PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
HIGHWAY 112 & ONR OVERHEAD
APPROXIMATELY 4.0 km SOUTH OF HIGHWAY 66
G.W.P. 140-88-00

Geocres Number: 42A-62

Report to

Ministry of Transportation Ontario
Northeastern Region, Engineering Office

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from a preliminary foundation investigation designed to explore the feasibility, from a geotechnical perspective, of construction a new overhead structure across the ONR track. The project is part of the overall feasibility study for the re-alignment of the highway to the east of the present alignment and the design of a new overhead structure at the ONR track.

The purpose of the investigation was to explore the subsurface conditions at the proposed bridge site and, based on the data obtained, to provide a borehole location plan, borehole logs, stratigraphic profile and cross-sections and a written description of the subsurface conditions. A preliminary model of the subsurface conditions has been developed along the proposed re-alignment. This model describes the geotechnical conditions influencing design and construction of the foundations for the bridge and of the immediate approach fills.

Thurber carried out the investigation for the Ministry of Transportation Ontario (MTO) under Agreement Number 5004-E-0054.

2 SITE DESCRIPTION

The site lies in the Geographic Townships of Otto and Teck approximately 4 km south of Hwy 66 at Kirkland Lake, Ontario. The site incorporates an area under consideration for construction of a new overhead crossing at the ONR track as part of a possible realignment of Hwy 112.

The general site area is located within the physiographic region known as the Canadian Shield, characterized by Pre-Cambrian bedrock typically occurring as rounded knobs and ridges where exposed. Locally, however, the bedrock is mantled by a deposit of silty clay overlain by silty sand.

There is intermittent residential and commercial development along existing Hwy 112 to the west and south of the site but the site itself lies in a wooded area. Approximately 200 to 300 m north of the site, the highway crosses a wetland for a distance of 500 m. Photograph F1 in Appendix F shows the general site area looking from the existing bridge and Photographs F2 and F3, respectively, show the areas to the north and south of the track.

3 SITE INVESTIGATION AND FIELD TESTING

Thurber carried out site investigation and field testing for this project between June 16 and June 21, 2005, inclusive

The site investigation consisted of drilling and sampling two boreholes to depths of 6.4 and 6.5 m at the approximate future abutment locations. The approximate locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix G.

The boreholes were advanced through the overburden using hollow stem augering techniques and samples were collected at regular intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). The undrained shear strength of the soil was assessed by means of vane shear testing at the north abutment.

The coordinates and elevations of the boreholes are given on the Borehole Locations and Soil Strata Drawing and on the individual Record of Borehole Sheets in Appendix A.

A standpipe piezometer, consisting of 19 mm PVC pipe with slotted tips, was installed at the north abutment to monitor the groundwater level.

The completion details for the piezometer are shown in Table 3.1.

Table 3.1 – Piezometer Details

Piezometer Location	Piezometer Details	
	Tip Depth/ Elevation	Completion Details
BH 05-9	6.4/302.9	Piezometer with 1.5 m tip installed at 6.4 m. Sand filter to 4.3 m, bentonite seal to the surface.

A member of Thurber's engineering staff supervised the drilling and sampling operations on a full time basis. The inspector logged the boreholes and the recovered samples and processed them for transport to Thurber's Oakville office.

4 LABORATORY TESTING

All recovered soil samples were subjected to visual identification and to natural moisture content determination. The results of this testing are shown on the Record of Borehole sheets in Appendix A.

Selected samples were subjected to gradation analysis (sieve and hydrometer) and Atterberg limit testing and the results are shown on the Record of Borehole sheets in Appendix A and on the charts in Appendix B. A total of three samples were selected for this testing.

5 DESCRIPTION OF SUBSURFACE CONDITIONS

5.1 General

Reference is made to the Record of Borehole sheets in Appendix A. Details of the encountered soil stratigraphy are presented in these appendices and on the attached Borehole Locations and Soil Strata Drawing. An overall description of the stratigraphy is given in the following paragraphs however the factual data presented in the borehole logs governs any interpretation of the site conditions.

