamp crossings at longitudinal intervals of 25 m for sections extending 250 m or less, and 50 m for sections longer than 250 m. Halfway between these centreline boreholes, one borehole was advanced at the embankment toe location and one DCPT was conducted at the opposite embankment toe location, alternating from side to side.

In addition, subsurface information obtained from the concurrent foundation investigation for design of any structural culverts located within swamp sections has been incorporated into this report. These boreholes are identified by a 'C' series designation.

A summary of the locations and depths of the boreholes and DCPTs carried out in each of the study areas is provided in Table A1, Appendix A. The approximate locations of the boreholes and DCPT tests are shown on the Borehole Locations and Soil Strata Drawings in Appendices B to J.

The borehole and DCPT locations (stations and offsets from centreline) were established by Thurber relative to centreline staking by MMM Group Limited. Ground elevations at the test locations were approximated from detailed topographic plans provided by MMM Group.

Prior to commencement of drilling, utility clearances were obtained for all borehole and DCPT locations. Road occupancy permits were obtained for boreholes drilled on the existing Highway 69 platform. A permit was obtained from the Department of Fisheries and Oceans (DFO) prior to investigation within the water body at Station 19+285 to 19+350.

In general, the boreholes were advanced using continuous flight hollow stem augers powered by a CME-55 track-mounted drill rig. Wash-boring methods with casing and tripod were employed at locations where work was conducted on ice. A CME-75 truck-mounted drill rig was used for boreholes drilled on the existing Highway 69 platform. Portable split spoon sampling equipment driven with a 22.7 kg hammer was used to advance supplementary explorations in November 2010 and June 2012.

Samples were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT) in the overburden soils. Where firm to soft cohesive soils were encountered, samples were also obtained using a thin-walled (Shelby) tube sampler. In situ vane shear testing was carried out to assess the undrained shear strength of soft to firm cohesive deposits.

Groundwater conditions in the open boreholes were observed throughout the drilling operations. Standpipe piezometers were installed in selected boreholes to monitor groundwater levels. The standpipe piezometers consisted of 19 mm diameter PVC pipe with a 1.5 m slotted tip enclosed in filter sand. A bentonite seal was placed above the filter sand and the remainder of the borehole was backfilled with bentonite/grout to the ground surface. Details of the piezometer installations are shown on the Record of Borehole sheets in Appendices B to J and summarized in Table A2, Appendix A.

Boreholes without piezometer installations were backfilled with bentonite/grout and/or auger cuttings upon completion. Completion of the boreholes and standpipe piezometers was carried out in accordance with the requirements of O. Reg. 903 (as amended by O. Reg. 372/07).

A member of Thurber's technical staff supervised the drilling and sampling operations on a full time basis. The supervisor logged the boreholes and processed the recovered soil samples for transport to Thurber's laboratory for further examination and testing.

4 LABORATORY TESTING

The recovered soil samples were subjected to Visual Identification (VI) and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets in Appendices B to J.

Selected samples were also subjected to gradation analysis (sieve and hydrometer) and Atterberg Limits testing where appropriate. The results of the testing program are shown on the Record of Borehole sheets and figures in the respective appendices.

Thin wall tube samples were also selected for Oedometer and Consolidated-Undrained (CU) Triaxial Testing.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

Details of the encountered soil stratigraphy are presented on the Record of Borehole sheets and the Borehole Locations and Soil Strata Drawings in Appendices B to J of this report. A general description of the stratigraphy based on the conditions encountered in the boreholes is given in this section. However, the factual data presented in the borehole logs takes precedence over this general description and interpretation of the site conditions. It must be recognized that soil conditions may vary between and beyond borehole locations.

The specific conditions encountered at individual sites vary. Generalized descriptions of the individual strata at each swamp crossing and high fill embankment are presented below.

5.1 Highway 69 NBL & SBL, Sta. 17+425 to 17+560 Harrison (Appendix B)

General

The site stratigraphy consists of bedrock outcrops, a discontinuous veneer of organic material overlying shallow bedrock, and organic material over relatively thin deposits of sand, sandy silt and/or clayey silt underlain by probable bedrock.

Organics

A 50 to 300 mm thick veneer of organic material was encountered above bedrock in Boreholes 01-01 to 01-04R and 01-06, and over sand in Boreholes 01-07R to 01-16. The thickness of organic material may vary between and beyond the borehole locations.

Sand to Silty Sand

Brown to grey sand to silty sand was contacted below the organic layer in Boreholes 01-07R to 01-16, as well as below clayey and sandy silts in Boreholes 01-09 and 01-10L. The thickness of the sand deposits ranged from 0.1 to 1.4 m. The lower boundary was at Elev. 193.9 to 199.4 m for the upper layer and Elev. 191.2 to 191.9 m in the lower layers in Boreholes 01-09 and 01-10L.

The SPT ‘N’ values of the sand ranged from 3 to 36 blows/0.3 m of penetration indicating a very loose to dense relative density. The natural moisture contents generally ranged from 10 to 21%.

Grain size distribution curves from four sand samples are presented on the Record of Borehole sheets and on Figure B1 of Appendix B. The results are summarized as follows:

Gravel %	1 to 8
Sand %	61 to 80
Silt %	14 to 26
Clay %	2 to 11

Clayey Silt to Silty Clay

A layer of brown to grey clayey silt to silty clay, some sand to sandy, was encountered below the sand in Boreholes 01-09 to 01-11R. The thickness of the silt/clay layer ranged from 1.0 to 1.4 m. The lower boundary was at depths of 1.8 to 2.2 m (Elev. 193.1 to 195.5 m).

SPT 'N' values obtained in the silt/clay typically ranged from 11 to 15 blows/0.3 m, indicating a stiff consistency. One value of 30 blows/0.3 m (very stiff to hard) was recorded in Borehole 01-11R. Moisture contents ranged from 17 to 39%.

The results of grain size distribution analyses conducted on samples of the clay/silt are presented on the Record of Borehole sheets and on Figure B2 of Appendix B. Atterberg Limits test results are presented on Figure B3. The results are summarized as follows:

Gravel %	0
Sand %	31 to 50
Silt %	35 to 49
Clay %	15 to 30
Liquid Limit	30
Plastic Limit	13

The above results show that the clayey silt/silty clay is of low plasticity with a group symbol of CL.

Sandy Silt

A 0.5 m thick layer of grey sandy silt was encountered below the clayey silt locally in Borehole 01-09. The lower boundary of this layer was at 2.4 m depth (Elev. 192.6 m).

An SPT 'N' value of 19 blows/0.3 m was obtained in the sandy silt, indicating a compact condition. A moisture content of 14% was obtained.

Bedrock

Bedrock/probable bedrock was observed surficially or contacted below the organic layer and sand deposits at depths of up to 3.8 m at all borehole and DCPT locations. The bedrock surface at the borehole and DCPT locations ranged from approximately Elev. 191.2 to 199.8 m. The depths and elevations of the probable bedrock surface at the borehole locations are summarized in Table 5.1.

Table 5.1 – Depth/Elevation of Probable Bedrock

Borehole	Probable Bedrock Surface	
	Depth below Ground Surface (m)	Elevation (m)
01-01	0.1	198.4
01-02L	0.3	196.8
01-03	0.1	193.6
01-04R	0.1	194.7
01-05	0.0	195.1
01-06	0.2	193.9
01-07R	1.1	193.9
01-08	1.1	194.3
01-09	3.8	191.2
01-10L	3.1	191.9
01-11R	2.2	195.5
01-12	0.4	196.0
01-13	1.4	194.5
01-14R	0.4	197.0
01-15	0.2	198.2
01-16	0.2	199.4
D01-01R	0.1	194.0
D01-02L	0.3	193.9
D01-03L	0.7	194.1
D01-04R	0.2	195.1
D01-05R	0.6	194.6
D01-06L	0.6	197.2
D01-07R	0.9	199.8

Groundwater Conditions

Water levels were observed in the boreholes during and upon completion of drilling. The water levels observed in the boreholes upon completion of drilling are summarized in Table 5.2. Water was not observed in the remaining boreholes.

Table 5.2 – Water Level Observations

Borehole	Date	Water Level		Comment
		Depth (m)	Elev. (m)	
01-09	Nov 9, 2010	0.4	194.6	Open borehole
01-10L	Nov 10, 2010	0.3	194.7	Open borehole
01-11R	Nov 9, 2010	0.3	197.4	Open borehole
01-13	Nov 9, 2010	0.9	195.0	Open borehole

The above values are short-term readings and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

5.2 Highway 69 NBL & SBL, Sta. 17+900 to 18+000 Harrison (Appendix C)

General

This swamp site was covered by up to 1.1 m depth of ice and water at the time of the field investigation. Below the water, the subsurface stratigraphy consisted of a relatively thick deposit of peat, underlain by bedrock and by silty sand within a depression in the bedrock surface.

Peat and Organics

A deposit of brown to black fibrous peat was encountered below the ice and water in all boreholes except Borehole 02-01 located outside of the swamp. A 450 mm thick layer of organic material was encountered over probable bedrock in Borehole 02-01.

The thickness of the peat deposit ranged from 0.6 to 5.1 m at the borehole locations. The lower boundary of the peat deposit varied from Elev. 199.9 to 204.5 m. The peat thickness may vary between and beyond the borehole locations.

SPT 'N' values obtained in the peat were typically 0 to 1 blows/0.3 m of penetration, with values of 4 and 5 also recorded. The natural moisture contents varied widely from 153 to 1173%, typically in the order of 300 to 800%.

Sand and Silty Sand

A 0.6 m thick layer of black to dark brown sand was contacted below the peat in Borehole 02-02L at 1.4 m. The split spoon sampler sank under self-weight in this layer, indicating a very loose condition. A moisture content of 50% was measured, reflecting the presence of peaty organics.

Native grey to brown sand to silty sand was contacted below the peat at depths of 3.8 to 5.3 m below the ice surface (2.7 to 5.1 m below the ground surface) in Boreholes 02-03, 04-02L, 04-03, 04-06R and 04-07L, and below the sand at 2.0 m depth (1.2 m below the ground surface) in Borehole 02-02L. A second sand layer was encountered below a sandy silt stratum in Borehole 04-07L. The thickness of the sand/silty sand deposit was 0.6 to 3.8 m. The upper boundary ranged from Elev. 199.9 to 203.9 m, and the lower boundary was at Elev. 196.8 to 202.9 m.

The SPT 'N' values of the silty sand ranged from 0 to 10 blows per 0.3 m of penetration indicating a typically loose to very loose relative density. 'N' values of 21 and 13 blows/0.3 m (compact) were obtained in Boreholes 04-06R and 04-07L. The natural moisture contents generally ranged from 12 to 38%.

Grain size distribution curves from five sand samples are presented on the Record of Borehole sheets and on Figure C1 of Appendix C. The results are summarized as follows:

	Four samples	One sample
Gravel %	0 to 2	0
Sand %	55 to 69	99
Silt %	24 to 31	1
Clay %	3 to 17	

Boreholes 02-02L, 02-03 and 04-03 were terminated below the sand layer at 3.0 to 9.1 m depth (Elev. 196.8 to 202.9 m), upon auger refusal on probable bedrock. Boreholes 04-06R and 04-07L were terminated upon refusal to the portable equipment in the sand at 5.8 m depth. The sand became gravelly at 8.1 m depth and probable bedrock was encountered at 8.4 m depth (Elev. 197.5 m) in Borehole 04-02L.

Silty Clay

A 0.8 m thick layer of silty clay was contacted below the sand in Borehole 04-07L at 3.8 m depth (Elev. 201.0 m). The depth to the base of the silty clay layer was 4.6 m (Elev. 200.2 m).

The split spoon sampler sank under self-weight in the silty clay, indicating a very soft consistency. A moisture content of 47% was obtained in a single sample.

Sandy Silt

Sandy silt containing some clay was contacted below the silty clay at 4.6 m depth in Borehole 04-07L. The sandy silt layer was 0.6 m thick with a lower boundary at 5.2 m depth (Elev. 199.6 m).

An SPT 'N' value of 1 blow/0.3 m was obtained in the sandy silt, indicating a very loose condition. A natural moisture content of 38% was measured.

A grain size distribution curve of a sandy silt sample is presented on the Record of Borehole sheet and on Figure C2 of Appendix C. The results are summarized as follows:

Gravel %	0
Sand %	34
Silt %	47
Clay %	19

Bedrock

All but two boreholes and DCPTs were terminated upon refusal on probable bedrock, contacted at depths ranging from 0.5 m to 12.6 m below the ice surface (0.5 to 11.5 m below the ground surface), Elev. 193.5 to 206.2 m. The depths and elevations of the probable bedrock surface at the borehole locations are summarized in Table 5.3.

Table 5.3 – Depth/Elevation of Probable Bedrock

Borehole	Probable Bedrock Surface	
	Depth below Ground Surface (m)	Elevation (m)
02-01	0.5	206.2
02-02L	2.2	202.9
02-03	8.3	196.8
02-04R	1.4	203.6
02-05	0.9	203.9
02-07L	1.1	203.7
02-08	4.3	200.5
D02-01R	2.3	203.5
D02-02L	8.0	196.9
D02-03	9.1	196.8
04-01	2.4	202.6
04-02L	7.6	197.5
04-03	5.4	199.5
04-04R	1.6	203.3
04-05	1.7	203.2
04-06R	-	-
04-07L	-	-
D04-01R	11.5	193.5
D04-02L	5.7	199.2
D04-05	1.7	203.2

Groundwater Conditions

In general, the boreholes were drilled from the ice surface approximately 0.6 to 0.8 m above the ground surface. Standpipe piezometers were installed in Boreholes 02-03 and 04-02L to monitor water levels after completion of drilling. The depth of ice and water at the borehole locations and the water levels measured in the piezometers are summarized in Table 5.4.

Table 5.4 – Water Level Observations

Borehole	Date	Water Level		Comment
		Depth (m)	Elev. (m)	
02-02L	Jan 27, 2009	0.8 ags	205.9	Drilling on ice
02-03	Jan 27, 2009	0.8 ags	205.9	Drilling on ice
	Feb 19, 2009	0.0	205.1	In piezometer
	Apr 21, 2009	0.0	205.1	In piezometer
	Jun 5, 2009	0.0	205.1	In piezometer
	Aug 20, 2009	0.0	205.1	In piezometer
	Aug 25, 2009	0.0	205.1	In piezometer
	Oct 27, 2009	0.0	205.1	In piezometer
	Nov 20, 2009	0.0	205.1	In piezometer
02-04R	Jan 27, 2009	0.8 ags	205.8	Drilling on ice
02-05	Jan 27, 2009	0.8 ags	205.6	Drilling on ice
02-07L	Nov 10, 2010	0.0	204.8	Open borehole
02-08	Nov 10, 2010	0.0	204.8	Open borehole
04-01	Jan 27, 2009	0.8 ags	205.8	Drilling on ice
04-02L	Jan 28, 2009	0.8 ags	205.9	Drilling on ice
	Apr 21, 2009	0.0	205.1	In piezometer
	Jun 5, 2009	0.0	205.1	In piezometer
	Aug 20, 2009	0.0	205.1	In piezometer
	Aug 25, 2009	0.0	205.1	In piezometer
	Oct 27, 2009	0.0	205.1	In piezometer
	Nov 20, 2009	0.0	205.1	In piezometer
04-03	Jan 28, 2009	0.7 ags	205.6	Drilling on ice
04-04R	Jan 28, 2009	0.8 ags	205.7	Drilling on ice
04-05	Jan 28, 2009	0.6 ags	205.5	Drilling on ice
04-06R	Nov 10, 2010	0.0	205.0	Open borehole
04-07L	Nov 10, 2010	0.0	204.8	Open borehole

ags = above ground surface

The above values are short-term observations. The surface water depth and depths to groundwater will vary depending upon seasonal fluctuations, rainfall patterns and swamp outlet conditions such as presented by beaver dams. In particular, water levels may be higher after the spring snowmelt or periods of heavy rainfall.

5.3 Highway 69 NBL, Sta. 18+180 to 18+230 Harrison (Appendix D)

General

This site consists of a localized deposit of peat overlying silty clay, sand and silt, and sand, bordered by a rock outcrop on the east and the existing highway platform on the west.

Peat and Organics

A deposit of brown to black fibrous peat was encountered surficially in boreholes 03-03 and 03-05. The peat layer was 0.9 and 0.6 m thick in these respective boreholes, with a lower boundary at Elevation 205.0 and 205.5 m. SPT 'N' values of 2 and 9 blows/0.3 m of penetration were obtained in the peat layer, indicating a very soft to stiff consistency. A natural moisture content of 182% was measured in one sample.

A 75 to 100 mm thick layer of organic material was encountered over shallow bedrock in Boreholes 03-01 and 03-02L. Moisture contents of 167 and 478% were measured in this material.

The thickness of peat and organics may vary between and beyond the borehole locations.

Silty Clay

A 0.5 m thick layer of silty clay was contacted below the peat in Borehole 03-03 at 0.9 m depth (Elev. 205.0 m). The depth to the base of the silty clay layer was 1.4 m (Elev. 204.5 m).

The split spoon sampler sank under self-weight in the silty clay, indicating a very soft consistency. A moisture content of 60% was obtained in a single sample.

Sand and Silt

Sand and silt containing trace gravel and trace to some clay was contacted below the silty clay at 1.4 m depth in Borehole 03-03 and surficially in Borehole 03-04R. The sand and silt layer was 0.8 and 0.7 m thick with a lower boundary at 2.2 m depth (Elev. 203.7 m) in Borehole 03-03 and 0.7 m depth (Elev. 205.4 m) in Borehole 03-04R.

SPT 'N' values in the sand and silt layer were 0 and 13 blows/0.3 m in Boreholes 03-03 and 03-04R, respectively, indicating very loose and compact conditions. Natural moisture contents of 21 to 51% were measured.

A grain size distribution curve of a sand and silt sample is presented on the Record of Borehole sheets and on Figure D1 of Appendix D. The results are summarized as follows:

Gravel %	5
Sand %	54
Silt %	38
Clay %	3

Sand

Native grey sand containing some gravel and trace silt was contacted below the sand and silt layer in Borehole 03-03 at 2.2 m depth. Borehole 03-03 was terminated below the sand layer at 2.4 m depth (Elev. 203.5 m), upon auger refusal on probable bedrock.

An SPT 'N' value of 6 blows/0.3 m was obtained in the sand, indicating a loose relative density. The moisture content of the sand was 15%.

Bedrock

The boreholes and DCPTs were terminated at depths ranging from 0.1 to 2.4 m (Elev. 203.5 to 209.3 m) upon refusal on probable bedrock. The depths and elevations of the probable bedrock surface at the borehole locations are summarized in Table 5.5.

Table 5.5 – Depth/Elevation of Probable Bedrock

Borehole	Probable Bedrock Surface	
	Depth below Ground Surface (m)	Elevation (m)
03-01	0.1	207.2
03-02L	0.1	209.3
03-03	2.4	203.5
03-04R	0.7	205.4
03-05	0.6	205.5
D03-01R	1.0	205.0
D03-02L	1.8	205.0

Groundwater Conditions

Water levels were observed in the boreholes during and upon completion of drilling. A standpipe piezometer was installed in Borehole 03-03 to monitor water levels after completion of drilling. The water levels observed in the boreholes upon completion of drilling and subsequently measured in the piezometer are summarized in Table 5.6.

Table 5.6 – Water Level Observations

Borehole	Date	Water Level		Comment
		Depth (m)	Elev. (m)	
03-03	Jan 29, 2009	0.6	205.3	Open borehole
	Apr 21, 2009	0.0	205.9	In piezometer
	Jun 5, 2009	0.0	205.9	In piezometer
	Aug 20, 2009	0.0	205.9	In piezometer
	Aug 25, 2009	0.0	205.9	In piezometer
	Oct 27, 2009	0.0	205.9	In piezometer
	Nov 20, 2009	0.0	205.9	In piezometer

The above values are short-term readings and seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

5.4 Naiscoot Access Road, Sta. 10+555 to 10+590 (Appendix E)

General

This site was covered by up to 0.8 m of ice and water at the time of the field investigation. Below the water, the subsurface stratigraphy consisted of thin deposits of peat and sand overlying shallow bedrock.

Peat

Black fibrous peat was encountered at the ground surface in Borehole 05-02L and below the water in Borehole 05-03. The peat deposit was 600 and 100 mm thick in Boreholes 05-02L and 05-03, respectively, with a lower boundary at Elev. 204.0 and 205.0 m. The peat thickness may vary between and beyond the borehole locations.

An SPT 'N' value of 3 blows/0.3 m was obtained in the peat, indicating a very soft consistency. The natural moisture content was 229%.

Sand

Native brown to black sand containing trace to some gravel and occasional peat was contacted below the water in Borehole 05-01, below the peat at 0.6 m depth in Borehole 05-02L, and surficially in Borehole 05-04.

The thickness of the sand layer ranged from 200 to 700 mm. Boreholes 05-01, 05-02L and 05-04 were terminated below the sand layer at depths of 0.4 to 1.4 m (Elev. 203.8 to 205.5 m) upon auger refusal on probable bedrock.

An SPT 'N' value of 7 blows/0.3 m was obtained in the sand, indicating a loose condition. Natural moisture contents ranged between 19 and 86%, reflecting the presence of organics.

Bedrock

The boreholes and DCPTs were terminated at depths ranging from 0.4 to 1.4 m below the ice/water surface, 0.0 to 0.8 m below the ground surface (Elev. 203.8 to 205.5 m) upon refusal on probable bedrock. The depths and elevations of the probable bedrock surface at the borehole locations are summarized in Table 5.7.

Table 5.7 – Depth/Elevation of Probable Bedrock

Borehole	Probable Bedrock Surface	
	Depth below Ground Surface (m)	Elevation (m)
05-01	0.7	204.4
05-02L	0.8	203.8
05-03	0.1	205.0
05-04	0.4	205.5
D05-01R	0.0	204.6

Groundwater Conditions

Three of the boreholes/DCPTs (05-01, 05-03 and D05-01R) were drilled from the ice surface approximately 0.6 to 0.8 m above the ground surface. The depth of ice and water at the borehole locations and the water levels observed during drilling are summarized in Table 5.8. Water was not observed in Borehole 05-04 during drilling.

Table 5.8 – Water Level Observations

Borehole	Date	Water Level		Comment
		Depth (m)	Elev. (m)	
05-01	Jan 28, 2009	0.7 ags	205.8	Drilling on ice
05-02L	Jan 28, 2009	0.0	204.6	Open borehole
05-03	Jan 28, 2009	0.6 ags	205.7	Drilling on ice
D05-01R	Jan 28, 2009	0.8 ags	205.4	Drilling on ice

The above values are short-term observations. The surface water depth and depths to groundwater will vary depending upon seasonal fluctuations, rainfall patterns and swamp outlet conditions such as presented by beaver dams. In particular, water levels may be higher after the spring snowmelt or periods of heavy rainfall.

5.5 Highway 69 NBL & SBL, Sta. 18+240 to 18+500 Harrison (Appendix F)

General

Much of this swamp site was covered by up to 0.8 m of ice and water at the time of the field investigation. Below the water, the subsurface stratigraphy consisted of a thick deposit of peat underlain by silty clay, sandy silt, silty sand and sand, overlying probable bedrock. A pavement structure and embankment fill was encountered in boreholes drilled on the existing Highway 69 platform.

Pavement Structure and Existing Embankment Fill

A pavement structure consisting of approximately 50 to 85 mm of asphalt overlying granular road base was encountered in Boreholes 08-01, 08-02L and 08-24 to 08-31 drilled on the existing Highway 69 shoulder.

The embankment fill immediately underlying the asphalt surface consists of brown sand containing trace gravel to gravelly, trace to some silt, trace clay and occasional cobbles and bedrock fragments. It graded to sand and gravel in Borehole 08-26. The sand fill extended to depths of 1.6 to 5.1 m (Elev. 205.4 and 202.1 m).

Rock fill requiring rock coring methods to penetrate was encountered below the sand fill in Boreholes 08-25, 08-26 and 08-28 to 08-31. The thickness of the rock fill layer ranged from 1.8 to 6.9 m at the borehole locations. The lower boundary of the rock fill was encountered at depths of 5.1 to 8.5 m (Elev. 202.0 to 198.5).