The soil stratigraphy consisted generally of shallow topsoil and silty sand overlying silty clay. The clay, in turn was underlain by a thin layer of sand followed by bedrock.

5.2 Topsoil

Topsoil was encountered at the ground surface at the north abutment. The thickness was measured to be 300 mm. The topsoil was dark brown, sandy and contained rootlets.

5.3 Silty Sand

A glaciofluvial outwash deposit of fine-grained, silty sand was encountered below the topsoil at the north abutment.

The deposit consists of silty sand, some clay and trace of gravel. The layer of silty sand was 0.6 m thick and the base of the layer lay at Elevation 308.3. Based on SPT values ranging of 5 to 6 blows for 0.3 m of penetration, the deposit is classified as loose.

The natural moisture content of this deposit is estimated to be in the order of 30%.

5.4 Silty Clay

A layer of silty clay was encountered underlying both abutment areas. The maximum recorded SPT value in this deposit was 10 blows for 0.3 m of penetration, indicating stiff conditions. However, at a depth of 4 m under the north abutment, the sampling spoon sank under the weight of the drill rods, indicating very soft conditions. The silty clay is, therefore, classified as very soft to stiff. A vane shear strength measured at the north abutment was 25 kPa, indicating soft to firm conditions. The sensitivity of the clay was 2.

The thickness of silty clay ranged from 3.4 m to 5.2 m at the south and north abutments respectively. The underside of the clay layer lay at Elevation 306.0 at the south abutment and 303.2 at the north abutment.

Hydrometer test carried out on selected samples of the silty clay are illustrated in Figure B1 in Appendix B. The test results show that the material consists of 97% to 100% fines, with clay content of approximately 50%.

Atterberg Limit tests, summarized in Figure B2 in Appendix B, indicates that the silty clay is classified as CI.

The water content ranged from 30 to 54%. The soil samples with higher water content had Liquidity Index values ranging from 1.0 to 1.8 whereas the samples with lower water content had Liquidity Index values ranging from 0.6 to 1.0.

5.5 Sand

At both abutments, the silty clay is underlain a deposit of sand, some gravel, trace silt. The presence of cobbles and boulders was inferred from auger grinding and is typical of such a deposit. The thickness of the deposit ranged from 0.3 to 0.5 m and was identified from auger samples. No SPT values were obtained. The underside of the sand layer was at 305.5 at the south abutment and at 302.9 at the north abutment.

The natural moisture contents were not determined but the soil is expected to be saturated, due to its location under the saturated clay layer.

5.6 Bedrock

Bedrock of the Canadian Shield, consisting of gneiss, meta-volcanic with calcite veins, was encountered at the site. At the south abutment, bedrock was proved at a depth of 3.9m (Elevation 305.5). At the north abutment, bedrock is assumed to lie at a depth of 6.4 m (Elevation 302.9), based on auger refusal. The bedrock was dark grey, fresh, very hard with RQD values of zero to 4.4m depth and 48% to 55% to the end of borehole at 6.5m depth, EL.302.9. The core was observed to be heavily fractured at approximately 5.8 to 6.0m depth. Unconfined compressive strengths inferred from Point Load Tests ranged from 114 to 232MPa, with an average value of 185MPa.

5.7 Water Levels

The recorded groundwater depths and elevations are shown in Table 5.2.

Table 5.2 – Groundwater Depths (in metres) and Elevations

Date	West Abutment	
	Depth	Elev.
Jun 20/05	2.1	307.2
Jun 21/05	1.6	307.7

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

6 MISCELLANEOUS

Field staking carried out by Ministry forces was used for preliminary layout of the boreholes. Surveying of the actual locations of the boreholes was carried out by Sutcliffe Rody Quesnel Inc. from North Bay, Ontario.

The drill rig and sampling equipment used in the investigation were supplied and operated by Colbar Resources, of Lively, Ontario.

Full time supervision of field activities, including obtaining utility clearances was carried out by Mr. Mark Farrant, B.Sc. and Mr. George Azzopardi of Thurber.

Overall supervision of the field program, interpretation of the data and preparation of the report were carried out by Mr. Alastair E. Gorman, P.Eng.

The report was reviewed by Dr. Paulo J. Branco, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

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PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

7 INTRODUCTION

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical design recommendations to assist the design team to select and design a suitable foundation system and approach fills for the proposed structure.

The general area around the proposed bridge site is relatively flat, see Photographs in Appendix F. The finished grade at the south abutment will lie at approximately Elevation 319.0 and the original ground surface is at Elevation 309.4, resulting in a 9.6 m high embankment.

The finished grade at the north abutment will lie at approximately Elevation 319 and the original ground surface at this location is at Elevation 309.3, giving a total embankment height of 9.7 m.

It is understood that a single-span structure would normally be considered for this overhead structure. However, the soils encountered at this site present problems with respect to embankment construction and stability. These issues are discussed briefly in this report, but a complete discussion and recommendations are contained in a separate report by Thurber and reference should be made to that report. The embankment stability and staging discussed in that report may require the construction of a two-span structure with one span to accommodate the ONR track and the other to accommodate a berm in front of the forward slope.

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

8 STRUCTURE FOUNDATIONS

Foundation alternatives are presented in the following sections together with the corresponding geotechnical design parameters.

Based on the results of the exploratory boreholes drilled at the proposed abutment locations, the stratigraphy consists of approximately 3 to 5 m of generally soft to stiff clay overlying a thin layer of sand overlying bedrock. At the north abutment, the clay is overlain by 0.9 m of silty sand and topsoil.

Initial consideration was given to the following foundation types:

- Spread footings on native soil
- Spread footings on engineered fill
- Spread footings on bedrock
- Driven steel H-piles
- Caissons (drilled shaft piles)

The technical advantages and disadvantages of the different foundation schemes are discussed below and are summarized in Appendix D.

8.1 Spread Footings

8.1.1 Footings on Native Soil

The existing native soils lying near the surface are considered unsuitable for the support of spread footings. The geotechnical resistance provided by the clay soil is less than 100 kPa and the footings would be subject to settlements in the order of 300 to 400 mm..

Accordingly spread footings founded on native soil are not recommended and were eliminated from further consideration.

8.1.2 Footings on Engineered Fill

Engineered fill pads could be designed to provide adequate bearing resistance for perched abutment foundations. However, the foundation and engineered fill pad would probably experience unacceptable settlement due to consolidation of the underlying clay.

This system is not considered to be feasible for piers since it would not be practical to develop sufficient thickness of engineered fill and the settlement concerns would apply to the pier foundations as with the abutments.

Accordingly spread footings founded on engineered fill pads are not recommended and were eliminated from further consideration.

8.1.3 Footings on Bedrock

The bedrock encountered below the site would provide good bearing resistance for spread footings. A spread footing bearing on the sound bedrock could be designed on the basis of a concentric, vertical geotechnical resistance of 10,000 kPa, factored ULS. The SLS condition will not govern for foundations bearing on bedrock.

The disadvantage of spread footings bearing on bedrock is the depth to bedrock at the bridge site. Based on the preliminary investigation, bedrock lies at a depth of 3.9 m below existing grade at the south abutment and at 6.4 m (based on auger refusal) at the north abutment.

Installation of spread footings to these depths would require extensive excavation, groundwater control and track protection. These factors are considered to make spread footings an impractical option at this site.

8.2 Driven Steel Piles

The geotechnical conditions encountered at this site are considered suitable for driven steel H-pile foundations.

The piles are expected to develop bearing resistance on the bedrock at approximate Elevation 305.5 at the south abutment and approximate Elevation 303.2 at the north abutment.

Driven steel piles may be designed on the basis of the axial geotechnical resistances given in Table 8.1.