A further 0.4 to 0.5 m of sand fill was encountered below the rock fill in Boreholes 08-26 and 08-31. The lower boundary of this layer was at 6.6 and 7.0 m depth (Elev. 200.8 and 199.9).

SPT 'N' values obtained in the sand fill ranged from 4 to 30 blows/0.3 m, indicating a loose to compact condition. An 'N' value of 2 blows/0.3 m was obtained in Borehole 08-31 and below the rock fill in Borehole 08-31, indicating a very loose condition. Values of 50 blows for 50 and 100 mm of penetration recorded in Boreholes 08-27 and 08-28 reflect the presence of cobbles. Moisture contents ranged from 2 to 23%, generally increasing with depth.

The results of grain size distribution analyses conducted on eight fill samples are presented on the Record of Borehole sheets and on Figures F1 and F2 of Appendix F. The results are summarized as follows:

	Sand	Sand and Gravel
Gravel %	0 to 27	52
Sand %	64 to 94	45
Silt & Clay %	6 to 19	3

Peat

Black to dark brown fibrous peat was contacted below the water and/or at the ground surface in all boreholes drilled off of the roadway platform, and below the fill in all boreholes drilled on the highway shoulder except Boreholes 08-2L, 08-27 and 08-28.

The peat thickness ranged from 3.0 to 5.9 m in Boreholes 08-03 to 08-14L, 08-32 and 08-33 located east of existing Highway 69, from 0.1 to 0.9 m in Boreholes 08-15 to 08-18 located at the north end of this section, and from 1.2 to 4.3 m in Boreholes 08-19 to 08-23 located west of the highway. Where encountered below the existing embankment fill, the peat layer was 0.1 to 1.0 m thick, locally 3.1 m in Borehole 08-25.

The depth to the base of the peat ranged from 0.1 to 6.1 m (Elev. 199.6 m to 206.0 m) in the off-road boreholes, and from 2.9 to 9.4 m in the roadway boreholes. The peat thickness may vary between and beyond the borehole locations.

The split spoon sampler generally sank under self-weight in the peat ('N' value of 0), with 'N' values of 1 and 2 blows/0.3 m obtained locally, indicating a very soft consistency. 'N' values of 2 to 10 blows/0.3 m were recorded in the buried peat in Boreholes 08-24 and 08-25, reflecting compression of the peat under the fill. In situ vane shear tests indicated undrained shear strengths ranging from 8 to 28 kPa.

Natural moisture contents in the peat ranged from 83 to 1439%. Values ranged from 31 to 669% in samples recovered from below the existing embankment fill.

Sandy Silt to Silt and Sand

A layer of grey sandy silt to silt and sand containing trace to some clay was contacted below the peat in Boreholes 08-06L and 08-31. This layer was 1.1 to 1.7 m thick with a lower boundary at 4.6 and 9.0 m depth (Elev. 201.5 and197.9 m).

The results of a grain size distribution analysis conducted on a sample of the silt and sand are presented on the Record of Borehole sheet (Borehole 08-31) and on Figure F3 of Appendix F. The results are summarized as follows:

Gravel %	1
Sand %	58
Silt %	38
Clay %	3

Silty Clay

Native grey silty clay containing trace to some sand and trace gravel was contacted below the peat in Boreholes 08-03 to 08-05, 08-07 to 08-11, 08-14L, 08-30, 08-32 and 08-33, below the fill in Borehole 08-27, and below the sand/silt layer in Boreholes 08-06L and 08-31. The silty clay thickness ranged between 0.3 and 2.9 m in these boreholes. In Borehole 08-23, successive layers of silty clay, 0.7 to 1.0 m thick, were encountered within the peat, below the peat, and within an underlying sand stratum. A 0.1 m thick clay layer was also encountered within a sand layer at 10.2 m depth in Borehole 08-25.

The depth to the base of the various silty clay layers ranged from 2.6 to 10.3 m (Elev. 197.1 to 204.5 m).

In general, the split spoon sampler sank under the self-weight of the hammer in the silty clay, indicating a very soft consistency. 'N' values of 1 to 3 blows/0.3 m were recorded locally. SPT 'N' values of 8 and 14 blows/0.3 m were obtained in single tests conducted near the base of this deposit, indicating a stiff consistency. In situ vane testing indicated undrained shear strengths in the order of 12 to 28 kPa, typically about 20 kPa. The natural moisture contents ranged from 21 to 77%, typically about 40 to 70%, and locally up to 145% in Borehole 08-32.

The results of grain size distribution analyses conducted on samples of the silty clay are presented on the Record of Borehole sheets and on Figures F4 to F6 of Appendix F. Atterberg Limits test results are presented on Figure F9 of Appendix F. The results are summarized as follows:

Gravel %	0
Sand %	2 to 16
Silt %	25 to 47
Clay %	39 to 73
Liquid Limit	39 to 61
Plastic Limit	19 to 22

The above results show that the silty clay is of medium to high plasticity with group symbols of CI-CH.

Sand to Silty Sand

Grey sand to silty sand deposits were contacted below the peat and/or silty clay at depths of 1.2 to 9.6 m in Boreholes 08-01, 08-03, 08-05, 08-06L, 08-10L, 08-12R, 08-13, 08-16R, 08-19 to 08-25, 08-27 and 08-30 to 08-32. The thickness of the sand deposits ranged from 0.2 to 3.2 m. The depth to the base of the sand and silty sand deposits ranged from 2.2 to 10.7 m (Elev. 196.2 to 204.0 m).

In many cases, the split spoon sampler sank under the weight of the hammer ('N' = 0, very loose), however SPT 'N' values of up to 15 blows/0.3 m (compact) were obtained locally, and values of 28 and 35 (compact to dense) were also recorded. The natural moisture content typically ranged from 12 to 30%. Moisture contents of 38 to 127% were also measured, reflecting the presence of organics.

Grain size distribution curves of nine sand and silty sand samples are presented on the Record of Borehole sheets and on Figures F7 and F8 of Appendix F. The results of the laboratory testing are summarized as follows:

	Sand	Silty Sand
Gravel %	0 to 13	0 to 10
Sand %	65 to 95	45 to 64
Silt %	4 to 22	23 to 38
Clay %		3 to 17

Bedrock

Probable bedrock was contacted in all of the boreholes and DCPTs at depths of 0.1 to 10.7 m below the ground surface (Elev. 195.0 to 206.0 m). The depths/elevations are listed in Table 5.9.

Table 5.9 – Depth/Elevation of Probable Bedrock

Borehole	Probable Bedrock Surface	
	Depth below Ground Surface (m)	Elevation (m)
08-01	6.5	200.7
08-02L	2.1	205.2
08-03	7.6	198.0
08-04R	6.5	198.9
08-05	8.3	197.2
08-06L	9.4	196.2
08-07	6.8	198.8
08-08R	5.4	200.1
08-09	6.4	199.1
08-10L	8.2	197.3
08-11	7.1	198.3
08-12R	4.5	201.0
08-13	4.6	200.8
08-14L	3.9	201.5
08-15	0.9	203.5
08-16R	2.1	203.5
08-17	0.1	205.8
08-18	0.2	206.0
08-19	2.2	203.2
08-20	2.8	202.5
08-21	4.1	201.6
08-22	5.2	200.3
08-23	7.3	198.1
08-24	6.0	201.5
08-25	10.4	197.0
08-26	6.7	200.7
08-27	3.1	204.0
08-28	5.1	202.0
08-29	6.4	200.6
08-30	10.7	196.3
08-31	10.7	196.2
08-32	7.5	197.5
08-33	4.8	200.3
D08-01R	10.5	195.0
D08-02L	8.4	198.7
D08-03R	5.3	200.1
D08-04L	7.7	197.8
D08-05R	7.6	197.9
D08-06L	7.8	197.8
D08-07R	4.0	201.5
D08-08L	0.8	204.5

In Boreholes 08-02 and 08-26 to 08-29, bedrock was proved by coring 1.1 to 2.5 m into bedrock. The bedrock is described as grey moderately weathered to fresh granitic gneiss. Core recovery and RQD values in the bedrock were 100%. The Fracture Index (FI) of the rock, expressed as fractures per 0.3 m of core, ranged from 0 to 2.

Groundwater Conditions

Nine of the boreholes were drilled from the ice surface approximately 0.5 to 0.8 m above the ground surface. Water levels were observed in the boreholes during and upon completion of drilling. Standpipe piezometers were installed in Boreholes 08-07 and 08-16R to monitor water levels after completion of drilling. The depth of ice and water at the borehole locations, the water levels observed during drilling, and the water levels measured in the piezometers are summarized in Table 5.10.

Table 5.10 – Water Level Observations

Borehole	Date	Water Level		Comment
		Depth (m)	Elev. (m)	
08-01	Aug 25, 2009	0.9	206.3	Open borehole
08-03	Feb 2, 2009	0.6	205.0	Open borehole
08-04R	Feb 2, 2009	0.0	205.4	Open borehole
08-05	Feb 2, 2009	0.7	204.8	Open borehole
08-06L	Feb 2, 2009	0.5 ags	206.1	Drilling on ice
08-07	Feb 1, 2009	0.5	205.1	Open borehole
	Apr 21, 2009	0.2	205.4	In piezometer
	Jun 4, 2009	0.0	205.6	In piezometer
	Aug 17, 2009	0.0	205.6	In piezometer
	Aug 25, 2009	0.0	205.6	In piezometer
	Oct 27, 2009	0.0	205.6	In piezometer
	Nov 20, 2009	0.0	205.6	In piezometer
08-08R	Feb 1, 2009	0.8 ags	206.3	Drilling on ice
08-09	Feb 1, 2009	0.6 ags	206.1	Drilling on ice
08-10L	Feb 3, 2009	0.5	205.0	Open borehole
08-11	Jan 30, 2009	0.8 ags	206.2	Drilling on ice
08-12R	Jan 30, 2009	0.8 ags	206.3	Drilling on ice
08-13	Jan 30, 2009	0.8 ags	206.2	Drilling on ice
08-14L	Jan 30, 2009	0.6 ags	206.0	Drilling on ice
08-15	Jan 30, 2009	0.6 ags	205.0	Drilling on ice
08-16R	Jan 30, 2009	0.7 ags	206.3	Drilling on ice
	Apr 21, 2009	0.0	205.6	In piezometer
	Jun 4, 2009	0.0	205.6	In piezometer
	Aug 17, 2009	0.0	205.6	In piezometer
	Aug 25, 2009	0.0	205.6	In piezometer
	Oct 27, 2009	0.0	205.6	In piezometer
08-23	Jun 28, 2012	0.0	205.4	In piezometer
08-33	Jun 28, 2012	0.0	205.1	In piezometer

ags = above ground surface

The above values are short-term observations. The surface water depth and depths to groundwater will vary depending upon seasonal fluctuations, rainfall patterns and swamp outlet conditions such as presented by beaver dams. In particular, water levels may be higher after the spring snowmelt or periods of heavy rainfall.

5.6 Highway 69 SBL, Sta. 18+890 to 18+990 Harrison (Appendix G)

General

The subsurface stratigraphy at this site consists of relatively thin deposits of organics and sand underlain by bedrock.

Organics

Dark brown organic material was identified at the ground surface in Boreholes 09-01, 09-03, 09-05, 09-07R and 09-08L. The thickness of the organic layer ranged from 100 to 600 mm. The thickness may vary between and beyond the borehole locations.

An SPT 'N' value 4 blows/0.3 m was recorded in this layer in two boreholes. Natural moisture contents of 47 to 410% were measured.

Sand

Native brown sand containing trace to some gravel, some silt and trace clay was contacted surficially in Boreholes 09-02L and 09-04R and below the organic material in Boreholes 09-03 and 09-05.

The thickness of the sand layer ranged from 0.5 to 1.3 m. The boreholes were terminated below the sand layer at depths ranging from 0.5 to 1.9 m (Elev. 192.2 to 195.7 m), upon auger refusal on probable bedrock.

SPT 'N' values in the sand were 5, 32 and 35 blows/0.3 m of penetration, indicating a loose to dense relative density. The natural moisture contents ranged from 10 to 19%.

Grain size distribution curves of two sand samples tested are presented on the Record of Borehole sheets and on Figure G1 of Appendix G. The results are summarized as follows:

Gravel %	13 to 15
Sand %	62 to 63
Silt & Clay %	22 to 25

Silty Clay

Brown silty clay containing some sand to sandy was encountered below the organic layer in Boreholes 09-07R and 09-08L. The thickness of the silty clay deposit was 1.5 and 1.6 m. Borehole 09-07R was terminated below this layer at 1.6 m depth (Elev. 196.2 m), upon auger refusal on probable bedrock. Borehole 09-08L was terminated on a possible boulder or bedrock at 1.7 m depth.

The cohesive layer is very soft to very stiff in consistency, with SPT 'N' values ranging from 1 to 25 blows/0.3 m. Moisture contents ranged from 20 to 36%.

The results of grain size distribution analyses conducted on two samples of the silty clay are presented on the Record of Borehole sheets and on Figure G2 of Appendix G. Atterberg Limits test results are presented on Figure G3. The results of the laboratory tests are summarized as follows:

Gravel %	0
Sand %	6 to 15
Silt %	36 to 37
Clay %	49 to 57
Liquid Limit	51 to 64
Plastic Limit	22 to 24

The above results indicate that the silty clay is of high plasticity with a group symbol of CH.

Bedrock

Bedrock/probable bedrock was observed surficially or contacted at depths of up to 5.1 m in all boreholes and DCPTs. The depths and elevations of the bedrock surface are summarized in Table 5.11.

Table 5.11 – Depth/Elevation of Probable Bedrock

Borehole	Probable Bedrock Surface	
	Depth below Ground Surface (m)	Elevation (m)
09-01	0.2	198.2
09-02L	0.5	195.0
09-03	1.9	192.2
09-04R	0.5	195.7
09-05	1.2	193.1
09-06L	0.0	196.9
09-07R	1.6	196.2
09-08L	1.7	190.8
D09-01R	0.9	196.9
D09-02L	5.1	187.4
D09-03R	0.0	199.5
D09-04R	1.6	196.6

Groundwater Conditions

Water was observed at 0.3 m depth (Elev. 197.5 m) in Borehole 09-07R upon completion of drilling. Water was not observed in the other boreholes during drilling.

Seasonal fluctuations of the groundwater level are to be expected. In particular, the groundwater level may be at a higher elevation after the spring snowmelt or after periods of heavy rainfall.

5.7 Highway 69 NBL & SBL, Sta. 19+285 to 19+350 Harrison (Appendix H)

General

This site consists of an open body of water bounded by steeply sloping bedrock outcrops. The investigation was carried out while the water surface was frozen, and the boreholes were advanced through 0.5 to 1.8 m depth of ice and water. The stratigraphy encountered below the water generally comprised a relatively thick peat deposit overlying layers of silty clay, silty sand, sand and gravel. Probable bedrock was encountered at depths of 0.5 to 18.3 m below the ice surface.

Upper Silty Clay

An upper layer of grey silty clay containing some sand and peaty organics was contacted below the ice and water at 0.9 to 1.5 m depth in Boreholes 10-01, 10-02, C313-1, C313-2, C313-3 and C314-1. The thickness of the upper clay layer was 0.2 to 1.1 m. The lower boundary of this layer was at 1.5 to 2.6 m below the ice surface (Elev. 190.6 to 191.7 m).

The split spoon sampler generally sank under the weight of the hammer in the silty clay, indicating a very soft consistency. The natural moisture contents were 37 to 77%.

Peat

A 0.6 to 4.3 m thick deposit of brown to dark brown fibrous peat was encountered below the ice and water in Boreholes 10-03L to 10-06 and 11-03L to 11-06, and below the upper silty clay layer in Boreholes 10-01, 10-02 and C313-1 to C313-3. A 25 to 50 mm thick veneer of peat was also encountered on the pond bottom in Boreholes C313-2, C314-1 and C314-2.

The depth to the base of the thicker peat deposits ranged from 1.5 m to 5.8 m below the ice surface, 0.6 to 4.3 m below the pond bottom (Elev. 187.4 to 191.7 m). The peat thickness may vary between and beyond the borehole locations.

The peat was described as clayey in the majority of the boreholes, and a 1.3 m thick layer of organic clay was contacted within the peat layer at 3.0 m depth in Borehole 10-04.

SPT 'N' values of the peat ranged from 0 to 3 blows per 0.3 m of penetration, indicating a very soft to soft consistency. The natural moisture contents ranged from 54 to 665%.

Silty Clay

Grey silty clay containing trace sand to sandy was encountered below the peat in Boreholes 10-02 to 10-06, 11-04, C313-1 and C313-2 at depths ranging from 3.7 to 5.8 m (Elev.

187.4 to 189.5 m). The thickness of the silty clay deposit ranged from 0.8 to 7.5 m. Locally in Borehole 11-04, the clay layer was interrupted by 0.3 and 1.0 m thick layers of gravelly sand and silty sand.

The lower boundary of the silty clay was encountered at depths of 4.5 to 13.3 m below the ice surface, 3.0 to 11.5 m below the ground surface (Elev. 179.9 to 188.7 m).

The cohesive layer is very soft in consistency, and the split spoon sampler generally sank under the weight of the hammer. 'N' values of 2 to 4 blows/0.3 m (soft) were recorded locally. Moisture contents in the clay ranged from 22 to 85%.

The results of grain size distribution analyses conducted on samples of the silty clay are presented on the Record of Borehole sheets and on Figures H1 and H2 of Appendix H. Atterberg Limits test results are presented on Figure H5 of Appendix H. The results of the laboratory tests are summarized as follows:

Gravel %	0 to 1
Sand %	1 to 21
Silt %	35 to 55
Clay %	27 to 62
Liquid Limit	47 to 57
Plastic Limit	20 to 24

The above results indicate that the silty clay is of medium to high plasticity with group symbols of CI-CH.

The results of oedometer (one-dimensional consolidation) testing conducted on a sample of the silty clay are included in Appendix H and summarized in Table 5.12.

Table 5.12 – Consolidation Test Parameters

Borehole	Sample Depth (m)	Soil Type	w_o (%)	γ (kN/m^3)	e_o	p_o' (kPa)	p_c' (kPa)	OCR	C_c	C_r
10-05R	7.6-8.2	CI	65	16.0	1.82	25	48	1.9	0.86	0.08

Comparison of the existing and preconsolidation pressures (p_o' and p_c') derived from the test results indicate that the natural silty clay is slightly overconsolidated. The coefficient of consolidation, c_v, recorded during the test generally varied from 10^{-3} to $5x10^{-4}$ cm^2/s in the normally consolidated range and about $8x10^{-4}$ cm^2/s for the preconsolidated pressure range. The compressibility characteristics will vary with depth in accordance with the moisture content and shear strength profiles.

The results of a Consolidated-Undrained (CU) Triaxial test carried out on a silty clay sample are summarized below and presented in Appendix H.

Table 5.13 – CU Triaxial Test Results

Borehole	Sample Depth (m)	Soil Type	w_o (%)	γ (kN/m^3)	c' (kPa)	ϕ' (deg)
10-05R	7.6-8.2	CI	60	16.1	2	28

Sand to Silt and Sand

Grey to brown sand to silt and sand was encountered below the ice/water in Borehole 11-02, below the peat in Boreholes 11-03L, 11-05R, 11-06, C313-2 and C314-2, within the silty clay deposit in Borehole 11-04, and below the silty clay in Boreholes 10-02, 10-04, 10-06 and C314-1. A thin (100 mm) layer was also encountered over the peat layer in Borehole C313-3.

The upper boundary of the sand layer was at depths of 1.5 to 11.1 m below the ice surface, or 0.0 to 9.6 m below the pond bottom (Elev. 182.1 to 191.7 m). The lower boundary was at depths of 2.1 to 14.8 m below the ice surface (Elev. 178.4 to 191.1 m). The thickness of the sand layer ranged from 0.5 to 5.5 m.

SPT 'N' values measured in the sand to silt and sand ranged from 0 and 43 blows/0.3 m of penetration, indicating a very loose to dense relative density. Moisture contents typically ranged from 15 to 30%, locally up to 63% reflecting the presence of organics.

Grain size distribution curves for samples of the sand to silt and sand are presented on the Record of Borehole sheets and on Figure H3 of Appendix H. The results are summarized as follows:

Gravel %	0 to 5
Sand %	38 to 80
Silt %	12 to 54
Clay %	2 to 25

Gravel to Gravelly Sand

Grey gravel to gravelly sand was contacted below the silty clay in Boreholes 10-03L, 10-05R, 11-04 and C313-1 at depths of 5.2 to 13.3 m below the ice surface (Elev. 179.9 to 188.0 m) and below the sand in Boreholes 10-04, 11-05R and C114-2 at 3.0 to 14.8 m below the ice surface (Elev. 178.4 to 190.2 m). In Boreholes 10-03L, 11-04, 11-05R, C313-1 and C314-2, the thickness of the gravel/sand ranged from 0.8 to 4.7 m. In boreholes 10-04 and 10-05R, 1.0 and 2.5 m of sand/gravel was encountered then a DCPT cone was driven a further 2.5 m to refusal.

The depths to the base of the sampled gravel/sand layers were 3.8 to 15.8 m (Elev. 177.4 to 189.4 m).

SPT 'N' values measured in the gravel ranged from 5 to 65 blows/0.3 m, indicating a loose to very dense relative density. Moisture contents ranged from 8 to 18%.

Grain size distribution curves for three samples of sand/gravel are presented on the Record of Borehole sheets and on Figure H4 of Appendix H. The results are summarized as follows:

Gravel	19 to 53
Sand	40 to 77
Silt & Clay	4 to 7

Bedrock

Probable bedrock was contacted in all boreholes and DCPTs at depths of 0.0 to 16.8 m below the pond bottom (Elev. 174.9 to 192.7 m). The depths and elevations of the probable bedrock surface are summarized in Table 5.14.

Table 5.14 – Depth/Elevation of Probable Bedrock

Borehole	Probable Bedrock Surface	
	Depth below Pond Bottom (m)	Elevation (m)
10-01	2.0	190.3
10-02	3.7	188.0
10-03L	8.4	183.3
10-04	16.8	174.9
10-05R	16.5	174.9
10-06	12.4	179.5
D10-01R	8.5	183.2
D10-02L	13.5	177.9
11-01	0.0	192.7
11-02	1.1	190.6
11-03L	3.6	187.8
11-04	9.5	181.9
11-05R	9.0	182.4
11-06	2.5	189.8
D11-01R	4.6	186.8
D11-02L	2.6	190.6
C313-1	8.0	183.8
C313-2	3.4	188.4
C313-3	2.8	189.2
C314-1	0.8	191.1
C314-2	2.2	189.4

Groundwater Conditions

The boreholes and DCPTs were advanced from the ice surface approximately 0.5 to 1.8 m above the pond bottom. At the time of drilling the ice surface was near Elev. 193.2 m.

The above values are short-term observations. The water depth will vary depending upon seasonal fluctuations, rainfall patterns and outlet conditions such as presented by beaver dams. In particular, water levels may be higher after the spring snowmelt or periods of heavy rainfall.

5.8 Highway 69 SBL, Sta. 19+455 to 19+555 Harrison (Appendix I)

General

The majority of this site was covered by ice and water at the time of the investigation, and the ice/water was up to 1.4 m deep at the borehole locations. Deposits of peat, silty clay and sand were encountered below the ice/water, underlain by probable bedrock at depths of 0.0 to 4.5 m below the ice surface.

Peat and Organics

Brown fibrous peat was contacted below the ice/water in Boreholes 12-03, 12-08R and 12-09. The thickness of the peat was 0.1 and 0.2 m in Boreholes 12-08R and 12-09, and 2.1 m in Borehole 12-03. The lower boundary of the peat was at Elev. 196.2 to 197.9 m. The peat thickness may vary between and beyond the borehole locations.

The peat is described as very soft to firm, based on SPT 'N' values of 1 to 6 blows/0.3 m. Moisture contents ranged from 38 to 189%.

A 500 mm thick layer of organic material was encountered over the bedrock at the location of Borehole 12-01.