Table 8.1 – Pile Geotechnical Resistance

Pile Section	ULS (Factored)
HP 310 X 110	2,000 kN
HP 310 X 132	2,400 kN
HP 310 X 152	2,750 kN
HP 360 X 132	2,400 kN

8.2.1 Pile Tips

To protect the piles and assist with seating in the bedrock, the tips of all piles should be fitted with cast steel, H-section rock points from an approved manufacturer such as Titus Steel (Standard H-point) or APF hard Bite or approved equivalent.

8.2.2 Pile Installation

Pile installation should be in accordance with Special Provision No. 903S01.

To facilitate pile installation, embankment fill through which piles will be driven must not contain oversize material, i.e. no particles exceeding 75 mm in size.

8.2.3 Pile Driving

The appropriate pile driving note is "Piles to be driven to bedrock.

8.3 Caissons

8.3.1 General

The bridge foundations could be supported on caissons (drilled shafts) bearing on the bedrock.

8.3.2 Geotechnical Resistance

For design, the caissons can be treated in two ways, as follows:

1. The caisson can be treated as a short pier bearing on the bedrock with a factored ULS geotechnical resistance of 10,000 kPa. For foundations bearing on bedrock, the SLS condition will not govern.

   In this case, it will be necessary, during construction, for the base of the caisson to be clear of soil and broken rock prior to concreting. It will be necessary to inspect the base to verify that it is in sound rock and has been cleaned.

2. The caisson can be advanced into bedrock and designed on the basis of geotechnical resistance developed in skin friction in the rock socket.

   In this case, the geotechnical resistance can be calculated on the basis of the shaft area in contact with bedrock and an assumed adhesion:

   $q_a = 0.05f_c$ where f_c is the compressive strength of the concrete in the caisson

8.3.3 Caisson Installation

The caissons will be installed through soft soils and it will be necessary to use a temporary liner to retain the wall of the caisson until concrete has been poured.

In the case of an end bearing design, the liner must also prevent the sand layer immediately overlying the bedrock from washing into the base of the caisson with groundwater flow. The contractor must demonstrate that he can keep the base of the caisson free of soil and permit inspection prior to concreting.

In the case of shaft friction design, the base of the caisson does not need to be cleaned.

For either design method, if the caisson cannot be effectively dewatered, provisions must be made to place the concrete by tremie methods.

8.4 Downdrag

Long-term secondary consolidation of the clay due to embankment construction will create negative skin friction on piles or caissons.

However, due to the comparatively small thickness of the clay, these forces are not expected to be significant. This issue must be checked during the detail design stage.

8.5 Lateral Resistance on Piles

For preliminary design purposes, the lateral geotechnical resistance acting on a driven pile may be assumed to be due to the layer of silty clay. Caissons (or steel piles) grouted into rock sockets may derive lateral resistance from both the soil and the bedrock.

The equations and recommended parameters provided below may be used to analyze the interaction between a pile and the soil or rock. The lateral pressures obtained from the analysis should not exceed the ultimate lateral resistance.

The spring constant, K, for analysis may be obtained by the expression, $K = k_s*L*D$ (kN/m), where k_s is the coefficient of horizontal subgrade reaction (kN/m^3), D is the pile width (m) and L is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance on any one segment of pile, P_{ult}, may be obtained from the expression, $P_{ult} = p_{ult}*L*D$. This represents the ultimate load at which the pile fails and will not support any additional load at greater displacements. It is recommended, however, that the total lateral resistance assumed in one pile be limited to no more than 150 kN at ULS and 50 kN at SLS.

8.5.1 Soil Resistance

The lateral geotechnical resistance on the piles due to interaction with the silty clay may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and ultimate lateral resistance (p_{ult}) as follows:

$k_s = 67S_u / D$ (kN/m^3)

$p_{ult} = 9\ S_u$ (kPa)

where D = pile width (m)

S_u = shear strength (kPa)

Table 8.2 – Shear Strength Parameters

Location	Elevation	S_u
South Abutment	OGL to 306.0	50 kPa
North Abutment	OGL to 307.0	50 kPa
	307.0 to 303.2	25 kPa

OGL = Original ground level.

8.5.2 Rock Resistance

For piles socketed into bedrock, typical values for lateral resistance are provided based on an assumed caisson diameter of 0.6 m. The values that may be used in analysis are shown in Table 8.3.

Table 8.3 – Lateral Resistance Factors for Bedrock (0.6 m Shaft)

Depth Below Rock Surface (m)	k_s (kN/ m^3)	p_{ult} (kPa)
0.0	480,000	130
0.3	800,000	225
0.6	1,100,000	320
0.9	1,440,000	410
1.2	1,760,000	500
1.5	2,100,000	600

8.5.3 Reduction Factors for Lateral Resistance

The modulus of subgrade reaction may have to be reduced, based on the pile spacing.

The following reduction factors should be used for a pile group oriented ***perpendicular*** to the direction of loading.

Pile spacing	Reduction Factor
4D	1.00
1D	0.5

The following reduction factors should be used for a pile group oriented ***parallel*** to the direction of loading.

Pile spacing	Reduction Factor
8D	1.00
6D	0.7
4D	0.4
3D	0.25

--- where "D" is the breadth of the pile, spacing is centre to centre

Intermediate values may be obtained by linear interpolation.

In the case on conventional abutments, i.e. not integral, horizontal loads may be resisted by means of battered piles.

8.6 Abutment Type

The design of an integral abutment generally requires a minimum pile length of 5 m in order to allow for the required 3 m free length and a minimum embedment of 2 m. The practicality of integral abutments at this site will depend on the general arrangement of the structure.

The preliminary investigation has shown that there is sufficient depth of soil overlying the bedrock (6.1 m) at the north abutment for the design of an integral abutment to be feasible.

At the south abutment, the depth to bedrock is 3.9 m, which is insufficient for integral abutment design. However, if the structure is designed with perched abutments, i.e. the piles will be driven through the approach fill, an integral abutment design may be feasible.

A false integral abutment may also be considered if it contributes to the overall efficiency of the design. In this design, the structure loads are carried on driven piles and the approach fill is retained by a RSS wall constructed immediately in front of the piles. The feasibility of a false integral abutment will be limited by considerations of global stability and settlement and these must be fully addressed during the detail design stage after the general arrangement has been selected and the construction sequence for surcharging has been finalized.

The integral abutment design requires that the piles posses flexibility in the upper 3 m of the pile length. At this site, the upper 3 m of the pile length will lie in very loose sandy silt that, in its original state, would provide sufficient flexibility. However, if the upper 3 m of the piles lies in compacted fill or if the native soil became compacted by the construction processes, the required flexibility may be compromised. Accordingly, to provide the required flexibility in the piles, the upper 3 m of the piles should be surrounded by one of the following systems:

- For a “true abutment” supported on top of the piles - a 600 mm diameter CSP filled with sand, or
- For “false abutment” - concentric CSPs in accordance with standard integral abutment design procedures.

The sand must be placed in the CSP after the pile has been driven to avoid the danger of the sand being densified by pile driving and the possibility of the CSP being dragged down by the pile.

Backfill sand should meet the gradation shown in Table 8.6.

Table 8.6 – Integral Abutment Sand Grading

MTO Sieve Designation		Percentage Passing
2 mm	#10	100%
600 μm	#30	80%-100%
425 μm	#40	40%-80%
250 μm	#60	5%-25%
150 μm	#100	0%-6%

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If the earth pressures acting on an integral abutment are to be modelled using springs, the following values of the modulus of horizontal subgrade reaction may be used:

Granular "B" Type I	k(s) = 4,500*z/h kN/m3
Granular "A"	k(s) = 5,600*z/h kN/m3

z = depth from top of abutment wall to point of interest (metres)

h = full height of the abutment wall (metres)

The upper limit of force on the abutment calculated in the analysis is the total passive force that can be mobilized in the backfill, calculated as described elsewhere in this report.