Silty Clay

Grey silty clay containing trace sand was encountered below the ice and water in Boreholes 12-04R, 12-05 and 12-07, and below a sandy silt layer in Borehole 12-06L. The thickness of the silty clay ranged from 0.8 to 2.4 m, and the lower boundary was at depths of 1.4 to 4.5 m below the ice surface, 0.8 to 3.1 m below the ground surface (Elev. 193.9 to 197.0 m).

The cohesive clay layers are very soft to soft in consistency, based on SPT 'N' values of 0 to 3 blows/0.3 m. In situ vane testing indicated undrained shear strengths in the order of 8 kPa. The moisture contents ranged from 38 to 83%.

The results of grain size distribution analyses conducted on samples of the silty clay are presented on the Record of Borehole sheets and on Figure I1 of Appendix I. Atterberg

Limits test results are presented on Figure I2 of Appendix I. The results are summarized as follows:

Gravel %	0
Sand %	2 to 8
Silt %	30 to 34
Clay %	60 to 66
Liquid Limit	53 to 61
Plastic Limit	21 to 23

The above results show that the silty clay is of high plasticity with a group symbol of CH.

Silty Sand to Sandy Silt

Native grey silty sand containing some clay and some gravel was contacted below the peat in Borehole 12-03 and below the silty clay in Borehole 12-04R. A layer of sandy silt containing trace gravel was encountered below the water in Borehole 12-06L.

The thickness of the silty sand and sandy silt layers was 0.5 to 0.8 m. The depth to the base of the silty sand and sandy silt deposits ranged from 2.2 to 2.7 m below the ice surface (Elev. 195.7 to 196.2 m).

SPT 'N' values of the silty sand and sandy silt ranged from were 2 to 7 blows/0.3 m, indicating a very loose to loose relative density. The natural moisture contents ranged from 10 to 38%.

Gravelly Sand

Grey gravelly sand containing trace clay was contacted below the silty clay at 2.2 m depth below the ice surface in Borehole 12-05. The thickness of the gravelly sand was 0.3 m. The borehole was terminated below the gravelly sand layer at 2.5 m depth (Elev. 195.9 m) upon auger refusal on probable bedrock.

The measured moisture content of the gravelly sand was 65%, indicative of an organic content.

Bedrock

Probable bedrock was contacted in all boreholes and DCPTs at depths of 0.0 to 5.0 m below the ground surface (Elev. 192.1 to 201.7). The depths and elevations of the probable bedrock surface are summarized in Table 5.15.

Table 5.15 – Depth/Elevation of Probable Bedrock

Borehole	Probable Bedrock Surface	
	Depth below Ground Surface (m)	Elevation (m)
12-01	0.5	201.7
12-02L	0.0	199.2
12-03	2.6	195.7
12-04R	1.6	196.2
12-05	1.1	195.9
12-06L	3.1	193.9
12-07	2.4	194.6
12-08R	0.1	197.8
12-09	0.2	197.9
D12-01R	0.0	199.0
D12-02L	3.7	193.3
D12-03R	5.0	192.1
D12-04L	3.3	193.9

Groundwater Conditions

The majority of the boreholes/DCPTs were drilled from the ice surface approximately 0.1 to 1.4 m above the ground surface. A standpipe piezometer was installed in Borehole 12-07 to monitor water levels after completion of drilling. The depth of ice and water at the borehole locations and the water levels measured in the piezometer are summarized in Table 5.16.

Table 5.16 – Water Level Observations

Borehole	Date	Water Level		Comment
		Depth (m)	Elev. (m)	
12-03	Feb 11, 2009	0.1 ags	198.4	Drilling on ice
12-04R	Feb 11, 2009	0.6 ags	198.4	Drilling on ice
12-05	Feb 11, 2009	1.4 ags	198.4	Drilling on ice
12-06L	Feb 11, 2009	1.4 ags	198.4	Drilling on ice
12-07	Feb 11, 2009	1.4 ags	198.4	Drilling on ice
	Apr 21, 2009	0.0	197.0	In piezometer
	Jun 4, 2009	0.1	196.9	In piezometer
	Aug 17, 2009	0.0	197.0	In piezometer
	Aug 25, 2009	0.0	197.0	In piezometer
	Oct 27, 2009	0.0	197.0	In piezometer
	Nov 20, 2009	0.0	197.0	In piezometer
12-08R	Feb 11, 2009	0.5 ags	198.4	Drilling on ice
12-09	Feb 11, 2009	0.3 ags	198.4	Drilling on ice

ags = above ground surface

The above values are short-term observations. The surface water depth and depths to groundwater will vary depending upon seasonal fluctuations, rainfall patterns and swamp outlet conditions such as presented by beaver dams. In particular, water levels may be higher after the spring snowmelt or periods of heavy rainfall.

5.9 Highway 69 NBL & SBL, Sta. 19+680 to 19+870 Harrison (Appendix J)

General

The subsurface stratigraphy encountered at this site generally consisted of surficial deposits of peat and silty sand overlying a thick layer of silty clay, underlain by basal deposits of silty sand to sand and gravel. Probable bedrock was encountered at depths of 0.3 to 12.3 m.

Peat

Brown to black fibrous peat was encountered surficially in all the boreholes drilled at this site except Boreholes 13-09, 14-11 and 14-12 at the north limit of the study area. The thickness of the peat layer ranged from 100 to 800 mm. The underside of the peat was at Elev. 190.9 to 194.4 m. The peat thickness may vary between and beyond the borehole locations.

The peat is described as very soft, based on SPT 'N' values of 0 and 1 blow/0.3 m of penetration. Moisture contents ranged from 33 to 266%, with one value of 621% measured.

Topsoil

A 100 to 200 mm thick layer of topsoil was identified at the ground surface in Boreholes 13-09, 14-11L and 14-12 located at the north limit of the study area. The topsoil thickness may vary between and beyond the borehole locations.

Upper Silty Sand to Sand

Native brown to grey silty sand to sand containing trace to some clay and trace to some gravel was contacted below the peat and above the underlying silty clay deposit or bedrock in Boreholes 13-01 to 13-04R, 13-10, 13-11R, 13-15R and C317-1. The thickness of the silty sand layer was 0.4 to 1.2 m. The depth to the base of the silty sand ranged from 0.6 to 1.5 m (Elev. 190.7 to 193.6 m).

SPT 'N' values of the silty sand ranged from 2 to 7 blows/0.3 m (loose to very loose) in Boreholes 13-01 to 13-03. Values of 28 and 55 blows/0.3 m (compact and very dense) were recorded in Boreholes 13-10 and 13-04R, respectively. A higher 'N' value of 59 blows/0.2 m was obtained on probable cobbles in the sand in Borehole C317-1. Moisture contents ranged from 16 to 41%.

The results of a grain size distribution analysis conducted on a single sample of the sand (Borehole 13-10) are presented on the Record of Borehole sheets and on Figure J8 of Appendix J. The results are summarized as follows:

Gravel %	16
Sand %	64
Silt %	16
Clay %	4

Silty Clay

A deposit of brown to grey silty clay containing trace sand to sandy was encountered below the peat in Boreholes 13-05 to 13-07, 13-12 to 13-14, 14-02 to 14-10, 14-13, 14-14R, 317-2 and 318-1, as well as below the silty sand in Boreholes 13-01 to 13-03.

The thickness of the silty clay ranged from 1.2 to 9.2 m. The lower boundary of the silty clay was encountered at depths of 1.4 to 10.0 m (Elev. 181.8 to 192.1 m).

SPT 'N' values obtained in the silty clay typically ranged from 0 to 9 blows/0.3 m, indicating very soft to stiff consistency. Locally towards the outer edges of the site, higher 'N' values of 11 to 21 blows/0.3 m were obtained, indicating stiff to very stiff conditions. In situ vane tests indicated undrained shear strengths in the order of 16 to 64 kPa. The moisture contents ranged from 15 to 89%.

The results of grain size distribution analyses conducted on samples of the silty clay are presented on the Record of Borehole sheets and on Figures J1 to J7, Appendix J. Atterberg Limits test results are presented on Figures J10 to J15 of Appendix J. The results are summarized as follows:

Gravel %	0 to 1
Sand %	1 to 47
Silt %	23 to 71
Clay %	24 to 76
Liquid Limit	31 to 74
Plastic Limit	15 to 26

The above results show that the silty clay is of low to high plasticity with group symbols of CL to CH.

Silty Sand to Sand

Grey to brown silty sand containing some gravel and trace to some clay was contacted below the silty clay in Boreholes 13-05 to 13-07, 14-03L, 14-08, 14-13, 14-14R and C318-1. In addition, brown to grey silty sand to sand containing trace to some clay and trace to some gravel was encountered below the peat and topsoil in Boreholes 13-08R, 13-09, 14-11L and 14-12 at the north end of the site. Cobbles were encountered within the silty sand in Boreholes 13-09 and 14-08.

The thickness of the silty sand to sand ranged from 0.2 to 3.1 m. The boreholes were terminated below the silty sand layer at depths of 0.7 to 12.3 m (Elev. 179.5 to 192.9 m) upon auger refusal on probable bedrock.

SPT 'N' values recorded in the silty sand to sand ranged from 1 to 77 blows/0.3 m, indicating a very loose to very dense relative density. The moisture contents ranged from 15 to 28%.

Grain size distribution curves for samples of the sand and silty sand are presented on the Record of Borehole sheets and on Figures J8 and J9 of Appendix J. The results are summarized as follows:

Gravel %	0 to 10
Sand %	38 to 75
Silt %	23 to 53
Clay %	6 to 12

Gravelly Sand to Sand and Gravel

Brown to grey gravelly sand to sand and gravel was contacted below the silty clay at depths of 3.0 to 4.6 m in Boreholes 13-01, 14-06 and 14-09R. The thickness of the sand/gravel layer ranged from 0.8 to 1.4 m.

The boreholes were terminated below the sand/gravel layer at 3.8 to 5.7 m depth (Elev. 186.4 to 188.4 m) upon auger refusal on probable bedrock.

The sand and gravel layer is loose to compact in relative density, based on an SPT 'N' value of 8 to 16 blows/0.3 m. The natural moisture contents were 16 to 19%.

Bedrock

Probable bedrock was contacted in all boreholes and DCPTs at depths of 0.3 to 12.3 m below the ground surface (Elev. 179.5 to 193.6 m). The depths and elevations of the probable bedrock surface are summarized in Table 5.17.

Table 5.17 – Depth/Elevation of Probable Bedrock

Borehole	Probable Bedrock Surface	
	Depth below Ground Surface (m)	Elevation (m)
13-01	4.4	188.4
13-02L	7.4	184.8
13-03	3.8	189.2
13-04R	1.1	192.7
13-05	3.9	188.5

Borehole	Probable Bedrock Surface	
	Depth below Ground Surface (m)	Elevation (m)
13-06L	4.3	187.7
13-07	1.8	190.9
13-08R	1.5	191.3
13-09	3.3	189.7
13-10	1.4	192.6
13-11R	0.9	193.6
13-12	1.8	191.4
13-13L	4.6	188.3
13-14	1.7	191.0
13-15R	0.6	192.8
D13-01R	2.0	191.3
D13-02L	8.3	183.0
D13-03R	0.3	193.0
D13-04L	2.2	189.3
D13-05L	1.5	191.5
D13-06R	2.3	191.5
D13-07L	4.9	187.4
14-01	0.5	192.3
14-02	5.3	187.2
14-03L	12.3	179.5
14-04	5.6	186.1
14-05R	7.4	184.4
14-06	3.8	188.0
14-07L	2.3	189.6
14-08	4.8	187.1
14-09R	5.7	186.4
14-10	2.8	189.8
14-11L	0.7	192.9
14-12	2.9	191.6
14-13	1.7	191.8
14-14R	2.1	190.5
D14-01R	10.9	180.8
D14-02L	4.1	187.8
D14-03R	3.9	188.1
D14-04L	5.2	187.6
D14-05R	4.2	189.8
D14-06R	2.7	190.4
D14-07L	2.9	190.1
C317-1	1.2	191.4
C317-2	4.3	188.1
C318-1	2.4	190.5

Groundwater Conditions

Water levels were observed in the boreholes during and upon completion of drilling. Standpipe piezometers were installed in Boreholes 13-06L and 14-02 to monitor water levels after completion of drilling. The water levels observed during drilling, and the water levels measured in the piezometers are summarized in Table 5.18.

Table 5.18 – Water Level Observations

Borehole	Date	Water Level		Comment
		Depth (m)	Elev. (m)	
13-01	Feb 11, 2009	0.0	192.8	Open borehole
13-02L	Feb 10, 2009	0.0	192.2	Open borehole
13-03	Feb 10, 2009	0.0	193.0	Open borehole
13-04R	Feb 10, 2009	0.6	193.2	Open borehole
13-05	Feb 12, 2009	0.0	192.4	Open borehole
13-06L	Apr 21, 2009	0.0	192.0	In piezometer
	Jun 4, 2009	0.0	192.0	In piezometer
	Aug 17, 2009	0.0	192.0	In piezometer
	Aug 25, 2009	0.0	192.0	In piezometer
	Oct 27, 2009	0.0	192.0	In piezometer
	Nov 20, 2009	0.0	192.0	In piezometer
13-07	Feb 12, 2009	0.5	192.2	Open borehole
13-08R	Feb 12, 2009	0.5	192.3	Open borehole
13-09	Feb 12, 2009	1.2	191.8	Open borehole
13-13L	Feb 10, 2011	2.2	190.7	Open borehole
	Feb 22, 2011	0.2	192.7	In piezometer
	Mar 1, 2011	0.2	192.7	In piezometer
	Mar 13, 2011	0.0	192.9	In piezometer
	Apr 27, 2011	0.2	192.7	In piezometer
13-14	Feb 10, 2011	1.0	191.7	Open borehole
14-02	Apr 21, 2009	0.0	192.5	In piezometer
	Jun 4, 2009	0.0	192.5	In piezometer
	Aug 17, 2009	0.0	192.5	In piezometer
	Aug 25, 2009	0.0	192.5	In piezometer
	Oct 27, 2009	0.0	192.5	In piezometer
	Nov 20, 2009	0.0	192.5	In piezometer
14-03L	Feb 10, 2009	0.0	191.8	Open borehole
14-04	Feb 11, 2009	0.0	191.7	Open borehole
14-05R	Feb 9, 2009	0.0	191.8	Open borehole
14-06	Feb 9, 2009	0.0	191.8	Open borehole
14-07L	Feb 9, 2009	0.0	191.9	Open borehole
14-08	Feb 12, 2009	0.0	191.9	Open borehole
14-09R	Feb 12, 2009	0.0	192.1	Open borehole
14-10	Feb 13, 2009	1.5	191.1	Open borehole
14-11L	Feb 13, 2009	0.6	193.0	Open borehole
14-12	Feb 12, 2009	2.4	192.1	Open borehole
14-14R	Feb 10, 2011	1.3	191.3	Open borehole

Borehole	Date	Water Level		Comment
		Depth (m)	Elev. (m)	
C317-1	Feb 11, 2011	0.9	191.8	Open borehole
C317-2	Feb 22, 2011	0.0	192.4	In piezometer
	Mar 1, 2011	0.0	192.4	In piezometer
	Mar 13, 2011	0.0	192.4	In piezometer
	Apr 27, 2011	0.1	192.3	In piezometer
C318-1	Feb 11, 2011	0.6	192.3	Open borehole

The above values are short-term observations. The surface water depth and depths to groundwater will vary depending upon seasonal fluctuations, rainfall patterns and swamp outlet conditions such as presented by beaver dams. In particular, water levels may be higher after the spring snowmelt or periods of heavy rainfall.

6 MISCELLANEOUS

MMM Group survey personnel staked the centreline alignment prior to drilling of the boreholes. The borehole locations were established by measuring offset distances from the centreline staking. The approximate ground surface elevations at the boreholes were interpreted from the contour plan provided by MMM Group Limited.

Eastern Ontario Diamond Drilling Ltd. of Hawkesbury, Ontario supplied and operated the drilling and sampling equipment for the field program. The portable sampling equipment provided for the supplementary boreholes in November 2010 was provided by OGS Inc. Full time supervision of the field activities, including obtaining utility clearances, was carried out by Mr. Stephane Loranger, Mr. Will Ball, Mr. Jason Mei and Ms. Eckie Siu of Thurber.

Supervision of the field program, interpretation of the field data, and preparation of the report was performed by Mrs. Rocío Palomeque Reyna, P. Eng. and Mr. Murray Anderson, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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FOUNDATION INVESTIGATION AND DESIGN REPORT
SWAMP CROSSINGS AND HIGH FILL EMBANKMENTS
HIGHWAY 69 FOUR-LANING
FROM THE SOUTH JUNCTION OF HIGHWAY 529 TO 15 KM NORTH
G.W.P. 5076-06-00
SOUTH SECTION – HIGHWAY 529 TO NAISCOOT LAKE

Geocres Number: 41H-96

PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 INTRODUCTION

This report presents interpretation of the geotechnical data in the factual report and presents foundation design recommendations for swamp crossings and high fill embankments required for the proposed four-laning of Highway 69.

The overall project consists of widening Highway 69 from a two-lane undivided roadway to a four-lane divided highway. The current widening section extends from the south junction of Highway 529 northerly for 15 km in the Townships of Harrison and Wallbridge, Ontario. This report addresses the southern part of this section, from Hwy 529 to north of Naiscoot Lake, a length of approximately 3.8 km. The remaining part of the section is discussed in a separate report.

Nine areas of swamp crossing or high fill embankments are addressed in this report. A summary of the sections, including locations, lengths, maximum fill height, generalized stratigraphy and groundwater conditions, is presented on Table A3 in Appendix A. The factual data for each section has been assigned to an appendix, Appendix B through J, and the respective appendix designation is included in Table A3.

The project information used for preparation of this report, including plans and profiles of the proposed alignments as of July 2010 and cross-sections as of January 2011, was provided by MMM Group. The discussion and recommendations presented in this report are based on the information provided by MMM and the factual data obtained during the course of the investigation.

8 ENGINEERING ANALYSIS METHODOLOGY

8.1 General

The subsurface conditions at the swamp crossing and high fill locations were investigated to assess the stability of proposed embankment slopes, potential embankment settlement issues, and construction concerns. Major factors to be addressed for embankment design on this project include:

- Embankment geometry (height, slope angle, stabilizing berms);
- Embankment material type (rockfill, granular fill or earth fill);
- The extent and thickness of peat, organic, compressible and/or excessively soft/loose soils underlying the embankment footprint;
- The thickness and engineering properties of the foundation soils;
- The elevations and properties of bedrock;
- Post-construction settlement of embankments;
- Construction procedures and groundwater conditions during removal of peat, organic or compressible soils.

The geotechnical analyses summarized in this report include assessment of the global stability of fill embankments for both short and long term conditions. Assessment of immediate and long-term settlements, including magnitude and time rate, was also carried out. The analyses were based on the soil profiles and properties encountered at various locations, selected for less favourable conditions.

For the purpose of preparing geotechnical design recommendations, a number of assumptions have been made that are consistent with MTO's standard highway design practices:

- The high fills and embankments crossing swamps will be constructed using rock fill. Sources of earth fill are not expected to be available on this project. Granular fill may be required for surcharge construction.
- High fills will be constructed with side slopes not exceeding 1.25H:1V for rock fill and 2H:1V for granular fill.
- Embankment slopes greater than 10 m high in rock fill will be provided with a 2 m wide mid-height berm.
- Organic deposits, topsoil, peat or other deleterious material will be removed prior to constructing fill embankments.

8.2 Stability Analyses

Where the planned fill height exceeds 4.5 m and the foundation does not consist of bedrock, stability analyses were carried out. The commercially available slope stability program GSLOPE developed by Mitre Software Inc. with the option for Bishop's modified method of slices was used for the limit equilibrium analyses.

Analyses were carried out for rock fill embankments, under static and seismic loading conditions. For cohesive soils, short term (undrained) and long term (effective stress) conditions were assessed. Mid-height berms of 2 m width were applied to all slope heights exceeding 10 m in rock fill.

Based on consideration of the risk involved and past experience with highway embankment design/monitoring, a factor of safety of 1.3 is considered appropriate to achieve both short and long-term stability for embankments founded on cohesionless soils. For cohesive foundation soils, the recommended factor of safety is 1.3 for short-term conditions and 1.5 for long-term conditions.

The input parameters and soil model used in the stability analyses, including soil stratigraphy, engineering properties, groundwater conditions, and embankment geometry, are shown for sample analyses on Figures 69HWY001 to 69HWY018 in Appendix A.

8.3 Settlement Analyses

Settlement analysis involved computation of the immediate settlement of the foundation soils under the imposed embankment loading, calculation of long-term consolidation settlement using Terzaghi one-dimensional consolidation theory, and estimation of long-term settlement of embankment fill materials due to compression under self-weight.

Immediate settlements due to compression of the embankment foundation soils have been estimated based on elastic theory as described in CHBDC Commentary Section C6.6.

For cohesive soils, the estimated primary consolidation settlement and time to achieve 98% of the consolidation was calculated based on Terzaghi's one-dimensional vertical consolidation formulation combined with computation of stresses in two-dimensional elastic half-space as described in CHBDC Commentary Section C6.6. The parameters used in the analyses were determined by laboratory oedometer tests conducted during the current study and soil moisture correlations developed during past projects.

Settlement due to secondary consolidation of the cohesive deposits was assessed for a design period of 30 years using the following equation and a secondary compression ratio selected based on soil moisture correlations:

$$S_{creep} = C_{\alpha\varepsilon} H \text{ Log } (t/t_p)$$

where S_{creep} = settlement due to secondary consolidation (m)
$C_{\alpha\varepsilon}$ = secondary compression ratio
H = initial thickness of compressible layer (m)
t = time over which secondary compression is to be calculated
t_p = time to completion of primary consolidation

Settlement of rock fill due to particle re-orientation and degradation of the interparticle contacts is expected to continue at a decreasing rate for many years. In accordance with the MTO document "Post-Construction Rock Fill Settlement and Guidelines for Estimating Rock Fill Quantity" (April 12, 2010), the magnitude of this settlement in compacted rock fill is expected to range from 0.5 to 1.0% of the embankment height within 1 year of embankment construction (90% in the first 6 months), and a further 0.1% after the 1 year period. For dumped rock fill (under the water level), these settlement values would be approximately doubled.

The estimated settlement of granular fill embankments due to compression of the compacted fill is 0.5% of the embankment height and is expected to be completed within one to two years after construction.

The estimated magnitudes and rates of settlement are considered approximate and may vary along and across the highway alignment subject to the thickness of compressible layers at a particular location, variations in the consolidation characteristics of the cohesive deposits with depth and location, layer boundary conditions, variations in the relative density of cohesionless soils, the presence of organics or silt/sand/clay partings within the various strata, the depth to bedrock, the height of embankment fill, and degree of compaction achieved in the fill.

8.4 Design Alternatives

Design alternatives considered during analysis of the embankments would typically include the following:

- Full and/or partial sub-excavation of soft cohesive soils in addition to removal of the peat and organic soils;
- Provision of berms and/or flattening of embankment slopes to improve global stability;
- Ground improvement techniques such as preloading/surcharging and geosynthetic reinforcement;

- Construction techniques such as wick drain installation to accelerate settlement or staged construction to maintain stability;
- Use of lightweight fill.

Based on the investigation findings, the results of the design analyses, and construction scheduling considerations, preference has been given to full excavation of peat and soft soils when selecting the recommended option to address stability and/or settlement issues.

8.5 Seismic Considerations

The following seismic parameters have been considered in design:

Velocity Related Seismic Zone	1
Zonal Velocity Ratio	0.05
Acceleration Related Seismic Zone	1
Zonal Acceleration Ratio	0.05

The soil profile type at all sites except Sta. 19+680 to 19+870 (Appendix J) has been classified as Type I. Therefore, according to Table 4.4.6.1 of the CHBDC, a Site Coefficient "S" (ground motion amplification factor) of 1.0 should be used in seismic design.

The thickness of soft clay at Sta. 19+680 to 19+870 exceeds 9 m, and therefore the soil profile is classified as Type III and a Site Coefficient of 1.5 is applicable.

A peak horizontal acceleration (PHA) at the ground surface of 0.08 g, where g is the acceleration due to gravity, has been used. The PHA value corresponds to a probability of exceedance of 10% in 50 years. The foundation soils at the site are assessed as not being prone to liquefaction.

9 EMBANKMENT DESIGN AND CONSTRUCTION (INCLUDING SWAMPS)

The generalized subsurface conditions and embankment heights for the various alignments are summarized on Table A3 of Appendix A. The groundwater level was near the ground surface in most sections of proposed fill.

9.1 Swamp Treatment and Subexcavation of Peat and Soft Soils

It is standard procedure on MTO projects to sub-excavate all peat deposits from within the footprint of the embankment, and backfill the resulting excavation with rock or granular fill. Full peat removal is an economical and efficient method of improving stability during construction and minimizing the potential for large post-construction settlements. It is therefore recommended that all peat and organic soils be sub-excavated from the footprint area of all embankments as per the OPSD 203 series.

In some locations, a layer of very soft silty clay or very loose sand with peat inclusions underlies the peat and organic soils. Full or partial subexcavation of the very soft/loose soil is recommended at selected locations to address stability or settlement issues. Further discussion of these areas is presented in subsequent sections.

The anticipated/recommended depth of sub-excavation for peat, soft clay or organic sand removal along the proposed alignments is summarized in Table A4, Appendix A. The depth of excavation is based on the thickness of organics and soft/organic material noted at the borehole locations. Subexcavation depths may vary at locations between and away from the boreholes.

The subexcavated foundation area should be backfilled with rock or granular material as described later in this report. Placement of rock fill is recommended where standing water is encountered.

In the stability and settlement analyses, it has been assumed that the peat and soft/organic deposits have been removed and replaced with rock or granular material as appropriate.

9.2 Site Specific Discussions and Recommended Treatment

Stability analyses were carried out for fill embankments exceeding 4.5 m in height (including sub-excavation depth) and not founded on bedrock. Three areas were identified for analysis. All other high fill and swamp section embankments were less than 4.5 m in height and/or founded on bedrock or thin cohesionless deposits overlying bedrock following subexcavation of peat, organics and soft soils. Results of the stability analyses carried out at selected critical locations within the swamp/fill sections analysed are summarized in Table A5.

Settlement analysis involved computation of the immediate (elastic) settlement of the foundation soils, the magnitude and time rate of primary consolidation of fine-grained foundation soils, long-term secondary compression, and the short and long-term compression of fill materials under self-weight. The predicted post-construction settlements for all sites are summarized in Table A6 of Appendix A.

Supplementary analyses were carried out as necessary to assess design alternatives such as full or partial sub-excavation of the soft clay, preloading/surcharging and wick drain installation. Discussions regarding the design alternatives for each specific swamp or high fill embankment section are provided below.

To mitigate the effects of the settlement, it is recommended that the embankments generally be constructed at least six months in advance of pavement construction. Embankment and platform width design should allow for the anticipated foundation and embankment compression settlements.

High Fill at Station 17+425 to 17+560 Harrison (Appendix B)

The site stratigraphy generally consists of exposed bedrock, a thin layer of organic material over bedrock, and localized deposits to a maximum depth of 3.8 m of loose to compact sand and stiff to very stiff silty clay overlying bedrock. The maximum proposed embankment height is about 9.4 m.

After removal of peat, organics and soft soils, the embankment foundation will essentially comprise bedrock and therefore stability of the embankment slopes is not a concern. Where localized deposits of sand and clay are present, foundation settlements in the order of 30 mm should be anticipated, and these settlements are expected to occur essentially as the embankment fill is placed.

Based on the above, specialized construction procedures will not be required to address stability or settlement issues at this site.

Swamp Crossing at Station 17+900 to 18+000 Harrison (Appendix C)

The site stratigraphy generally consists of up to 5.1 m of peat overlying bedrock or overlying up to 3.8 m of silty sand within a bedrock depression. The maximum proposed embankment height is about 2 m. The total thickness of rock fill following peat removal will be a maximum of about 6.5 m.

After removal of peat, organics and soft soils, the embankment foundation will primarily comprise bedrock and stability of the rock fill embankment slopes is not a concern in these areas. Stability analyses conducted for a representative section of rock fill overlying sand indicated a factor of safety in the order of 1.6, exceeding the minimum acceptable value or 1.3 for this type of analysis.

Where the deeper sand deposits are present below the peat, foundation settlements in the order of 35 to 50 mm should be anticipated, and these settlements are expected to occur essentially as the embankment fill is placed.

Based on the above, specialized construction procedures will not be required to address stability or settlement issues at this site.

Swamp Crossing at Station 18+180 to 18+230 Harrison (Appendix D)

The site stratigraphy generally consists of deposits of organic material, peat and/or silty clay to a maximum depth of 1.4 m, as well as sand/silt to a maximum depth of 2.4 m, overlying bedrock. The maximum proposed embankment height is about 1.5 m.

After removal of peat, organics and soft soils, the embankment foundation will essentially comprise bedrock and therefore stability of the rock fill embankment slopes is not a concern. The surficial deposits of organic material, peat and very soft silty clay should be removed prior to rock fill placement. Where localized deposits of sand/silt remain, foundation settlements should be less than 25 mm and occur essentially as the embankment fill is placed.

Based on the above, specialized construction procedures will not be required to address stability or settlement issues at this site.

Swamp Crossing at Naiscoot Access Road Station 10+555 to 10+590 (Appendix E)

This site stratigraphy generally consists of thin deposits of peat and sand to a maximum depth of 1.4 m, overlying bedrock. The maximum proposed embankment height is about 2.5 m.

After removal of peat, organics and soft soils, the embankment foundation will essentially comprise bedrock and therefore stability of the rock fill embankment slopes is not a concern. Where localized deposits of sand are present, foundation settlements should be less than 25 mm and these settlements are expected to occur essentially as the embankment fill is placed.

Based on the above, specialized construction procedures will not be required to address stability or settlement issues at this site.

Swamp Crossing at Station 18+240 to 18+500 Harrison (Appendix F)

The stratigraphy at this site generally consists of up to 5.9 m of peat underlain by silty clay and various localized deposits of sandy silt, silty sand and sand, overlying bedrock at depths of up to 10.5 m. The maximum proposed embankment height is about 3.0 m above the existing swamp surface. The new highway alignment crosses over the existing Highway 69 embankment within this swamp.

Stability analyses were initially carried out assuming that the peat would be removed and the rock fill embankment would be constructed directly over the very soft clay underlying the peat. The total thickness of rock fill following peat removal and backfilling will be a maximum of about 9 m.

A critical section at approximate Station 18+420 was selected for the analysis (Figures 69HWY001 to 69HWY005 in Appendix A). The computed factors of safety against slope instability for this case were approximately 1.9 for both short-term (undrained) and long-term (drained) conditions. With a 2 m high surcharge, the short-term safety factor (immediately after placement) reduced to 1.31 and the safety factor immediately prior to surcharge removal (effective stress analysis) was 1.28. These factors of safety are considered to be acceptable.

Estimation of the short and long-term foundation settlements to be expected for an embankment constructed on the clay, for cases with and without a surcharge load, produced the following results:

Surcharge Condition	Primary Consolidation		Secondary Compression		Fill Compression	
	Settlement	Time to 98% consolidation	First year	After first year	Short term	Long term
Without surcharge	125 mm	4.5 months	20 mm	90 mm	100 mm	15 mm
With 2 m surcharge	250 mm	10 months	0 mm	5 mm	100 mm	15 mm

Based on the above figures, application of a 2 m surcharge for a period of at least ten months is a practical option to limit long-term settlements to less than 25 mm. Maximum settlement within the first year of embankment construction including surcharge is expected to be in the order of 350 mm. Monitoring will be required to confirm the magnitude and time-rate of settlement if the fill is placed over the clay layer.

The thickness of the silty clay at this site ranges from 0.6 to 2.9 m, and the lower boundary was encountered at depths of 4.5 to 8.1 m. Complete excavation of this additional thickness of very soft material is considered practical and may be considered as an alternative. Stability of the embankment founded on bedrock or over the thin deposits of sand/silt below the clay layer is not a concern. Embankment settlement would essentially be limited to compression of the fill, estimated to be about 140 and 20 mm in the short-term and long-term, respectively.

Considering the improved embankment stability, reduced uncertainty in foundation settlement, elimination of monitoring requirements, and simpler construction procedures, it is recommended that embankment construction at this location include subexcavation of the very soft clay to the full maximum depth of 8.1 m.

Boreholes drilled through the existing Highway 69 embankment encountered 2.1 to 8.5 m of granular material and rock fill typically underlain by 0.1 to 1.0 m, locally 3.1 m, of peat. Excavation of the peat and clay from adjacent to the existing highway embankment constructed over swamp deposits may result in disturbance of the existing roadway embankment and settlement and/or cracking of the pavement surface. To minimize the potential impacts on the existing highway, it is recommended that peat/clay removal within 10 m of the existing slope toe be followed closely by backfilling to 1m above the existing ground surface such that no more than 10 m length of excavation is left open adjacent to the highway at any time. The existing embankment should be visually monitored during construction of the new northbound lanes and maintenance provided as required to repair any cracks or settled areas that develop on the highway surface.

From a geotechnical viewpoint, it is preferred that project staging include construction of the new southbound lanes and shifting traffic to the new southbound lanes prior to swamp excavation and construction of the new northbound lanes. We understand that this cannot be accommodated within the staging plans however.

High Fill at Station 18+890 to 18+990 Harrison (Appendix G)

This site stratigraphy generally consists of thin deposits of organics, sand or silty clay to a maximum depth of 1.9 m (5.1 m in one DCPT), overlying bedrock. The maximum proposed embankment height is about 9.1 m.

After removal of peat, organics and soft soils, the embankment foundation will essentially comprise bedrock and therefore stability of the rock fill embankment slopes is not a concern. Where localized deposits of sand or silty clay are present, foundation settlements should be less than 25 mm and these settlements are expected to occur essentially as the embankment fill is placed.

Based on the above, specialized construction procedures will not be required to address stability or settlement issues at this site.

Pond/Swamp Crossing at Station 19+285 to 19+350 Harrison (Appendix H)

This site consists of a water body with up to 1.8 m depth of ice and water at the time of investigation. Below the water, the subsurface stratigraphy generally consists of up to 4.3 m of peat underlain by 0.8 to 7.5 m of very soft silty clay extending to a maximum depth of 11.5 m below the pond bottom, overlying or interbedded with deposits of sand, silty sand, gravel and gravelly sand. Probable bedrock was encountered at depths of 0.0 to 16.8 m. The maximum proposed embankment height at this site is about 8.6 m.

Stability analyses were initially carried out assuming the peat was removed and the rock fill embankment was constructed directly over the underlying very soft silty clay. A critical section at Station 19+325 of the northbound lanes was selected for the analyses. The resulting total thickness of rock fill including swamp backfill was 12.9 m. The resulting factors of safety against slope instability were less than 1.0, indicating that standard embankment construction is not feasible at this location.

Supplementary analyses were carried out to assess design alternatives such as staged construction, stabilizing berms and full or partial sub-excavation of the soft clay. An iterative approach was applied to produce a practical and cost-efficient solution achieving acceptable factors of safety against slope instability and limiting post-construction settlement to acceptable levels. The need for surcharging and/or waiting periods between successive lifts of staged construction was given consideration in assessing the various options.

The alternative design options and findings of the analyses were as follows:

i. Staged embankment construction without subexcavation of the soft clay was considered. However, it was found that acceptable factors of safety could not be achieved even with up to four stages of rock fill placement.

ii. The effect of stabilizing berms at the toe of the embankment slope was assessed for single stage construction and berm geometries of up to 3.7 m in height (above the pond bottom) and 17 m in width (Figure 69HWY006 in Appendix A). These analyses indicated short-term factors of safety in the order of 1.0 for the maximum berm size evaluated.

iii. Staged embankment construction in conjunction with a stabilizing berm, with and without a 2m high surcharge load was evaluated (Figures 69HWY007 to 69HWY009 in Appendix A). The results indicated that the following staged approach was feasible:

- Stage 1: subexcavate the peat and place rock fill to Elev. 198.0 m within the embankment footprint, and provide a 17 m wide stabilizing berm with a top at Elev. 195.0 m. Provide a 6 month waiting period for dissipation of pore pressures and increase in strength of the clay foundation.
- Stage 2: place the remainder of the rock fill to the full embankment height plus the 2m high surcharge. Provide a 12 month waiting period prior to removal of the surcharge and construction of the pavement.

The computed factors of safety at the completion of each stage of fill placement were in the order of 1.3 for this methodology. The long-term factor of safety was about 2.5. Comments regarding post-construction settlement are provided below.

iv. The analyses were repeated assuming that partial subexcavation of the clay was carried out to a depth of about 8 m below the pond bottom (Elev. 183.5 m), considered to be the practical limit for swamp excavation with readily available equipment. This option would leave a maximum of approximately 3.5 m of clay below the rock fill. These analyses indicated that construction of the embankment to final grade in one stage would result in factors of safety of about 1.3 and 1.5 for short-term and long-term conditions, respectively (Figures 69HWY010 and 69HWY011).

v. For the case where all peat and soft clay is excavated (maximum 11.5 m depth) and the rock fill embankment is constructed over the underlying cohesionless deposits and bedrock, assuming specialized equipment could be mobilized to the site, the factors of safety for short-term and long-term conditions would exceed the required values of 1.3 and 1.5, respectively.

Estimation of the total and post-construction settlements to be expected for the feasible embankment construction methods outlined above produced the following results:

Method	Primary Consolidation		Secondary Compression		Fill Compression	
	Settlement	Time to 98% consolidation	First year	After first year	Short term	Long term
Two stage construction with stabilizing berm, no surcharge	420 mm	10 mths	30 mm	120 mm	190 mm	20 mm
Two stage construction with stabilizing berm, 2m surcharge	490 mm	18 mths	0 mm	15 mm	190 mm	20 mm
One stage construction with partial subexcavation of clay	95 mm	4 mths	10 mm	30 mm	270 mm	30 mm
One stage construction with full subexcavation of peat and clay	-	-	-	-	340 mm	35 mm

Based on the anticipated post-construction settlements presented in the above table, staged construction with a 2 m surcharge and stabilizing berm, or one stage construction with partial or full excavation of the soft clay are considered feasible foundation treatment alternatives at this site. However, to reduce the complexity and duration of construction, and minimize the footprint of the embankment works, full subexcavation of the peat and soft clay to a maximum 11.5 m depth is the recommended treatment option.

It is noted that the maximum subexcavation depth will generally be required only in the northbound lanes towards the north end of the site where the clay depths are greatest. On the remainder of the site, the excavation depth will be limited by the presence of cohesionless soils or bedrock above Elevation 183.5 m. In general, the excavation will be carried out in wet conditions below the water level in the pond.

<u>Swamp Crossing at Station 19+455 to 19+555 Harrison (Appendix I)</u>

The site stratigraphy generally consists of localized deposits of organic material, peat, silty clay and/or sand/silt to a maximum depth of 5.0 m, overlying bedrock. The maximum proposed embankment height is about 2.3 m.

Considering the presence of peaty organics in the silty clay layer, the very low strength, limited thickness and areal extent of the very soft clay deposits, it is recommended that the organic material, peat and very soft silty clay be removed prior to construction of the rock fill embankment at this site.

The embankment foundation remaining after soft material removal will essentially comprise bedrock locally overlain by thin sand/silt deposits. Stability of the rock fill embankment slopes constructed over this material is not a concern. Where localized deposits of sand/silt remain, foundation settlements should be less than 25 mm and occur essentially as the embankment fill is placed.

Based on the above, specialized construction procedures will not be required to address stability or settlement issues at this site.

Swamp Crossing at Station 19+680 to 19+870 Harrison (Appendix J)

The stratigraphy at this site generally consists of a thin peat layer over up to 9.2 m of soft clay and various localized deposits of silty sand to sand and gravel, overlying bedrock at depths of up to 12.3 m. The maximum thickness of clay appears to exist between approximate Stations 19+750 to 19+800, and is generally thinner (less than 4.6 m) on the remainder of the site. The maximum proposed embankment height is about 6.0 m.

Short-term (undrained) and long-term (drained) stability analyses were carried out for a typical section where the foundation clay thickness is less than 4.6 m (Figures 69HWY012 and 69HWY013, Appendix A). The analyses indicated that in these areas, the short-term and long-term factors of safety for a rock fill embankment inclined at 1.25H:1V will be in the order of 1.6 and 1.5, respectively. Embankment stability through the majority of this site is therefore not considered to be an issue.

Stability analysis at the critical location where deeper clay deposits were encountered (Sta. 19+780) was initially carried out assuming only the peat (0.8 m thick) was removed and the rock fill embankment was constructed with standard 1.25H:1V sideslope inclinations over the underlying very soft to soft silty clay (Figure 69HWY014). The computed factor of safety for the short-term condition was less than 1.0, indicating that standard embankment construction would not be feasible at this location.

Supplementary analyses were carried out to assess design alternatives such as staged construction, stabilizing berms, slope flattening, and full or partial sub-excavation of the soft clay. An example analysis is shown on Figure 69HWY015, Appendix A. An iterative process was applied to produce a practical and cost-efficient solution achieving acceptable factors of safety against slope instability and limiting post-construction settlement to acceptable levels. The need for surcharging and/or waiting periods between successive lifts of staged construction was given consideration in assessing the various options.

The alternative design options and findings of the analyses were as follows:

i. Staged embankment construction without subexcavation of the soft clay was considered. However, it was found that acceptable factors of safety could not be achieved even with up to four stages of rock fill placement.

ii. The effect of stabilizing berms at the toe of the embankment slope was assessed for single stage construction. These analyses indicated short-term factors of safety could not be achieved for reasonable berm sizes evaluated.

iii. Single stage construction combined with partial subexcavation of the clay (removal of 4m of clay) was evaluated. When a 2 m surcharge was applied, the short-term factor of safety computed for this analysis was marginally above 1.0 (Figure 69HWY015) and considered inadequate.

iv. Staged embankment construction in conjunction with partial subexcavation of the clay (removal of 4m of clay) and a 2m high surcharge load was evaluated (Figures 69HWY016 to 69HWY018 in Appendix A). The results indicated that the following staged approach was feasible:

- Stage 1: subexcavate the peat and 4 m of clay to Elev. 187.0 m, then place rock fill to Elev. 197.0 m. Provide a 9 month waiting period for dissipation of pore pressures and increase in strength of the clay foundation.
- Stage 2: place the remainder of the rock fill to the full embankment height plus the 2m high surcharge. Provide a 12 month waiting period prior to removal of the surcharge and construction of the pavement.

The computed factors of safety at the completion of each stage of fill placement were in the order of 1.3 for this methodology. The long-term factor of safety was about 1.7. Comments regarding post-construction settlement are provided below.

v. For the case where all peat and soft clay is excavated (maximum 10.0 m depth) and the rock fill embankment is constructed over the underlying cohesionless deposits and bedrock, the factors of safety for short-term and long-term conditions would exceed the required values of 1.3 and 1.5, respectively.

Estimation of the total and post-construction settlements to be expected for the feasible embankment construction methods outlined above produced the following results:

Method	Primary Consolidation		Secondary Compression		Fill Compression	
	Settlement	Time to 98% consolidation	First year	After first year	Short term	Long term
Clay thickness less than 4.1m: one stage construction	60 mm	3 mths	20 mm	60 mm	45 mm	< 10 mm
Area of deep clay: two stage construction with partial subexcavation of clay, no surcharge	525 mm	21 mths	20 mm	125 mm	150 mm	15 mm
Area of deep clay: two stage construction with partial subexcavation of clay, 2m surcharge	680 mm	21 mths	0 mm	15 mm	150 mm	15 mm
Area of deep clay: one stage construction with full subexcavation of peat and clay	-	-	-	-	255 mm	25 mm

Based on the anticipated post-construction settlements, two stage construction with a 2 m surcharge and partial excavation of the soft clay, or single stage construction with full excavation of the soft clay, are considered feasible foundation treatment alternatives in the area of deep clay. However, to reduce the complexity and duration of construction, and minimize the footprint of the embankment works, full subexcavation of the peat and soft clay to a maximum 10.0 m depth is the recommended treatment option.

Subexcavation of the full depth of peat and clay is recommended for the complete length of this swamp. It is noted however that the maximum subexcavation depth will generally be required only between approximate Stations 19+750 to 19+800 where the clay depths are greatest. On the remainder of the site, the excavation depth will be limited by the presence of cohesionless soils or bedrock at higher levels.

9.3 Summary of Site-Specific Recommendations

A summary of the primary recommendations for each specific area of high fill or swamp crossing is presented on Table A7 in Appendix A. The summary is based on the discussions presented above, and these discussions should be referenced for further detail.

The anticipated and/or recommended depth of subexcavation of peat, organics and soft soils at all sites is summarized in Table A4, Appendix A.

10 SEISMIC CONSIDERATIONS

Provided embankment construction is carried out in accordance with the site-specific recommendations provided above, a minimum factor of safety of 1.0 was computed for seismic loading conditions at all sites. This value is considered acceptable for seismic design.

Based on the subsurface conditions encountered at the embankment locations, the potential for liquefaction of the foundation soils during a seismic event is considered to be low in accordance with CHBDC Section C4.6. Some local liquefaction and resulting toe failure may occur during a seismic event, but this is expected to be readily repaired.

11 EMBANKMENT CONSTRUCTION

Embankment construction should be carried out in accordance with SP 206S03. Earth fill may consist of granular materials and Select Subgrade Material (SSM) in compliance with Special Provision 110S13. Rock size should be controlled in accordance with SP 206S03.

Construction of new embankments over swamp should be carried out in accordance with OPSS 209 "Construction Specification for Embankments Over Swamps", March 1998, with specific reference to OPSD 203.010 "Embankments Over Swamp, New Construction".

In areas where staged construction, surcharging and/or preloading is recommended, a monitoring program will be required to confirm the duration of staging and the magnitude and rate of foundation settlements. Further recommendations and a suggested NSSP for monitoring will be prepared when the selected treatment options have been established. Embankment construction in areas requiring treatment should commence at the beginning of the contract period.

Water levels at all sites were generally at or above the ground surface at the time of the field investigations. Removal of peat and subexcavation of soft soils will generally be carried out in wet conditions below the surface water and groundwater levels. Construction operations should include measures such a temporary dewatering and drainage/lowering of ponded water wherever practical (for example, where excavation depths are small), and provision of equipment suitable for excavation of soft materials below the water level where dewatering is not practical (for example, Station 19+285 to 19+350). The surface water depths and depths to groundwater at the time of construction will vary depending upon seasonal fluctuations, rainfall patterns and swamp outlet conditions such as presented by beaver dams.

Rock fill placed above the water table should be placed in a controlled manner (not end dumped) including blading, dozing and chinking of the rock to minimize voids and bridging. Rock fill used to backfill subexcavated areas below the water table may be placed by end dumping and SP 206S03 will not apply. Earth fill must not be used to backfill excavations below the water table.

At the pavement subgrade level or if granular/earth fill is to be placed over rock fill, the rock fill subgrade must be blinded with spall material and covered by a minimum 600 mm thickness of OPSS Granular B Type II fill.

Mid-height berms comprising 2 m wide benches should be incorporated along the length of embankments with heights exceeding 8 m in earth fill and 10 m in rockfill. Where new embankment fill is placed against existing embankment slopes or on a sloping ground surface, the existing earth or fill slope must be benched in accordance with OPSD 208.010.

Earth slopes must be provided with erosion protection in accordance with OPSS 804.

12 CONSTRUCTION CONCERNS

During construction, qualified geotechnical staff should be retained to observe activities related to embankment construction and advise the Contract Administrator on construction concerns or issues related to embankment stability or settlement.

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

- The thickness and presence of organic deposits were investigated at the borehole locations only. Organic deposits may extend to greater depths or be encountered at other locations between boreholes.
- Geotechnical confirmation is required that all organics, peat and soft silt/clay materials within the proposed embankment footprint are sub-excavated and replaced with approved backfill.
- Movement of construction equipment may be difficult in areas of organic or excessively soft, loose and/or saturated subgrade. Disturbance of the subgrade by construction traffic should be minimized.
- Bedrock elevations may vary between and beyond the borehole locations. The limits of sub-excavation may require modification during construction based on the conditions encountered in the field.