The design of semi-integral and conventional abutments is also feasible at this site.

8.7 Frost Protection

The depth of earth cover required to provide frost protection for footings and pile caps at this site is 2.4 m.

Frost protection is not an issue for spread footings bearing on sound bedrock at this site.

9 EXCAVATION

All excavation must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the soils within the probable depth of excavation at this site may be classed as Type 3 soils above the water table. Excavation slopes should not exceed 1V:1H above the groundwater level.

Excavation requirements will not be known until the general arrangement has been established. If excavation is carried out in proximity to the ONR track, it is expected that track protection will be required.

10 UNWATERING

If excavation below the groundwater level is required, the excavation must be unwatered.

The design of the dewatering system that may be required should be the responsibility of the Contractor and the Contract Documents should alert him to this responsibility and the need to engage a dewatering specialist. While the responsibility for dewatering should remain with the Contractor, suitable systems that might be employed include pumping from filtered sumps for penetration of no more than 0.5 m below the groundwater level and the use of vacuum wellpoints for deeper penetration below the groundwater level.

11 APPROACH EMBANKMENTS

The design of the immediate approaches, within 20 m of the structure, must be coordinated with the design of the mainline embankment. Reference must be made to the separate report by Thurber Engineering Ltd. that presents geotechnical analysis and recommendations for embankment design and for construction staging. This report is titled Draft Preliminary Foundation Investigation and Design Report, Embankments for Highway 112 Realignment and ONR Overhead Structure, Approximately 4 km South of Highway 66 (Embankment Report).

Separate analyses were carried out for the south and the north abutments on account of the different soil conditions encountered. It is important that a more extensive investigation be carried out during the detail design phase in order to better delineate the soils at the site and to permit a re-evaluation of the stability of the embankment.

Selected output from the slope stability calculations is shown in Appendix F.

11.1 South Abutment

The one borehole drilled at the south abutment indicated that there is approximately 3.4 m of firm to stiff, brown clay overlying 0.5 m of sand and bedrock.

Stability analysis was carried out assuming that a rock fill embankment will be constructed and that it will be surcharged using 2 m of Granular "B" Type II.

For the purpose for preliminary analysis, it has been assumed that the surcharge will continue forward from the abutment location. Depending on the span arrangement selected for the bridge, this may encroach on the ONR ROW. It may be necessary to obtain agreement from ONR for the encroachment. The encroachment can be reduced and possibly eliminated by constructing a toe wall or a mechanically stabilized earth (MSE) mass at the front of the fill.

Stability analysis showed that the south approach can be constructed in one stage and that it will have a Factor of Safety 1.4 against instability at the end of construction. After surcharge removal, the long term factor of safety will be 1.5.

For embankments on cohesive foundation soils, a long term Factor of Safety of at least 1.5 is recommended to mitigate possible creep effects.

11.2 North Abutment

The one borehole drilled at the north abutment indicated that there is approximately 5 m of soft to firm clay overlying 0.3 m of sand and bedrock.

Stability analysis was carried out assuming that a rock fill embankment will be constructed and that it will be surcharged using 2 m of Granular "B" Type II.

The stability analysis showed that the approach fill must be constructed in two stages and that stability considerations require a 15 m wide by 4 m high berm to achieve a Factor of

Safety of 1.3 at the end of construction. The long term Factor of Safety, after removal of the surcharge, is 2.4.

As discussed in the section dealing with foundations, the requirement for this berm may necessitate the construction of a two-span bridge, or the use of surcharge followed by EPS fill behind the abutment.

11.3 Settlement

The settlement at the north approach under the full embankment loading is expected to be approximately 700 mm. At the south approach, where the clay is thinner and generally overconsolidated, the settlement is expected to be smaller, probably in the order of 200 mm.

It is recommended that the immediate approach embankments be constructed in conjunction with the mainline embankments as described in the Embankment Report.