13 CLOSURE

Engineering analysis and preparation of the foundation design report were carried out by Mr. Murray Anderson, P.Eng. The report was reviewed by Dr. P.K. Chatterji, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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Appendix A

Tables and Figures

Table A1

Borehole Summary

Borehole	Station	Offset (m)	Direction	Description	Depth (m)
Hwy 69 NBL & SBL, Sta. 17+425 to 17+560 Harrison					
01-01	17+426	CL	NBL	centreline	0.1
01-02L	17+443.5	13.4 LT	NBL	left toe of slope	0.3
01-03	17+451	CL	NBL	centreline	0.1
01-04R	17+463.5	17.6 RT	NBL	right toe of slope	0.1
01-05	17+466	CL	NBL	centreline	0.0
01-06	17+465	CL	SBL	centreline	0.2
01-07R	17+477.5	19.0 RT	SBL	right toe of slope	1.1
01-08	17+490	CL	SBL	centreline	1.1
01-09	17+490	CL	NBL	centreline	3.8
01-10L	17+502.5	15.0 LT	SBL	left toe of slope	3.1
01-11R	17+502.5	17.0 RT	NBL	right toe of slope	2.2
01-12	17+515	CL	SBL	centreline	0.4
01-13	17+515	CL	NBL	centreline	1.4
01-14R	17+527.5	19.0 RT	SBL	right toe of slope	0.4
01-15	17+540	CL	SBL	centreline	0.2
01-16	17+540	CL	NBL	centreline	0.2
D01-01R	17+443.5	13.4 RT	NBL	DCPT, right toe of slope	0.1
D01-02L	17+463.5	17.6 LT	NBL	DCPT, left toe of slope	0.3
D01-03L	17+477.5	15.0 LT	SBL	DCPT, left toe of slope	0.7
D01-04R	17+477.5	17.0 RT	NBL	DCPT, right toe of slope	0.2
D01-05R	17+502.5	19.0 RT	SBL	DCPT, right toe of slope	0.6
D01-06L	17+527.5	15.0 LT	SBL	DCPT, left toe of slope	0.3
D01-07R	17+527.5	17.0 RT	NBL	DCPT, right toe of slope	0.9
Hwy 69 NBL & SBL, Sta. 17+900 to 18+000 Harrison					
02-01	17+901	CL	NBL	centreline	0.5
02-02L	17+924	6.6 LT	NBL	left toe of slope	3.0
02-03	17+926	CL	NBL	centreline	9.1
02-04R	17+938.5	8.6 RT	NBL	right toe of slope	2.2
02-05	17+941	CL	NBL	centreline	1.7
02-07L	17+947.5	12.0 LT	NBL	left toe of slope	1.1
02-08	17+955	CL	NBL	centreline	4.3
D02-01R	17+924	6.6 RT	NBL	DCPT, right toe of slope	2.3
D02-02L	17+938.5	8.6 LT	NBL	DCPT, left toe of slope	8.8
D02-03	17+926	0.6 RT	NBL	DCPT, centreline	9.1
04-01	17+938	CL	SBL	centreline	3.2
04-02L	17+943.5	8.7 LT	SBL	left toe of slope	8.4
04-03	17+951	CL	SBL	centreline	6.1
04-04R	17+963.5	8.5 RT	SBL	right toe of slope	2.4
04-05	17+971	CL	SBL	centreline	2.3
04-06R	17+940	9.0 RT	SBL	right toe of slope	5.8

Table A1

Borehole Summary

Borehole	Station	Offset (m)	Direction	Description	Depth (m)
04-07L	17+980	9.0 LT	SBL	left toe of slope	5.8
D04-01R	17+938.5	8.7 RT	SBL	DCPT, right toe of slope	12.6
D04-02L	17+963.5	8.5 LT	SBL	DCPT, left toe of slope	6.6
D04-05	17+971	0.6 RT	SBL	DCPT, centreline	2.3
Hwy 69 NBL, Sta. 18+180 to 18+230 Harrison					
03-01	18+180	CL	NBL	centreline	0.1
03-02L	18+192.5	7.6 LT	NBL	left toe of slope	0.1
03-03	18+205	CL	NBL	centreline	2.4
03-04R	18+217	5.0 RT	NBL	right toe of slope	0.7
03-05	18+230	2.0 LT	NBL	centreline	0.6
D03-01R	18+192.5	7.6 RT	NBL	DCPT, right toe of slope	1.0
D03-02L	18+217	8.0 LT	NBL	DCPT, left toe of slope	1.8
Naiscoot Access Road, Sta. 10+555 to 10+590					
05-01	10+555	CL	NBL/SBL	centreline	1.4
05-02L	10+562.5	9.0 LT	NBL/SBL	left toe of slope	0.8
05-03	10+575	CL	NBL/SBL	centreline	0.7
05-04	10+590	CL	NBL/SBL	centreline	04
D05-01R	10+562.5	9.0 RT	NBL/SBL	DCPT, right toe of slope	0.8
Hwy 69 NBL & SBL, Sta. 18+240 to 18+500 Harrison					
08-01	18+301	CL	NBL	on existing Hwy 69	6.5
08-02L	18+313.5	7.5 LT	NBL	on existing Hwy 69	3.7
08-03	18+326	CL	NBL	centreline	7.6
08-04L	18+338.5	8.9 RT	NBL	right toe of slope	6.5
08-05	18+351	CL	NBL	centreline	8.3
08-06L	18+363.5	9.2 LT	NBL	left toe of slope	9.9
08-07	18+376	CL	NBL	centreline	6.8
08-08R	18+388.5	9.4 RT	NBL	right toe of slope	6.2
08-09	18+401	CL	NBL	centreline	7.0
08-10L	18+413.5	9.4 LT	NBL	left toe of slope	8.2
08-11	18+426	CL	NBL	centreline	7.9
08-12R	18+438.5	9.4 RT	NBL	right toe of slope	5.3
08-13	18+451	CL	NBL	centreline	5.4
08-14L	18+463.5	9.3 LT	NBL	left toe of slope	4.5
08-15	18+476	CL	NBL	centreline	1.5
08-16R	18+488.5	9.0 RT	NBL	right toe of slope	2.8
08-17	18+501	CL	NBL	centreline	0.1
08-18	18+505	CL	NBL	centreline	0.2
08-19	18+278	16.5 LT	SBL	left toe of slope	2.2
08-20	18+288	CL	SBL	centreline	2.8
08-21	18+299	14.5 RT	SBL	right toe of slope	4.1
08-22	18+372	CL	SBL	centreline	5.2
08-23	18+411	15.0 LT	SBL	left toe of slope	7.3

Table A1

Borehole Summary

Borehole	Station	Offset (m)	Direction	Description	Depth (m)
08-24	18+303	14.0 LT	NBL	on existing Hwy 69	6.0
08-25	18+397	CL	SBL	on existing Hwy 69	10.4
08-26	18+426	4.0 LT	SBL	on existing Hwy 69	7.8
08-27	18+240	15.0 RT	NBL	on existing Hwy 69	5.6
08-28	18+272	8.5 RT	NBL	on existing Hwy 69	7.2
08-29	18+331	6.0 LT	NBL	on existing Hwy 69	8.0
08-30	18+360	12.5 LT	NBL	on existing Hwy 69	10.7
08-31	18+388	18.5 LT	NBL	on existing Hwy 69	10.7
08-32	18+249	32.5 RT	NBL	right toe of slope	7.5
08-33	18+286	25.5 RT	NBL	right toe of slope	4.8
D08-01R	18+313.5	7.5 RT	NBL	DCPT, right toe of slope	10.5
D08-02L	18+338.5	3.9 LT	NBL	DCPT, left toe of slope	8.4
D08-03R	18+363.5	9.2 RT	NBL	DCPT, right toe of slope	5.3
D08-04L	18+388.5	9.4 LT	NBL	DCPT, left toe of slope	7.7
D08-05R	18+413.5	9.4 RT	NBL	DCPT, right toe of slope	7.6
D08-06L	18+438.5	9.4 LT	NBL	DCPT, left toe of slope	7.8
D08-07R	18+463.5	9.3 RT	NBL	DCPT, right toe of slope	4.0
D08-08L	18+488.5	9.4 LT	NBL	DCPT, left toe of slope	0.8
Hwy 69 SBL, Sta. 18+890 to 18+990 Harrison					
09-01	18+926	CL	SBL	centreline	0.2
09-02L	18+938.5	16.8 LT	SBL	left toe of slope	0.5
09-03	18+951	CL	SBL	centreline	1.9
09-04R	18+963.5	17.9 RT	SBL	right toe of slope	0.5
09-05	18+976	CL	SBL	centreline	1.2
09-06L	18+986	9.7 LT	SBL	left toe of slope	0.0
09-07R	18+913.5	18.0 RT	SBL	right toe of slope	1.6
09-08L	18+963.5	18.0 LT	SBL	left toe of slope	1.7
D09-01R	18+938.5	16.8 RT	SBL	DCPT, right toe of slope	0.9
D09-02L	18+963.5	17.9 LT	SBL	DCPT, left toe of slope	5.1
D09-03R	18+986	9.7 RT	SBL	DCPT, right toe of slope	0.0
D09-04R	18+888.5	18.0 RT	SBL	DCPT, right toe of slope	1.6
Hwy 69 NBL & SBL, Sta. 19+285 to 19+350 Harrison					
10-01	19+286	CL	NBL	centreline	2.9
10-02	19+301	CL	NBL	centreline	5.2
10-03L	19+313.5	15.3 LT	NBL	left toe of slope	9.9
10-04	19+326	CL	NBL	centreline	18.3
10-05R	19+338.5	15.1 RT	NBL	right toe of slope	18.3
10-06	19+341	CL	NBL	centreline	13.7
D10-01R	19+313.5	15.3 RT	NBL	DCPT, right toe of slope	10.0
D10-02L	19+338.5	15.1 LT	NBL	DCPT, left toe of slope	15.3
11-01	19+291	CL	SBL	centreline	0.5

Table A1

Borehole Summary

Borehole	Station	Offset (m)	Direction	Description	Depth (m)
11-02	19+301	CL	SBL	centreline	2.6
11-03L	19+313.5	15.3 LT	SBL	left toe of slope	5.4
11-04	19+326	CL	SBL	centreline	11.3
11-05R	19+338.5	15.1 RT	SBL	right toe of slope	10.8
11-06	19+341	CL	SBL	centreline	3.4
D11-01R	19+313.5	15.3 RT	SBL	DCPT, right toe of slope	6.4
D11-02L	19+332.5	15.1 LT	SBL	DCPT, left toe of slope	2.6
C313-1	19+305	22.0 RT	NBL	culvert	9.4
C313-2	19+303	2.0 RT	NBL	culvert	4.8
C313-3	19+301	17.0 LT	NBL	culvert	4.0
C314-1	19+299	2.0 LT	SBL	culvert	2.1
C314-2	19+297	22.0 LT	SBL	culvert	3.8
Hwy 69 SBL, Sta. 19+455 to 19+555 Harrison					
12-01	19+456	CL	SBL	centreline	0.5
12-02L	19+463.5	6.3 LT	SBL	left toe of slope	0.0
12-03	19+476	CL	SBL	centreline	2.7
12-04R	19+488.5	6.3 RT	SBL	right toe of slope	2.2
12-05	19+496	CL	SBL	centreline	2.5
12-06L	19+513.5	7.5 LT	SBL	left toe of slope	4.5
12-07	19+526	CL	SBL	centreline	3.8
12-08R	19+546.5	6.5 RT	SBL	right toe of slope	0.6
12-09	19+546	CL	SBL	centreline	0.5
D12-01R	19+463.5	6.3 RT	SBL	DCPT, right toe of slope	0.0
D12-02L	19+488.5	6.3 LT	SBL	DCPT, left toe of slope	3.7
D12-03R	19+513.5	7.5 RT	SBL	DCPT, right toe of slope	5.0
D12-04L	19+538.5	6.5 LT	SBL	DCPT, left toe of slope	3.3
Hwy 69 NBL & SBL, Sta. 19+680 to 19+870 Harrison					
13-01	19+756	CL	NBL	centreline	4.4
13-02L	19+763.5	13.0 LT	NBL	left toe of slope	7.4
13-003	19+776	CL	NBL	centreline	3.8
13-04R	19+788.5	12.8 RT	NBL	right toe of slope	1.1
13-05	19+801	CL	NBL	centreline	3.9
13-06L	19+813.5	12.6 LT	NBL	left toe of slope	4.3
13-07	19+826	CL	NBL	centreline	1.8
13-08R	19+836	12.4 RT	NBL	right toe of slope	1.5
13-09	19+846	CL	NBL	centreline	3.3
13-10	19+681	CL	NBL	centreline	1.4
13-11R	19+693.5	13.0 RT	NBL	right toe of slope	0.9
13-12	19+706	CL	NBL	centreline	1.8
13-13L	19+718	13.0 LT	NBL	left toe of slope	4.6
13-14	19+731	CL	NBL	centreline	1.7

Table A1

Borehole Summary

Borehole	Station	Offset (m)	Direction	Description	Depth (m)
13-15R	19+743.5	12.8 RT	NBL	right toe of slope	0.6
D13-01R	19+763.5	13.0 RT	NBL	DCPT, right toe of slope	2.5
D13-02L	19+788.5	12.8 LT	NBL	DCPT, left toe of slope	8.8
D13-03R	19+813.5	12.6 RT	NBL	DCPT, right toe of slope	0.3
D13-04L	19+836	12.4 LT	NBL	DCPT, left toe of slope	2.8
D13-05L	19+693.5	14.0 LT	NBL	DCPT, left toe of slope	1.5
D13-06R	19+718.5	13.0 RT	NBL	DCPT, right toe of slope	2.3
D13-07L	19+742.5	13.0 LT	NBL	DCPT, left toe of slope	4.9
14-01	19+746	CL	SBL	centreline	0.5
14-02	19+751	CL	SBL	centreline	5.3
14-03L	19+763.5	13.1 LT	SBL	left toe of slope	12.3
14-04	19+776	CL	SBL	centreline	5.6
14-05R	19+788.5	13.0 RT	SBL	right toe of slope	7.4
14-06	19+801	CL	SBL	centreline	3.8
14-07L	19+813.5	12.9 LT	SBL	left toe of slope	2.3
14-08	19+826	CL	SBL	centreline	4.8
14-09R	19+838.5	12.0 RT	SBL	right toe of slope	5.7
14-10	19+851	CL	SBL	centreline	2.8
14-11L	19+863.5	10.9 LT	SBL	left toe of slope	0.7
14-12	19+871	CL	SBL	centreline	2.9
14-13	19+726	CL	SBL	centreline	1.7
14-14R	19+735.5	13.1 RT	SBL	right toe of slope	2.1
D14-01R	19+763.5	13.1 RT	SBL	DCPT, right toe of slope	11.4
D14-02L	19+788.5	13.0 LT	SBL	DCPT, left toe of slope	4.1
D14-03R	19+813.5	12.9 RT	SBL	DCPT, right toe of slope	3.9
D14-04L	19+838.5	12.0 LT	SBL	DCPT, left toe of slope	5.2
D14-05R	19+863.5	10.9 RT	SBL	DCPT, right toe of slope	4.2
D14-06R	19+713.5	13.1 RT	SBL	DCPT, right toe of slope	2.7
D14-07L	19+738.5	13.0 LT	SBL	DCPT, left toe of slope	2.9
C317-1	19+830	18.0 RT	NBL	culvert	1.2
C317-2	19+830	18.0 LT	NBL	culvert	4.3
C318-1	19+830	18.0 LT	SBL	culvert	2.4

Table A2

Piezometer Installation Details

Borehole	Piezometer Tip Depth (m)	Installation Details
02-03	8.2	Piezometer with 1.5 m slotted screen installed, sand filter from 8.2 to 6.3 m, bentonite seal from 6.3 m to ground surface.
03-03	2.4	Piezometer with 1.5 m slotted screen installed, sand filter from 2.4 to 1.6 m, bentonite seal from 1.6 m to ground surface.
04-02L	8.4	Piezometer with 1.5 m slotted screen installed, sand filter from 8.4 to 6.0 m, bentonite seal from 6.0 m to ground surface.
08-07	6.8	Piezometer with 1.5 m slotted screen installed, sand filter from 6.8 to 5.0 m, bentonite seal from 5.0 m to ground surface.
08-16R	2.4	Piezometer with 1.5 m slotted screen installed, sand filter from 2.4 to 1.6 m, bentonite seal from 1.6 m to ground surface.
08-23	5.6	Piezometer with 1.5 m slotted screen installed, sand filter from 5.6 to 3.2 m, bentonite seal from 3.2 m to ground surface.
08-33	4.3	Piezometer with 1.5 m slotted screen installed, sand filter from 4.3 to 2.5 m, bentonite seal from 2.5 to 0.2 m, cuttings to ground surface.
12-07	3.8	Piezometer with 1.5 m slotted screen installed, sand filter from 3.8 to 2.1 m, bentonite seal from 2.1 m to ground surface.
13-06L	4.3	Piezometer with 1.5 m slotted screen installed, sand filter from 4.3 to 2.6 m, bentonite seal from 2.6 m to ground surface.
13-13L	4.6	Piezometer with 1.5 m slotted screen installed, sand filter from 4.6 to 2.4 m, bentonite seal from 2.4 to 2.1 m, cuttings from 2.1 to 0.6 m, bentonite seal from 0.6 to 0.3 m, cuttings to ground surface.
14-02	5.3	Piezometer with 1.5 m slotted screen installed, sand filter from 5.3 to 3.5 m, bentonite seal from 3.5 m to ground surface.
C317-2	4.3	Piezometer with 1.5 m slotted screen installed, sand filter from 4.3 to 2.5 m, bentonite seal from 2.5 to 2.0 m, cuttings from 2.0 to 0.6 m, bentonite seal from 0.6 to 0.3 m, cuttings to ground surface.

Table A3

Summary of Swamp Crossings/High Fill Embankment Locations and Conditions

Appendix	Alignment	Stations	Length of Section (m)	Section Type	Maximum Embankment Height (m)	Boreholes and Cones (D)	Generalized Stratigraphy	Groundwater Conditions
B	Hwy 69 NBL & SBL	17+425 to 17+560	135	High Fill	9.4	01-01 to 01-16 D01-01R to D01-07R	ORGANICS, 0.05 to 0.3 m thick; over SAND, trace silt to silty, loose to dense, 0.1 to 1.4 m thick; over/under SILT, clayey, stiff to very stiff, 1.0 to 1.4 m thick (three boreholes); over BEDROCK at 0.0 to 3.8 m depth.	0.3 to 0.9 m depth in three boreholes upon completion of drilling
C	Hwy 69 NBL & SBL	17+900 to 18+000	100	Swamp Crossing	2.0	02-01 to 02-05 02-07L, 02-08 D02-01R to D02-03 04-01 to 04-07L D04-01R D04-02L, D04-05	PEAT and ORGANICS, 0.5 to 5.1 m thick; over SAND, trace silt to silty, very loose to loose, 0.6 to 3.8 m thick; over PROBABLE BEDROCK at 0.5 to 11.5 m depth.	Site overlain by up to 1.1 m of ice and water during fieldwork; water level at ground surface in piezometers
D	Hwy 69 NBL	18+180 to 18+230	50	Swamp Crossing	1.5	03-01 to 03-05 D03-01R D03-02L	PEAT and ORGANICS, 0.1 to 0.9 m thick; over CLAY, silty, very soft, 0.5 m thick (one borehole); over SANDS and SILTS, very loose to compact, 0.7 to 1.0 m thick; over PROBABLE BEDROCK at 0.1 to 2.4 m depth.	0.6 m depth in one borehole upon completion of drilling; at ground surface in piezometer
E	Naiscoot Access Road	10+555 to 10+590	35	Swamp Crossing	2.5	05-01 to 05-04 D05-01R	PEAT, 0.1 to 0.6 m thick; and/or SAND, loose, 0.2 to 0.7 m thick; over BEDROCK at 0.1 to 0.8 m depth.	Site overlain by up to 0.8 m of ice and water during fieldwork
F	Hwy 69 NBL & SBL	18+240 to 18+500	260	Swamp Crossing	3.0	08-01 to 08-33 D08-01R to D08-08L	PAVEMENT STRUCTURE and FILL (on existing highway platform), sand and gravel, loose to compact, 2.1 to 8.5 m thick; or PEAT, 0.1 to 5.9 m thick; over SILT, sandy, very loose, 1.1 to 1.7 m thick (two boreholes); over CLAY, silty, very soft, 0.3 to 2.9 m thick, and/or SAND, trace silt to silty, very loose to loose, 0.2 to 3.2 m thick, over PROBABLE BEDROCK at 0.1 to 10.7 m depth.	Most of the site was overlain by up to 0.8 m of ice and water during fieldwork; water at ground surface in piezometers
G	Hwy 69 SBL	18+890 to 18+990	100	High Fill	9.1	09-01 to 09-08L D09-01R to D09-04R	ORGANICS, 0.1 to 0.6 m thick; over SAND, loose to dense, 0.3 to 1.3 m thick; or CLAY, silty, very soft to very stiff, 1.5 to 1.6 m thick; over BEDROCK at 0.0 to 5.1 m depth.	Water at 0.3 m depth in one borehole; no water observed in remaining boreholes during fieldwork
H	Hwy 69 NBL & SBL	19+285 to 19+350	65	Pond	8.6	10-01 to 10-06 D10-01R D10-02L 11-01 to 11-06 D11-01R D11-02L C313-1 to C313-3 C314-1, C314-2	CLAY, silty, very soft, 0.2 to 1.1 m thick (six boreholes); over PEAT, 0.6 to 4.3 m thick; over CLAY, silty, very soft, 0.8 to 7.5 m thick; and/or SAND, trace silt to silty, very loose to dense, 0.5 to 5.5 m thick; over GRAVEL to SAND, gravelly, loose to very dense, 0.8 to 5.0 m thick, PROBABLE BEDROCK at 0.0 to 16.8 m depth.	Site overlain by up to 1.8 m of ice and water during fieldwork

Table A3

Summary of Swamp Crossings/High Fill Embankment Locations and Conditions

Appendix	Alignment	Stations	Length of Section (m)	Section Type	Maximum Embankment Height (m)	Boreholes and Cones (D)	Generalized Stratigraphy	Groundwater Conditions
I	Hwy 69 SBL	19+455 to 19+555	100	Swamp Crossing	2.3	12-01 to 12-09 D12-01R to D12-04L	PEAT and ORGANICS, 0.1 to 2.1 m thick; over CLAY, silty, very soft to soft, 0.8 to 2.4 m thick; and/or SAND, silty to SILT, sandy, very loose to loose, 0.5 to 0.8 m thick, and/or SAND, gravelly, 0.3 m thick; over PROBABLE BEDROCK at 0.0 to 5.0 m depth.	Most of the site was overlain by up to 1.4 m of ice and water during fieldwork; water at ground surface in piezometer
J	Hwy 69 NBL & SBL	19+680 to 19+870	190	Swamp Crossing	6.0	13-01 to 13-15R D13-01R to D13-07L 14-01 to 14-14R D14-01R to D14-07L C317-1, C317-2 C318-1	PEAT and TOPSOIL, 0.1 to 0.8 m thick; over SAND, trace silt to silty, loose to very loose, 0.4 to 1.2 m thick, and/or CLAY, silty, very soft to very stiff, 1.2 to 9.2 m thick; over SAND, trace silt to silty, very loose to very dense, 0.2 to 3.1 m thick; or SAND and GRAVEL, loose to compact, 0.8 to 1.4 m thick; over PROBABLE BEDROCK at 0.3 to 12.3 m depth.	0.0 to 2.4 m depth upon completion of drilling; at ground surface in piezometers

Table A4

Anticipated/Recommended Depth of Peat and Soft Soil Subexcavation

Appendix	Alignment	Stations	Maximum Embankment Height (m)	Maximum Depth of Subexcavation to Remove Peat, Organics and Soft Soil (m)
B	Hwy 69 NBL	17+425 to 17+560	9.4	0.3
C	Hwy 69 NBL & SBL	17+900 to 18+000	2.0	5.4
D	Hwy 69 NBL	18+180 to 18+230	1.5	1.4
E	Naiscoot Access Road	10+555 to 10+590	2.5	0.6
F	Hwy 69 NBL & SBL	18+240 to 18+500	3.0	8.1
G	Hwy 69 SBL	18+890 to 18+990	9.1	0.7
H	Hwy 69 NBL & SBL	19+285 to 19+350	8.6	11.5
I	Hwy 69 SBL	19+455 to 19+555	2.3	5.0
J	Hwy 69 NBL & SBL	19+680 to 19+870	6.0	10.1

Table A5
Results of Stability Analyses

Appendix	Alignment	Analysis Profile Location	Embankment Height (m)	Condition	Computed Factor of Safety	Figures in Appendix A
F	Hwy 69 NBL &SBL	18+240 to 18+500	3.0	Short-term (undrained analysis)	1.93	69HWY001
				Long-term (drained analysis)	1.88	69HWY005
				Short-term with 2 m surcharge	1.31	69HWY002
				Immediately before surcharge removal	1.28	69HWY004
H	Hwy 69 NBL & SBL	19+285 to 19+350	8.6	Short-term (undrained analysis)	< 1.0	-
				Short-term with 17 m wide stabilizing berm	< 1.0	69HWY006
				Staged construction with stabilizing berm		
				Stage 1 - short-term	1.34	69HWY007
				Stage 2 - short-term	1.28	69HWY008
				Long-term – after surcharge removal	2.56	69HWY009
				Partial subexcavation of clay and provision of stabilizing berm (one stage, no surcharge)		
				Short-term	1.31	69HWY010
				Long-term	1.51	69HWY011

Table A5
Results of Stability Analyses

Appendix	Alignment	Analysis Profile Location	Embankment Height (m)	Condition	Computed Factor of Safety	Figures in Appendix A
J	Hwy 69 NBL & SBL	19+680 to 19+870	6.0	Areas with clay thickness less than 4.1 m:		
				Short-term (undrained analysis)	1.58	69HWY012
				Long-term (drained analysis)	1.50	69HWY013
				Area of deep clay:		
				Short-term (undrained analysis)	< 1.0	69HWY014
				Short-term with partial clay subexcavation	1.05	69HWY015
				Staged construction with partial clay subexcavation		
				Stage 1 - short-term	1.36	69HWY016
				Stage 2 - short-term	1.30	69HWY017
				Long-term – after surcharge removal	1.73	69HWY018

Table A6

Results of Settlement Analyses

Appendix	Alignment	Analysis Profile Location (Sta.)	Embankment Height (m)	Foundation Soil Type*	Elastic Settlement (mm)	Primary Consolidation		Secondary Compression (mm)	Embankment Compression (mm)		Total Post-Construction Settlement (mm)
						Settlement (mm)	Time to 98% Consolidation (months)		Short-term	Long-term	
B	Hwy 69 NBL & SBL	17+425 to 17+560	9.4	Up to 3.8 m of loose to compact sand and stiff to very stiff clay over bedrock	30	-	-	-	70	10	< 25
C	Hwy 69 NBL & SBL	17+900 to 18+000	2.0	Up to 3.8 m of loose to very loose silty sand over bedrock	50	-	-	-	95	15	< 25
D	Hwy 69 NBL	18+180 to 18+230	1.5	Up to 1.0 m of sand and silt over bedrock	< 25	-	-	-	15	< 10	< 25
E	Naiscoot Access Road	10+555 to 10+590	2.5	Up to 0.7 m of sand over bedrock	< 25	-	-	-	15	< 10	< 25
F	Hwy 69 NBL & SBL	18+240 to 18+500	3.0	Up to 2.9 m of very soft silty clay and/or up to 3.2 m of sand to silty sand, over bedrock OR ***As above but with complete subexcavation of soft clay***	< 25 < 25	125 (no surcharge) 250 (2m surcharge) -	4.5 10 -	90 5 -	110 110 140	15 15 20	105 < 25 < 25
G	Hwy 69 SBL	18+890 to 18+990	9.1	Up to 1.3 m of sand over bedrock	< 25	-	-	-	75	10	< 25
H	Hwy 69 NBL & SBL	19+285 to 19+350	8.6	Up to 7.5 m of silty clay and/or up to 5.5 m of sand to silty sand, and up to 5.0 m of gravel to gravelly sand, over bedrock OR As above but with partial (4m) subexcavation of clay (up to 3.6 m of clay remaining) OR ***As above but with complete subexcavation of clay***	< 25 40 50	420 (no surcharge) 490 (2m surcharge) 95 (no surcharge) -	10 18 4 -	120 15 30 -	190 190 240 275	20 20 25 30	140 35 55 30

* Note: Where more than one option is shown, the recommended foundation type/treatment is italicized/bold.

THURBER

Table A6

Results of Settlement Analyses

Appendix	Alignment	Analysis Profile Location (Sta.)	Embankment Height (m)	Foundation Soil Type*	Elastic Settlement (mm)	Primary Consolidation		Secondary Compression (mm)	Embankment Compression (mm)		Total Post-Construction Settlement (mm)
						Settlement (mm)	Time to 98% Consolidation (months)		Short-term	Long-term	
I	Hwy 69 SBL	19+455 to 19+555	2.3	Up to 2.4 m of silty clay and/or 0.8 m of sandy silt to silty sand, over bedrock	< 25	-	-	-	75	10	< 25
J	Hwy 69 NBL & SBL	19+750 to 19+800	6.0	Up to 9.2 m of silty clay over up to 3.1 m of silty sand to sand or 1.4 m of sand and gravel, over bedrock	< 25	700 (no surcharge) 950 (2m surcharge)	48 84	150 5	55 55	< 10 < 10	160 < 25
				OR As above but with partial (4m) subexcavation of clay (up to 5.2 m of clay remaining)	< 25	525 (no surcharge) 680 (2m surcharge)	15 (one stage) 21 (two stage)	125 15	150 150	15 15	140 30
				OR ***As above but with complete subexcavation of clay***	< 25	-	-	-	255	25	25
		19+680 to 19+750 19+800 to 19+870	6.0	Up to 4.1 m of silty clay over up to 3.1 m of silty sand to gravelly sand, over bedrock	< 25	60	3	60	45	< 10	60
				OR ***As above but with complete subexcavation of clay***	< 25	-	-	-	150	15	< 25

* Note: Where more than one option is shown, the recommended foundation type/treatment is italicized/bold.

Table A7

Summary of Recommendations for Embankment Construction and Swamp Treatments

Appendix	Alignment	Stations	Length of Section (m)	Section Type	Maximum Embankment Height or Cut Depth (m)	Summarized Recommendations
B	Hwy 69 NBL	17+425 to 17+560	135	High Fill	9.4	**Stability:** Standard embankment sideslopes of 1.25H:1V in rock fill and 2H:1V in earth fill in conjunction with mid-height berms should be stable. **Settlement:** Settlement of embankment foundations is not an issue. Embankment construction should be carried out at least six months in advance of pavement construction to allow for compression of embankment materials.
C	Hwy 69 NBL & SBL	17+900 to 18+000	100	Swamp Crossing	2.0	**Swamp Treatment:** Excavate peat and replace with rock fill prior to embankment construction. Maximum anticipated depth of peat is 5.4 m. **Stability:** Standard embankment sideslopes of 1.25H:1V in rock fill and 2H:1V in earth fill should be stable. **Settlement:** Settlement of foundation soils is expected to occur essentially as embankment construction proceeds. Embankment construction should be carried out at least six months in advance of pavement construction to allow for compression of embankment materials.
D	Hwy 69 NBL	18+180 to 18+230	50	Swamp Crossing	1.5	**Swamp Treatment:** Excavate peat, organics and very soft silty clay prior to embankment construction. Maximum anticipated depth of peat and very soft silty clay is 1.4 m. **Stability:** Standard embankment sideslopes of 1.25H:1V in rock fill and 2H:1V in earth fill should be stable. **Settlement:** Settlement of foundation soils is expected to occur essentially as embankment construction proceeds. Embankment construction should be carried out at least six months in advance of pavement construction to allow for compression of embankment materials.
E	Naiscoot Access Road	10+555 to 10+590	35	Swamp Crossing	2.5	**Swamp Treatment:** Excavate peat prior to embankment construction. Maximum anticipated depth of peat is 0.6 m. **Stability:** Standard embankment sideslopes of 1.25H:1V in rock fill and 2H:1V in earth fill should be stable. **Settlement:** Settlement of foundation soils is expected to occur essentially as embankment construction proceeds. Embankment construction should be carried out at least six months in advance of pavement construction to allow for compression of embankment materials.
F	Hwy 69 NBL & SBL	18+240 to 18+500	260	Swamp Crossing	3.0	**Swamp Treatment:** Excavate peat and soft clay full depth, and replace with rock fill prior to embankment construction. Maximum anticipated depth of subexcavation is 8.1 m. Will require excavation of peat and clay in short sections to avoid disturbance to existing highway platform. **Stability:** Standard embankment sideslopes of 1.25H:1V in rock fill and 2H:1V in earth fill should be stable provided the peat and soft clay are removed prior to new fill placement. **Settlement:** Settlement of embankment foundations is not an issue provided the existing peat and clay is removed prior to rock fill placement. Embankment construction should be carried out at least six months in advance of pavement construction to allow for compression of embankment materials.
G	Hwy 69 SBL	18+890 to 18+990	100	High Fill	9.1	**Stability:** Standard embankment sideslopes of 1.25H:1V in rock fill and 2H:1V in earth fill in conjunction with mid-height berms should be stable. **Settlement:** Settlement of embankment foundations is not an issue. Embankment construction should be carried out at least six months in advance of pavement construction to allow for compression of embankment materials.

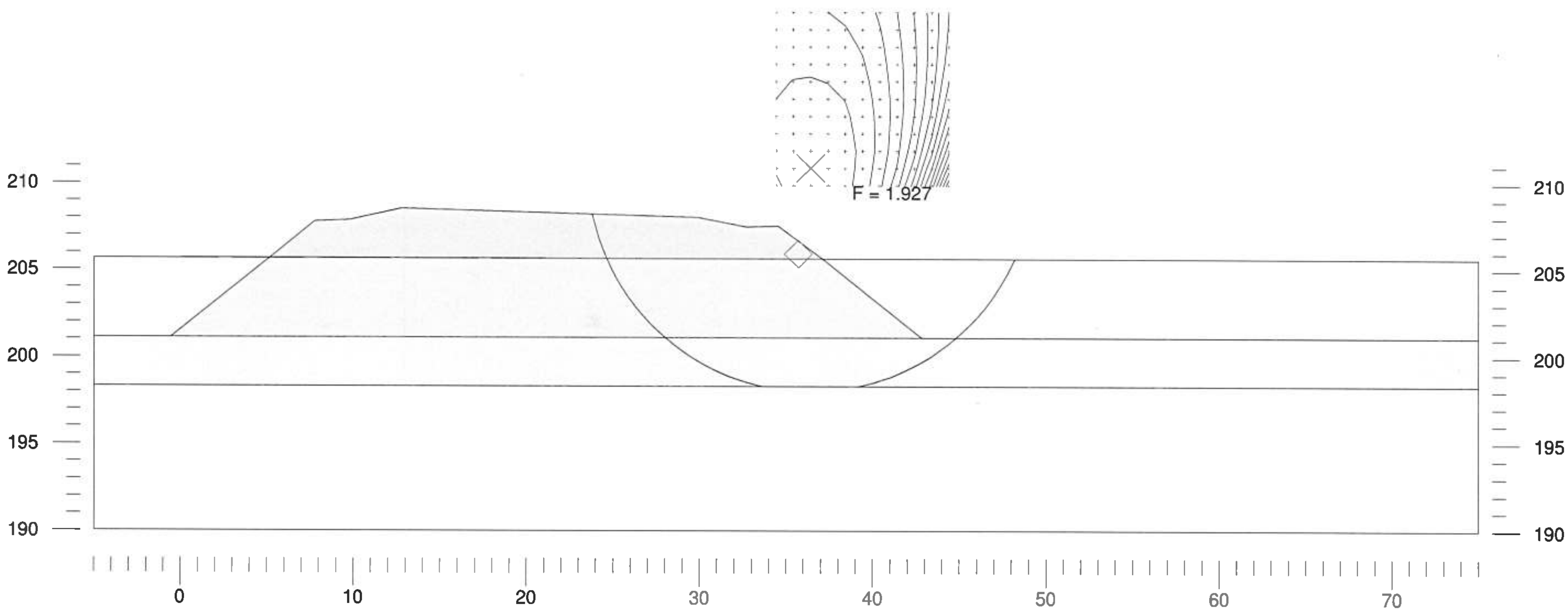
Table A7

Summary of Recommendations for Embankment Construction and Swamp Treatments

Appendix	Alignment	Stations	Length of Section (m)	Section Type	Maximum Embankment Height or Cut Depth (m)	Summarized Recommendations
H	Hwy 69 NBL & SBL	19+285 to 19+350	65	Pond	8.6	**Swamp Treatment:** Excavate peat and soft clay to inorganic cohesionless soil or bedrock, whichever is higher, and replace with rock fill prior to embankment construction. Maximum anticipated depth of subexcavation is 11.5 m below the pond bottom. **Stability:** Standard embankment sideslopes of 1.25H:1V in rock fill and 2H:1V in earth fill should be stable. **Settlement:** Provided embankment construction includes full subexcavation of peat and soft clay as outlined above, post-construction settlement due to compression of the embankment materials is expected to be in the order of 35 mm. Embankment construction should be carried out at least six months in advance of pavement construction to allow for compression of embankment materials.
I	Hwy 69 SBL	19+455 to 19+555	100	Swamp Crossing	2.3	**Swamp Treatment:** Excavate peat, organics and soft silty clay prior to embankment construction. Maximum anticipated depth of peat and soft silty clay is 5.0 m. **Stability:** Standard embankment sideslopes of 1.25H:1V in rock fill and 2H:1V in earth fill should be stable. **Settlement:** Settlement of foundation soils is expected to occur essentially as embankment construction proceeds. Embankment construction should be carried out at least six months in advance of pavement construction to allow for compression of embankment materials.
J	Hwy 69 NBL & SBL	19+680 to 19+870	190	Swamp Crossing	6.0	**Swamp Treatment:** Excavate peat and soft clay to inorganic cohesionless soil or bedrock, whichever is higher, and replace with rock fill prior to embankment construction. Maximum anticipated depth of subexcavation is 10.1 m. **Stability:** Standard embankment sideslopes of 1.25H:1V in rock fill and 2H:1V in earth fill should be stable. **Settlement:** Provided embankment construction includes full subexcavation of peat and soft clay as outlined above, post-construction settlement due to compression of the embankment materials is expected to be in the order of 25 mm. Embankment construction should be carried out at least six months in advance of pavement construction to allow for compression of embankment materials.

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Rockfill	19	0	42	0	1
Peat	13	10	0	0	1
Peat replacement	19	0	42	0	1
Clay	19	20	0	0	1
Bedrock	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-5161-21
Highway 69 Four Laning
January 2011
Total Stress Analysis (Bbar = 0.0)
One stage construction to top of pavement (208.3m)



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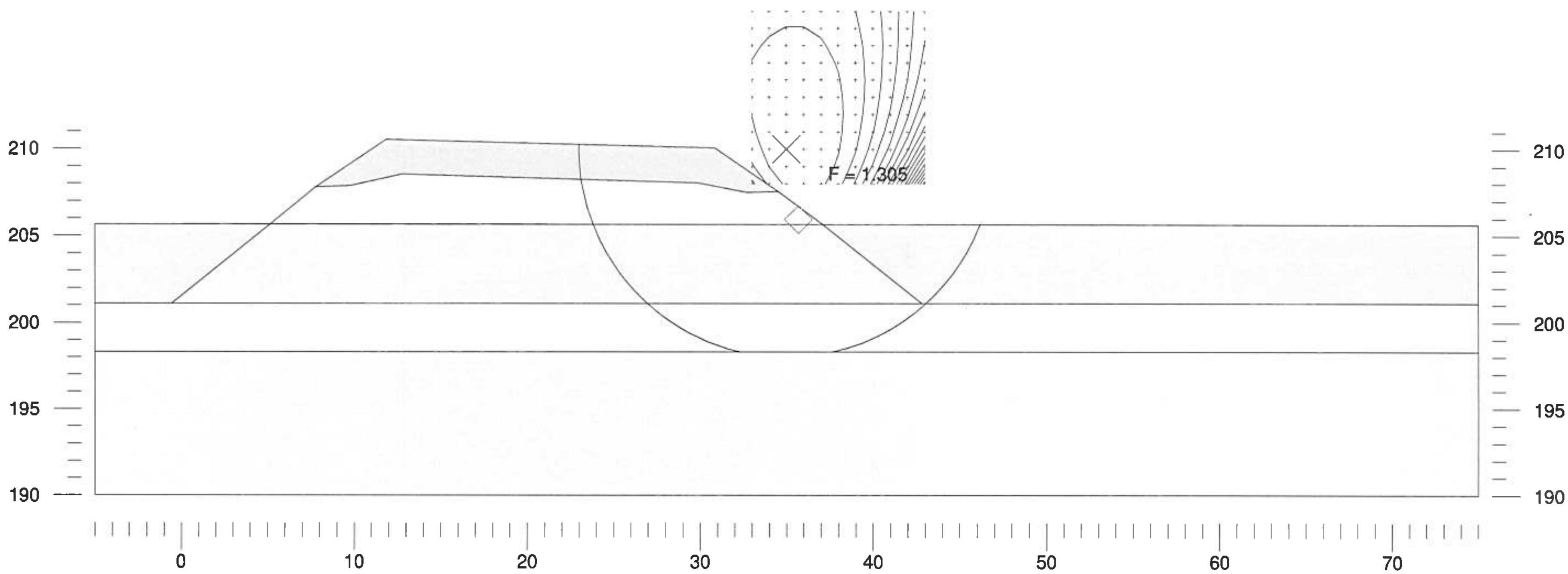
69HWY001

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Surcharge	21	0	32	0	1
Rockfill	19	0	42	0	1
Peat	13	10	0	0	1
Peat replacement	19	0	42	0	1
Clay	19	20	0	0	1
Bedrock	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-5161-21
Highway 69 Four Laning
January 2011
Total Stress Analysis (Bbar = 0.0)
One stage construction to top of surcharge (210.3m)



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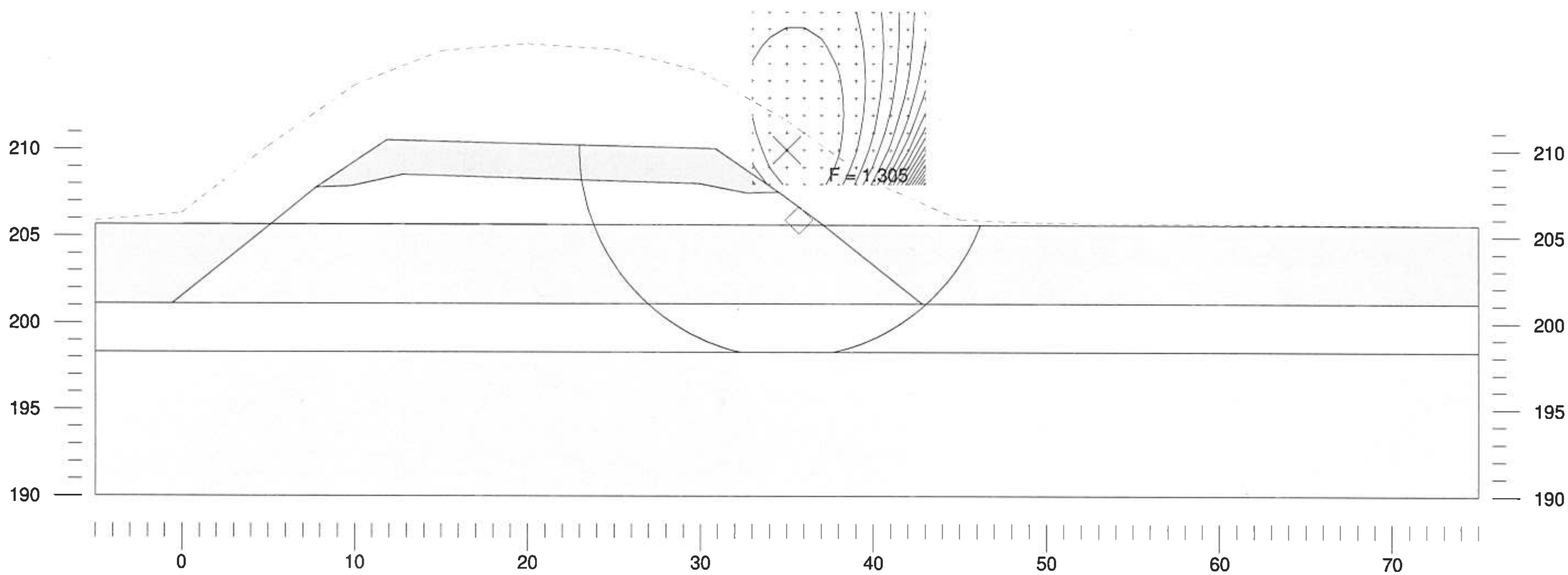
69HWY002

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Surcharge	21	0	32	0	1
Rockfill	19	0	42	0	1
Peat	13	10	0	0	1
Peat replacement	19	0	42	0	1
Clay	19	20	0	.22	2
Bedrock	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-5161-21
Highway 69 Four Laning
January 2011
Undrained Strength Analysis (Bbar = 0.9)
One stage construction to top of surcharge (210.3m)



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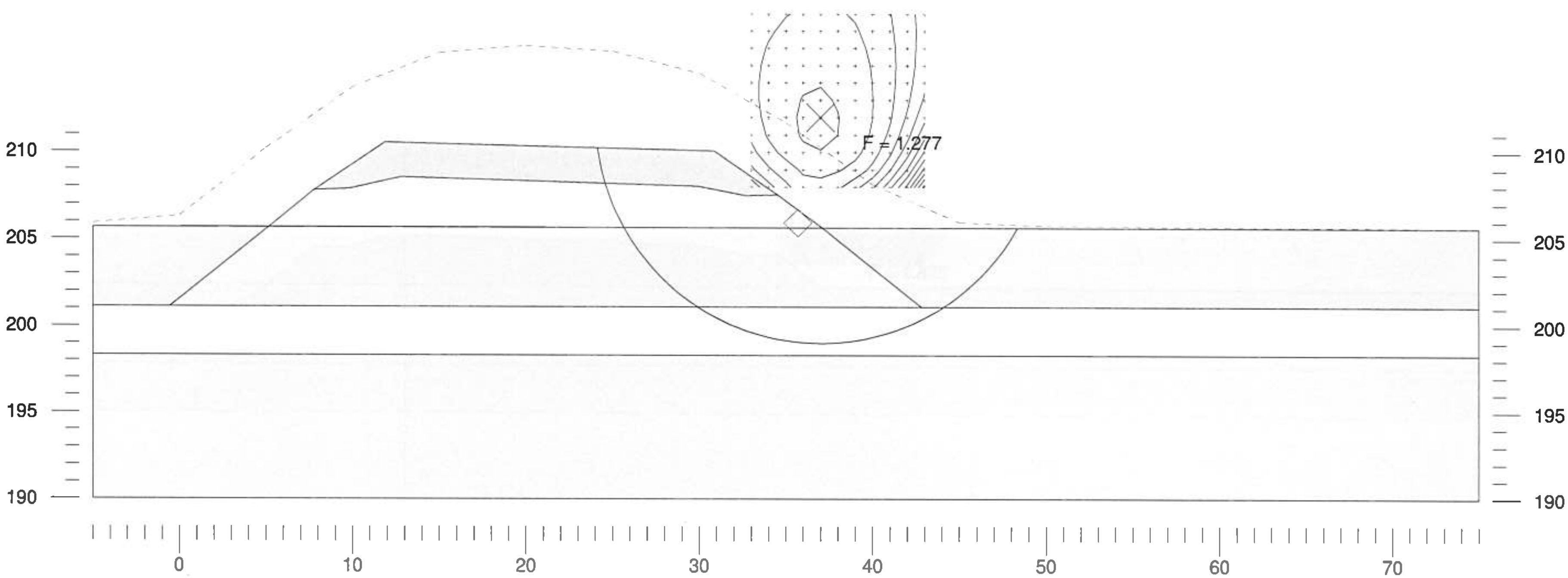
69HWY003

	Gamma	C	Phi	Min	Piezo
	kN/m3	kPa	deg	c/p	Surf.
Surcharge	21	0	32	0	1
Rockfill	19	0	42	0	1
Peat	13	2	29	0	1
Peat replacement	19	0	42	0	1
Clay	19	0	28	0	2
Bedrock	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-5161-21
Highway 69 Four Laning
January 2011
Effective Stress Analysis (Bbar = 0.9)
One stage construction to top of surcharge (210.3m)



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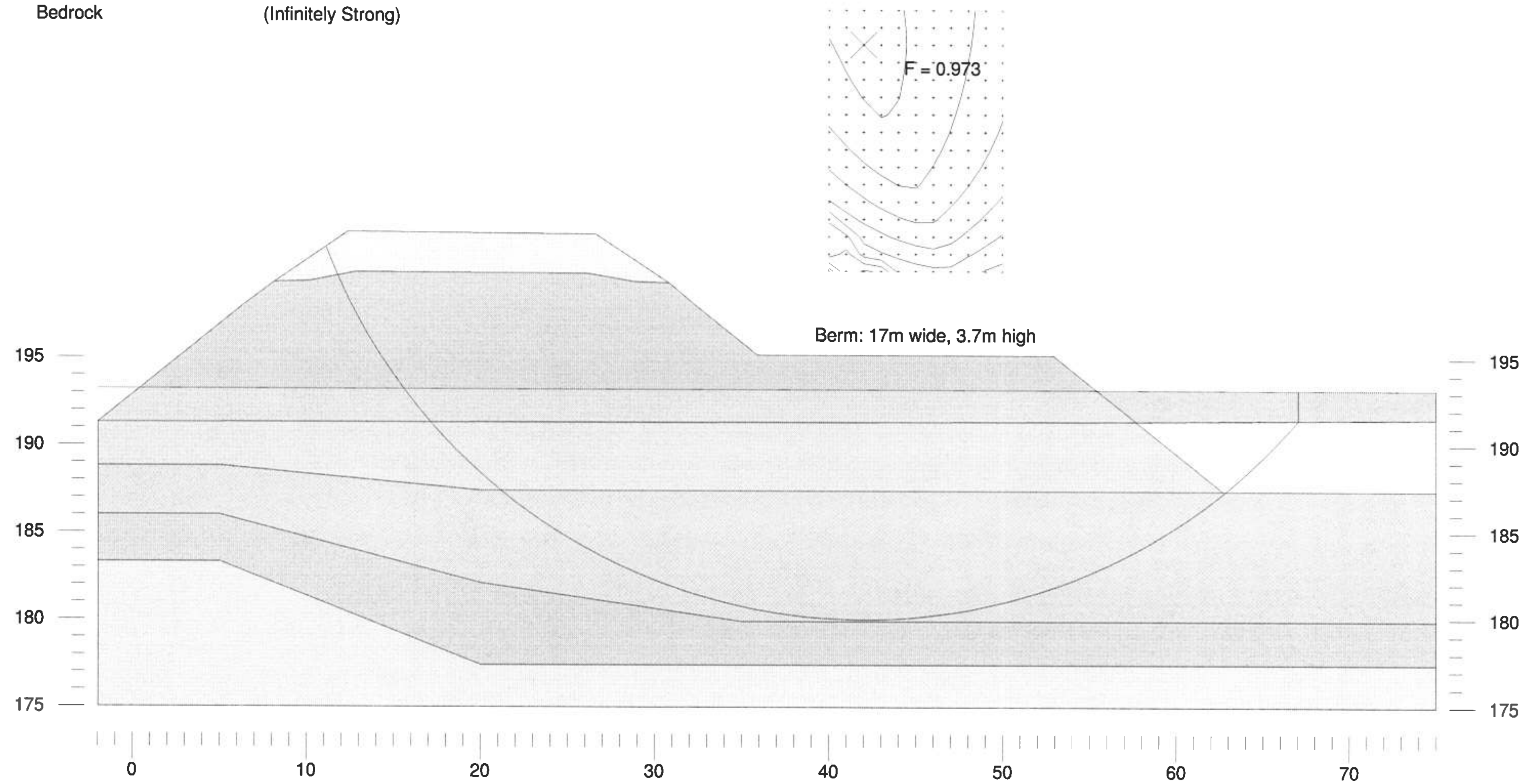
69HWY004

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Water	9.81	0	0	0	1
Surcharge	21	0	32	0	1
Rockfill	19	0	42	0	1
Rockfill (sub.)	19	0	42	0	1
Peat	13	10	0	0	1
Peat Replacement	19	0	42	0	1
Clay	19	20	0	0	1
Sand and Gravel	19	0	38	0	1
Bedrock	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-5161-21
Highway 69 Four Laning
January 2011
Total Stress Analysis (Bbar = 0.0)
One stage of construction (202.0m)



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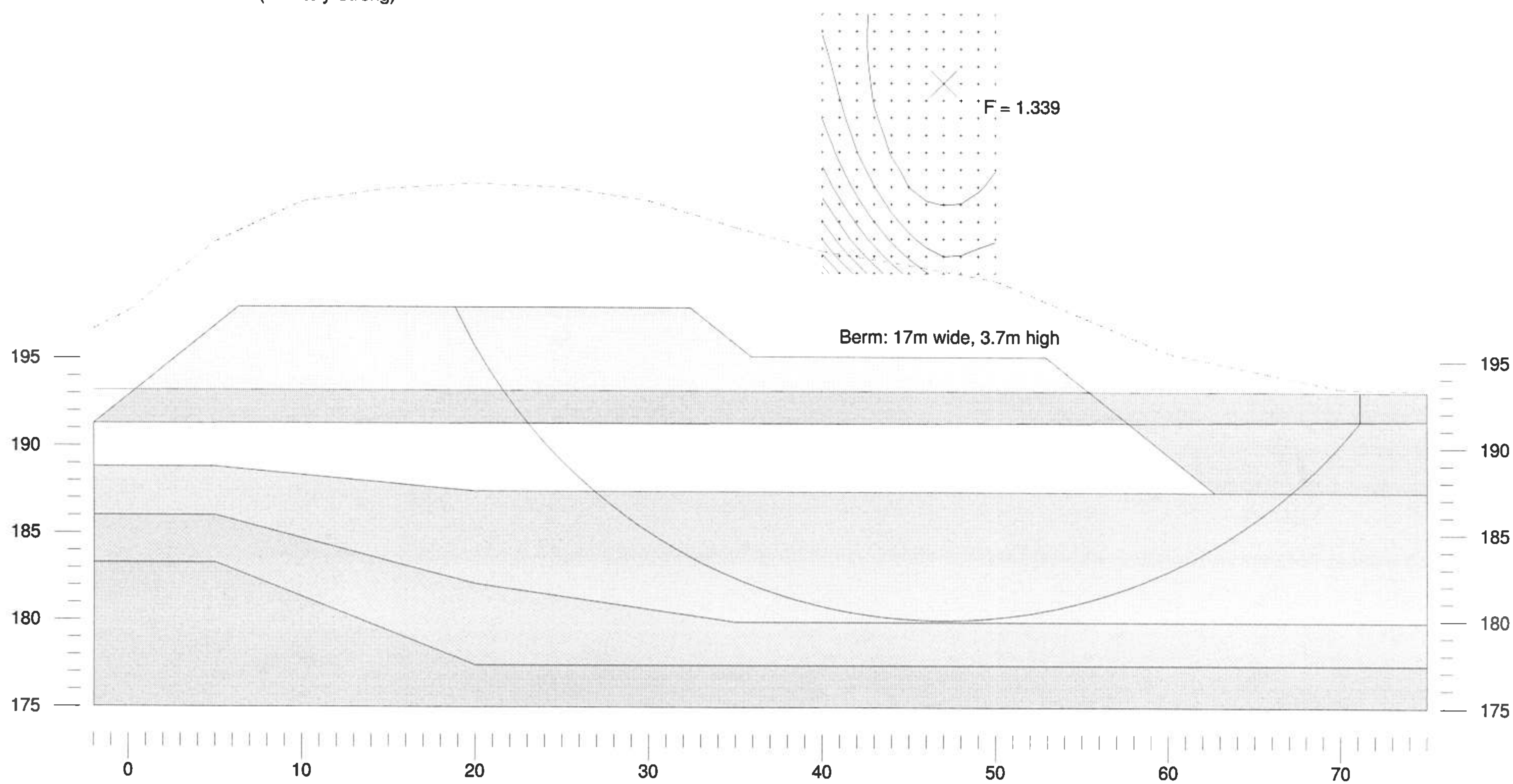
69HWY006

Thurber Engineering Ltd. - Toronto
19-5161-21
Highway 69 Four Laning
January 2011
Undrained Strength Analysis (Bbar = 0.9)
1st stage of construction (198.0m)

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Water	9.81	0	0	0	1
Rockfill	19	0	42	0	1
Rockfill (sub.)	19	0	42	0	1
Peat	13	10	0	0	1
Peat Replacement	19	0	42	0	1
Clay	19	20	0	.22	2
Sand and Gravel	19	0	38	0	1
Bedrock	(Infinitely Strong)				



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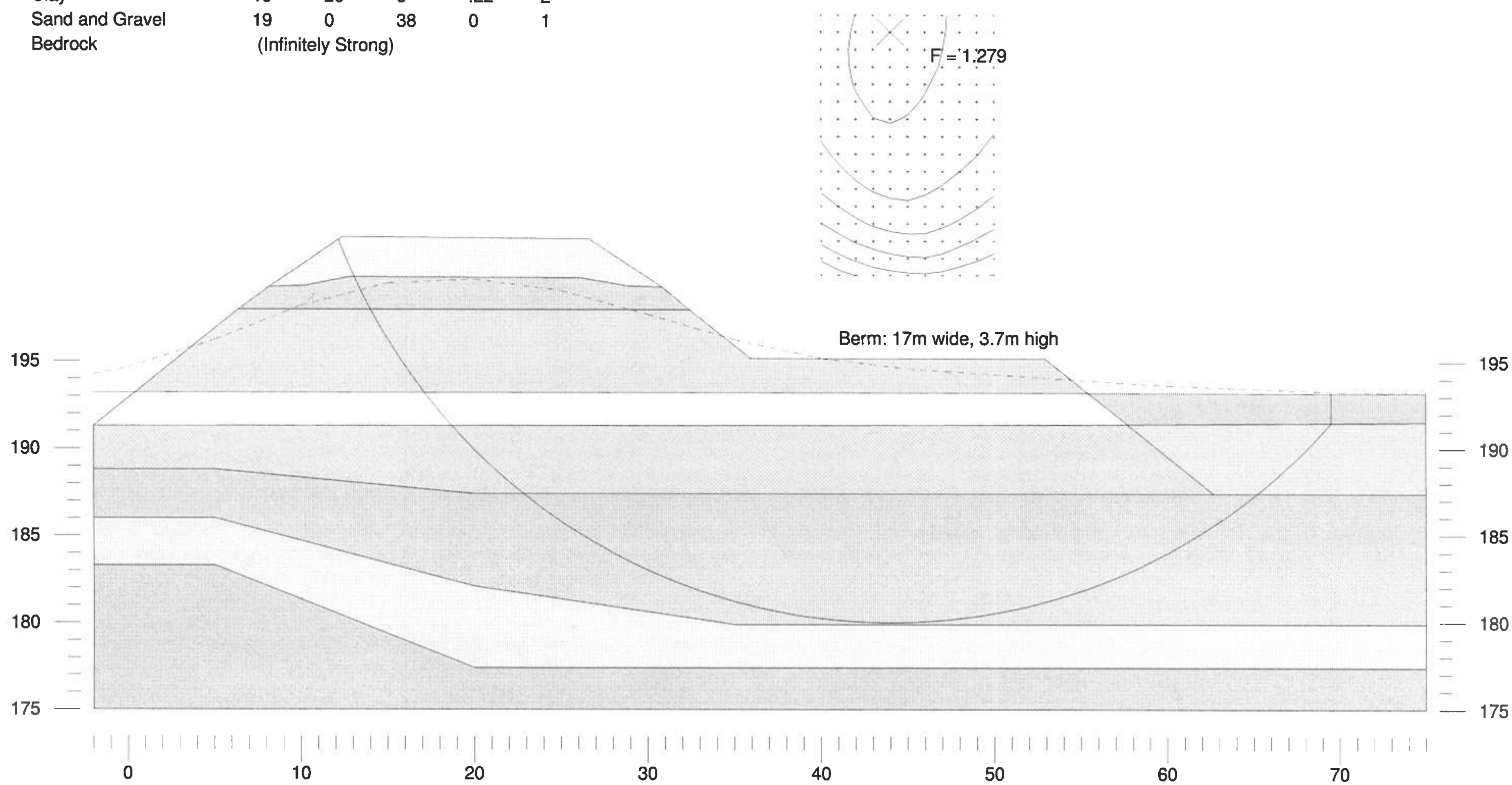
69HWY007

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Water	9.81	0	0	0	1
Surcharge (2)	21	0	32	0	1
Rockfill (2)	19	0	42	0	1
Rockfill	19	0	42	0	1
Rockfill (sub.)	19	0	42	0	1
Peat	13	10	0	0	1
Peat Replacement	19	0	42	0	1
Clay	19	20	0	.22	2
Sand and Gravel	19	0	38	0	1
Bedrock	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-5161-21
Highway 69 Four Laning
January 2011
Undrained Strength Analysis (Bbar2 = 0.9, Bbar1= 0.1)
2nd stage of construction (202.0m)



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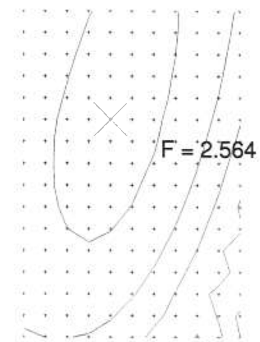
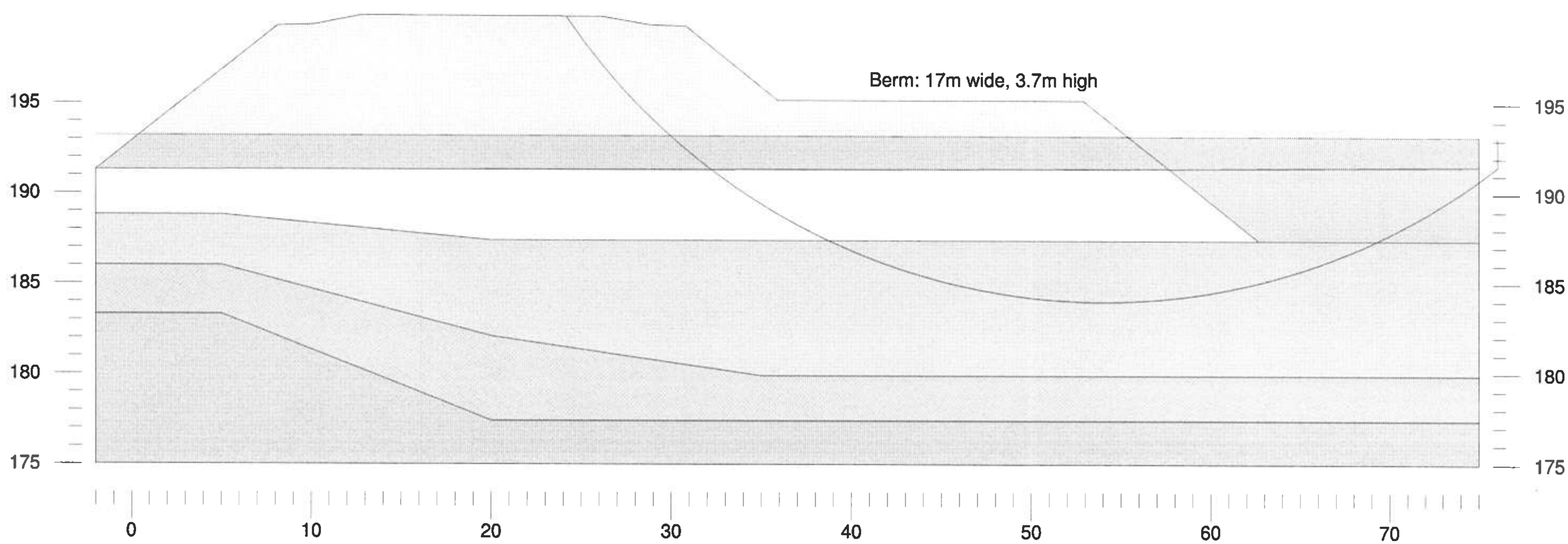
69HWY008

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Water	9.81	0	0	0	1
Rockfill	19	0	42	0	1
Rockfill (sub.)	19	0	42	0	1
Peat	13	2	29	0	1
Peat Replacement	19	0	42	0	1
Clay	19	0	28	0	1
Sand and Gravel	19	0	38	0	1
Bedrock	(Infinitely Strong)				



Thurber Engineering Ltd. - Toronto
19-5161-21
Highway 69 Four Laning
January 2011
Effective Stress Analysis (Bbar = 0.0)
Long Term (199.8m)



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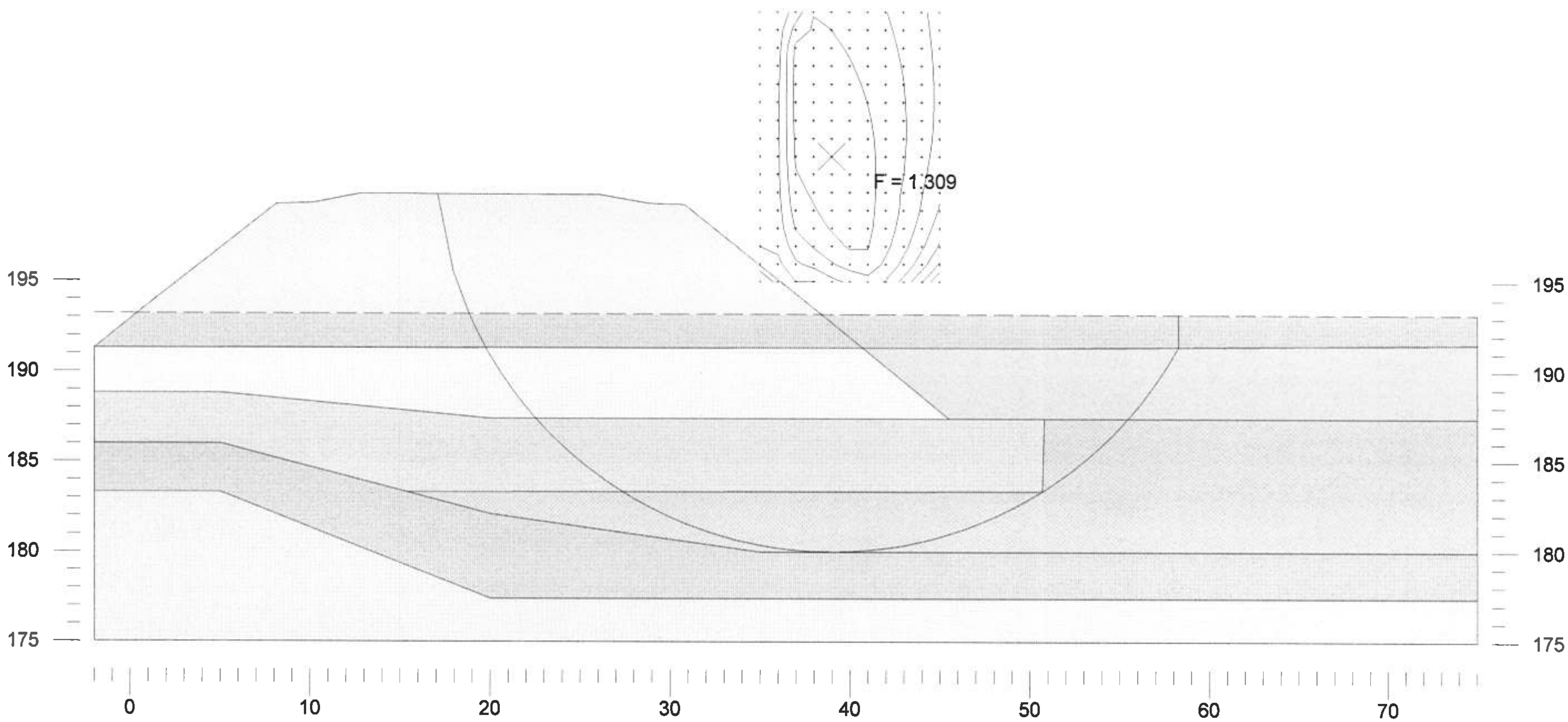
69HWY009

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Water	9.81	0	0	0	1
Rockfill	19	0	42	0	1
Rockfill (sub.)	19	0	42	0	1
Peat	13	10	0	0	1
Peat Replacement	19	0	42	0	1
Clay Replacement	19	0	42	0	1
Clay	19	20	0	0	1
Sand and Gravel	19	0	38	0	1
Bedrock	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-5161-21
Highway 69 Four Laning
January 2011
Total Stress Analysis (Bbar = 0.0)
One stage construction (199.8m)



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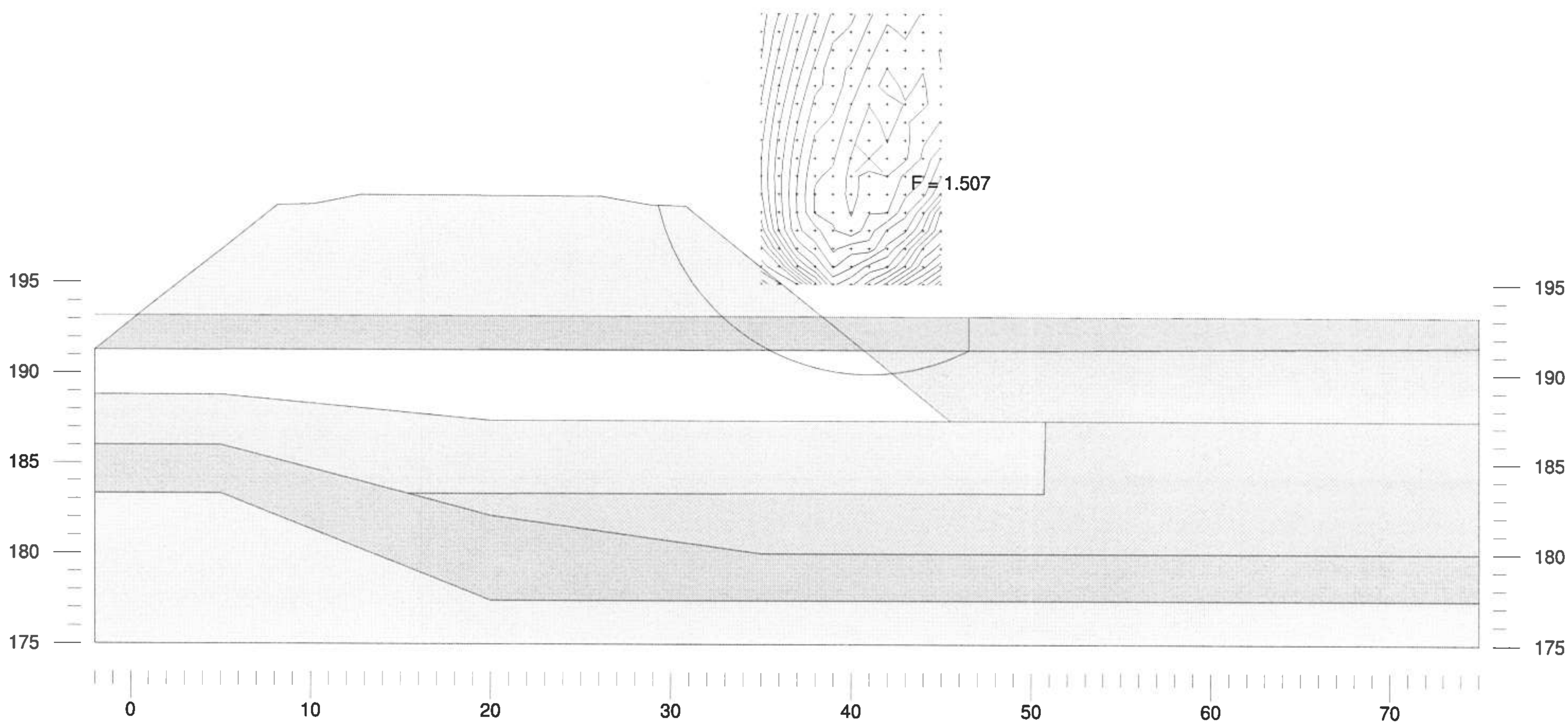
69HWY010

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Water	9.81	0	0	0	1
Rockfill	19	0	42	0	1
Rockfill (sub.)	19	0	42	0	1
Peat	13	2	29	0	1
Peat Replacement	19	0	42	0	1
Clay Replacement	19	0	42	0	1
Clay	19	0	28	0	1
Sand and Gravel	19	0	38	0	1
Bedrock	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-5161-21
Highway 69 Four Laning
January 2011
Effective Stress Analysis (Bbar = 0.0)
Long term (199.8m)



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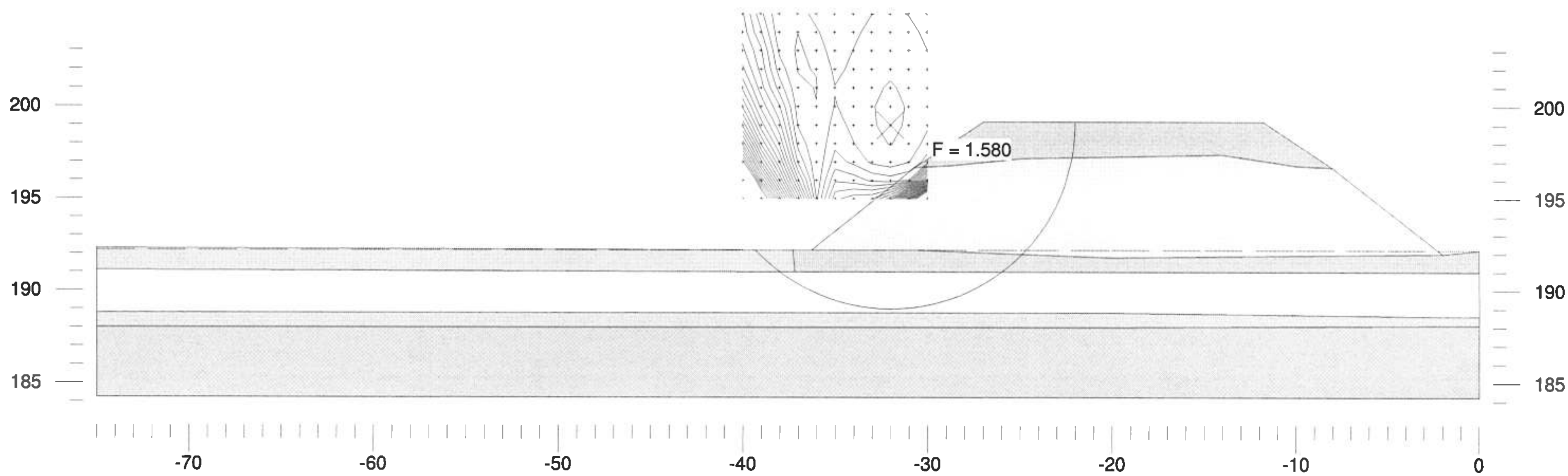
69HWY011

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Surcharge	21	0	32	0	1
Rockfill	19	0	42	0	1
Peat Replacement	19	0	42	0	1
Peat	13	10	0	0	1
Clay	18	35	0	0	1
Sand	19	0	38	0	1
Bedrock	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-5161-21
Highway 69 Four Laning
January 2011
Total Stress Analysis (Bbar = 0.0)
One stage construction (199.2m)



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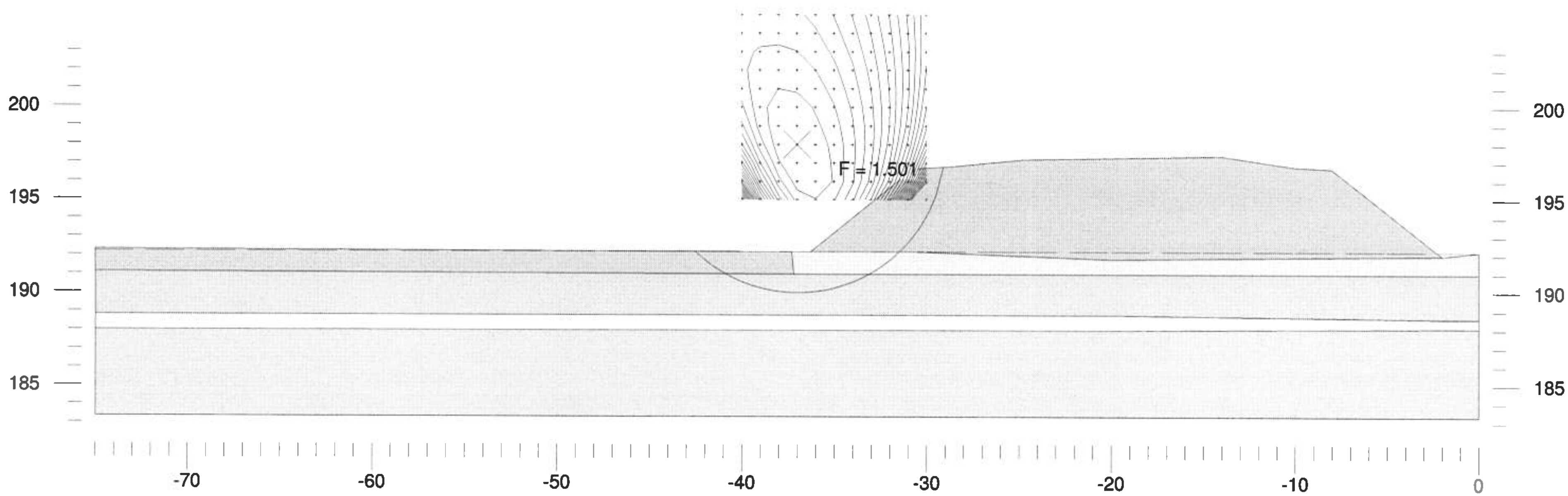
69HWY012

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Rockfill	19	0	42	0	1
Peat Replacement	19	0	42	0	1
Peat	13	2	29	0	1
Clay	18	0	28	0	1
Sand	19	0	38	0	1
Bedrock	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-5161-21
Highway 69 Four Laning
January 2011
Effective Stress Analysis (Bbar = 0.0)
Long term (197.2m)



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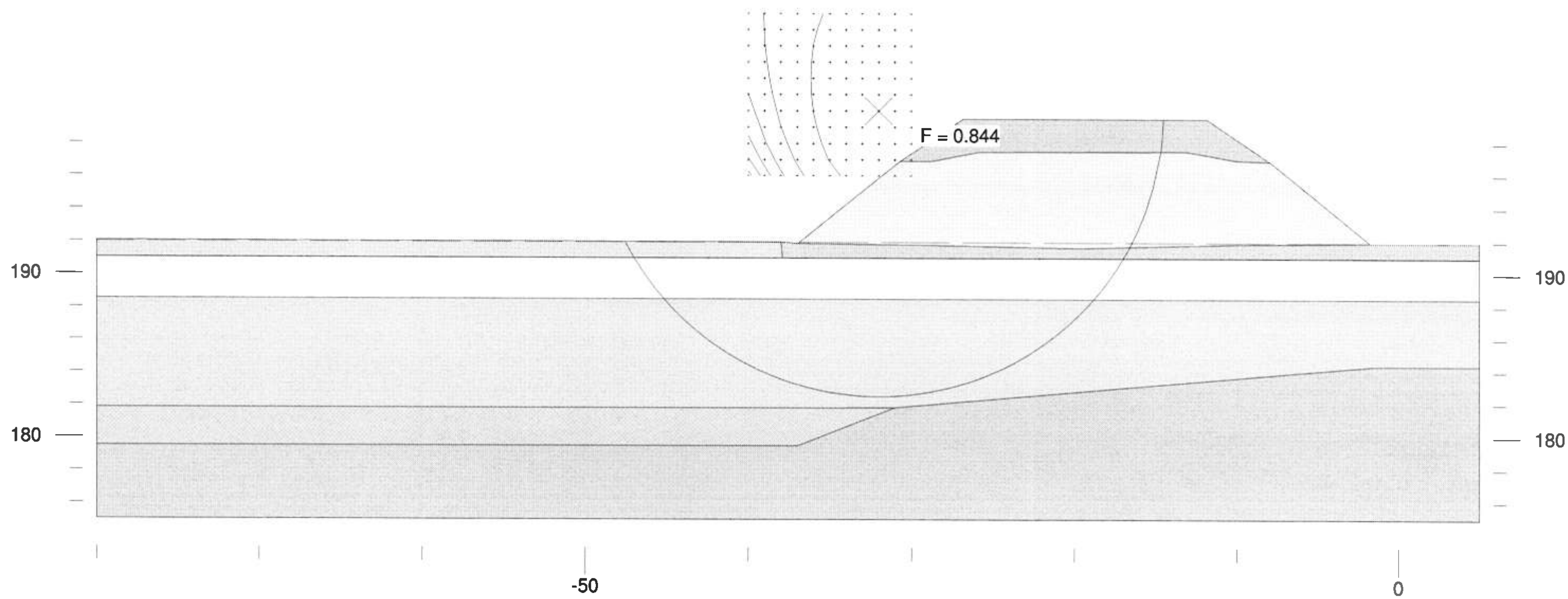
69HWY013

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Surcharge	21	0	32	0	1
Rockfill	19	0	42	0	1
Peat replacement	19	0	42	0	1
Peat	13	10	0	0	1
Clay Upper	18	35	0	0	1
Clay Lower	19	20	0	0	1
Sand and Gravel	21	0	32	0	1
Bedrock	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-5161-21
Highway 69 Four Laning
January 2011
Total Stress Analysis (Bbar = 0.0)
One stage construction (199.5m)



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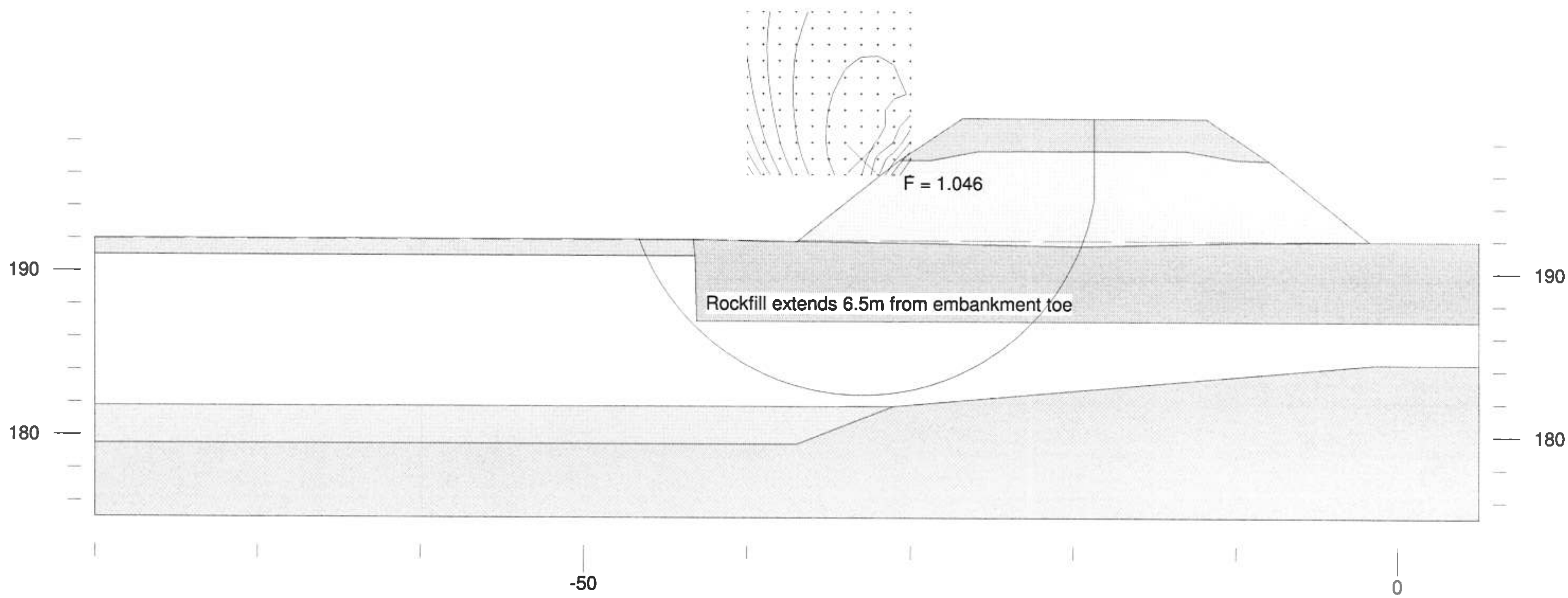
69HWY014

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Surcharge	21	0	32	0	1
Rockfill	19	0	42	0	1
Peat replacement	19	0	42	0	1
Peat	13	10	0	0	1
Clay	19	20	0	0	1
Sand and Gravel	21	0	32	0	1
Bedrock	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-5161-21
Highway 69 Four Laning
January 2011
Total Stress Analysis (Bbar = 0.0)
One stage construction (199.5m)



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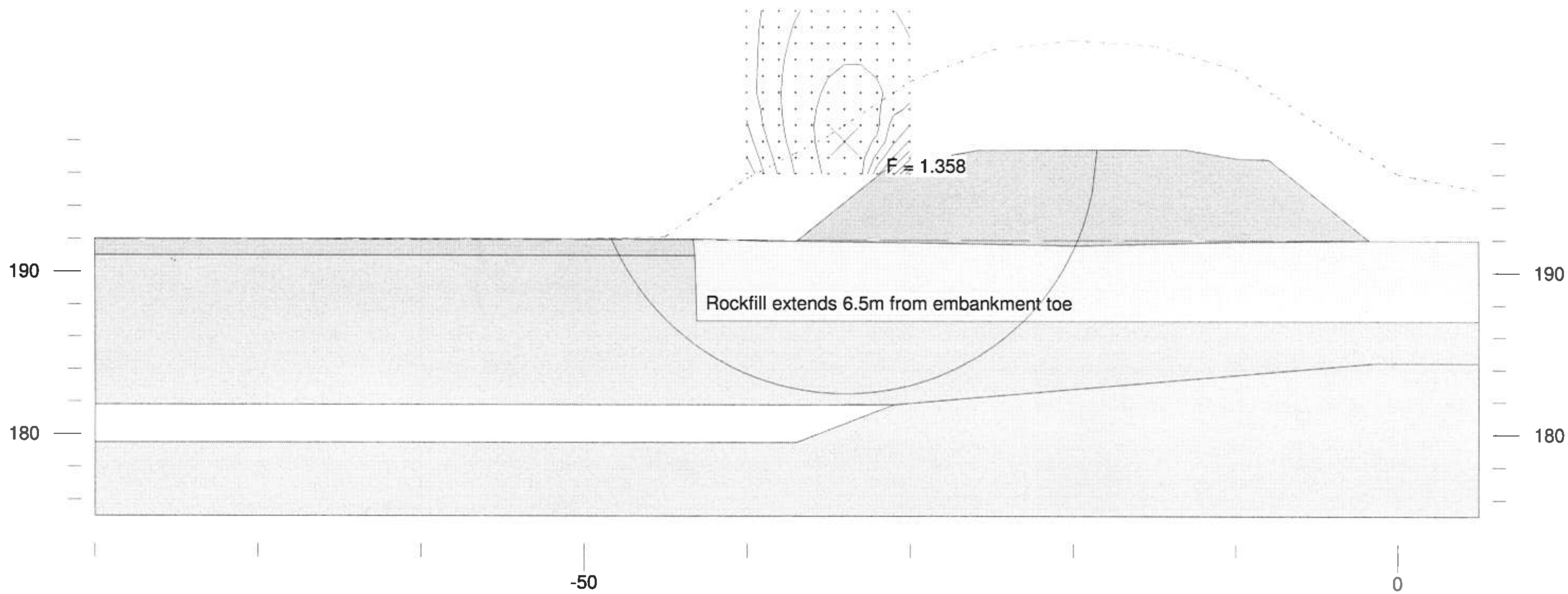
69HWY015

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Rockfill	19	0	42	0	1
Peat replacement	19	0	42	0	1
Peat	13	10	0	0	1
Clay	19	20	0	.22	2
Sand and Gravel	21	0	32	0	1
Bedrock	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-5161-21
Highway 69 Four Laning
January 2011
Undrained Strength Analysis (Bbar = 0.9)
1st stage of construction (197.0m)



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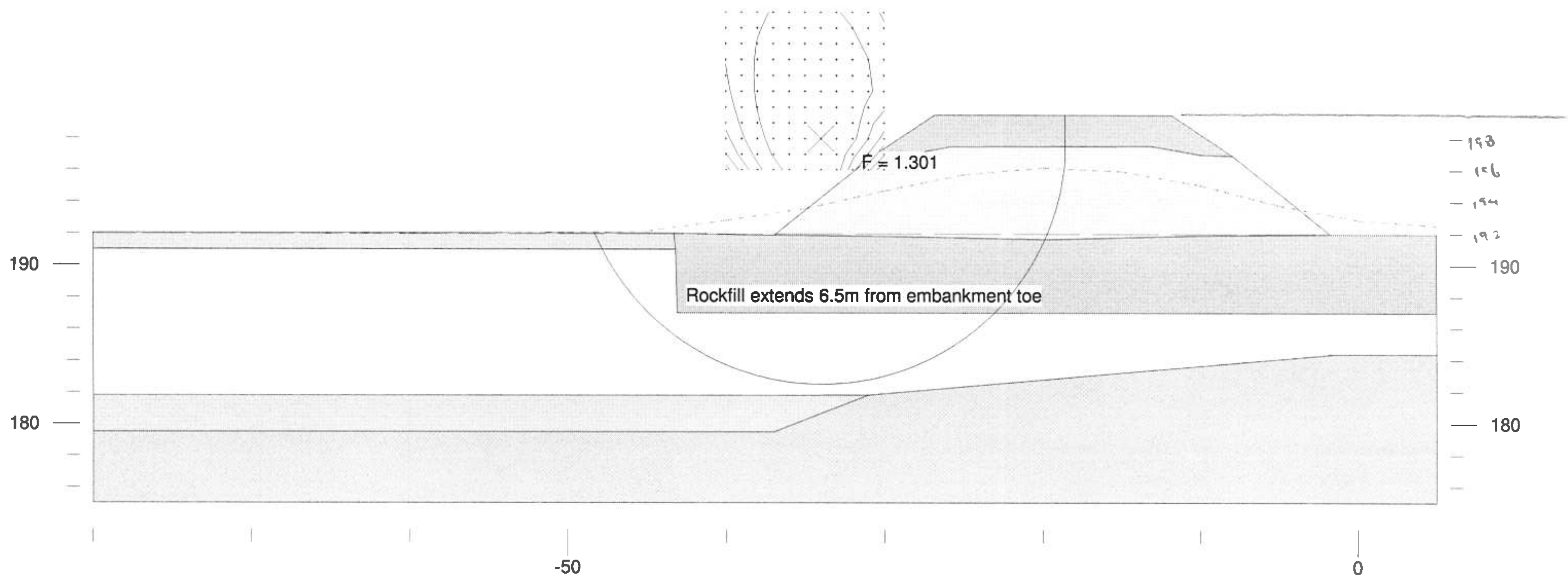
69HWY016

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Surcharge (2)	21	0	32	0	1
Rockfill	19	0	42	0	1
Peat replacement	19	0	42	0	1
Peat	13	10	0	0	1
Clay	19	20	0	.22	2
Sand and Gravel	21	0	32	0	1
Bedrock	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-5161-21
Highway 69 Four Laning
January 2011
Undrained Strength Analysis (Bbar2 = 0.9, Bbar1 = 0.1)
2nd stage of construction (199.5m)



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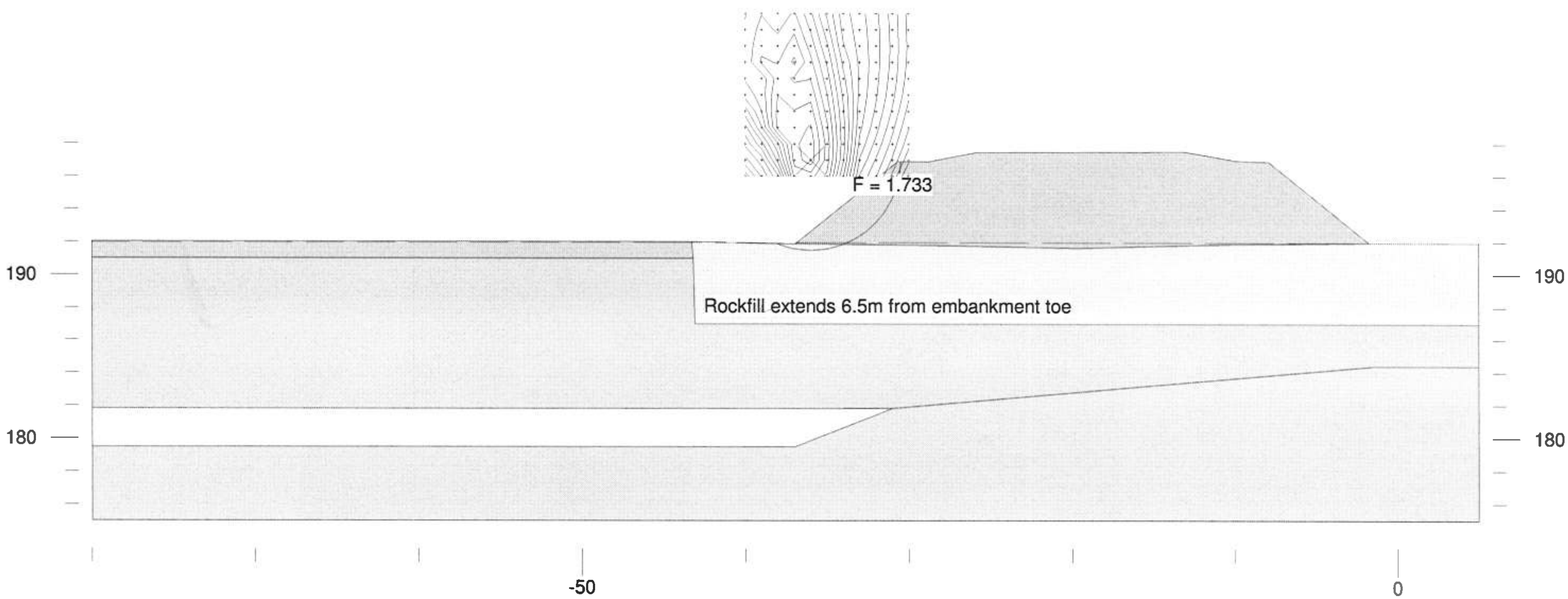
69HWY017

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.
Rockfill	19	0	42	0	1
Peat replacement	19	0	42	0	1
Peat	13	2	29	0	1
Clay	19	0	28	0	1
Sand and Gravel	21	0	32	0	1
Bedrock	(Infinitely Strong)				

Thurber Engineering Ltd. - Toronto
19-5161-21
Highway 69 Four Laning
January 2011
Effective Stress Analysis (Bbar = 0.0)
Long term (197.5m)



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69HWY018