11.4 General Embankment Requirements

All topsoil and organic soils should be stripped from the footprint of the immediate approach fills.

Embankment construction should be in accordance with OPSS 206, as amended by Special Provision "Amendment to OPSS 206, December 1993", dated November 2002

Where earth fill embankments are higher than 8 m or rock fill embankments are higher than 10 m, mid-height berms should be incorporated in the design. The berms should:

- extend for the length through which the embankment height exceeds 6 m
- be 2 m wide
- have 2% positive drainage to shed run-off water (earth fill embankments).

If a forward slope is incorporated in the design, the requirement for a berm may be evaluated separately from the side slope requirements.

Earth fill embankment slopes must be provided with erosion protection in accordance with OPSS 572.

12 BACKFILL TO ABUTMENTS

In the case of integral or semi-integral abutments, backfill to the abutment should be granular material.

In the case of a conventional abutment, granular backfill is recommended but rock backfill can be permitted. A NSSP is required to specify grading limits for the rock fill. The rock fill used as backfill to the abutment should be limited to fragments no greater than 75 mm.

In all cases where the approach embankment consists of rock fill and granular backfill to the abutment wall is used, the granular backfill must consist of OPSS Granular "B" Type II.

The backfill to the abutment walls must be in accordance with OPSS 902 as amended by Special Provision 902S01. Granular backfill must be placed to the extents shown in OPSD 3501.000, and rock backfill must be placed to the extents shown in OPSD 3505.000.

Compaction equipment to be used adjacent to retaining structures must be restricted in accordance with Special Provision 105S10.

The design of the abutment must incorporate a subdrain as shown in OPSD 3501.000 or OPSD 3505.000, as applicable.

13 EARTH PRESSURE COEFFICIENTS (ABUTMENTS)

Earth pressures acting on the structure may be assumed to be triangular and to be governed by the characteristics of the abutment backfill. For a fully drained condition, the pressures should be computed in accordance with the CHBDC but generally are given by the expression:

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

Where:

P_h = horizontal pressure on the wall at depth h (kPa)

K = earth pressure coefficient (see below)

γ = unit weight of retained soil (see table below)

h = depth below top of fill where pressure is computed (m)

q = value of any surcharge (kPa)

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

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are shown in Table 13.1.

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) might be preferred as it results in lower earth pressures acting on the wall. In the case of integral abutments, material with a lower passive pressure coefficient (e.g. Granular B Type I) might be preferred as it results in lower forces acting on the ballast wall as the wall moves toward the soil mass.

The factors in Table 13.1 above are "ultimate" values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the Canadian Highway Bridge Design Code.

Table 13.1 – Earth Pressure Coefficient (K)

Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35°, \gamma = 22.8\ kN/m^3$		OPSS Granular B Type I $\phi = 32°, \gamma = 21.2\ kN/m^3$		Rock Fill (Limited to 300 mm size) $\phi = 42°, \gamma = 19\ kN/m^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall(2H:1 V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall(2H:1 V)
Active (Unrestrained Wall)	0.27	0.40*	0.31	0.43*	0.2	.30*
At rest (Restrained Wall)	0.43	-	0.47	-	.33	-
Passive (Movement Towards Soil Mass)	3.7	-	3.3	-	5.0	-

* For wing walls.

14 SEISMIC CONSIDERATIONS

For design purposes, the site is treated as lying in Seismic Zone 1.

14.1 Seismic Design Parameters

The following seismic parameters should be used for design:

- Velocity Related Seismic Zone 1
- Zonal Velocity Ratio 0.05
- Acceleration Related Seismic Zone 1
- Zonal Acceleration Ratio 0.05
- Peak Horizontal Acceleration 0.08

The Soil Profile Type at this site has been classified as Type I. Thus, according to Table 4.4.6.1 of the CHBDC, a Site Coefficient "S" of 1.0 should be used in seismic design.

14.2 Liquefaction Potential

The potential for liquefaction of the foundation soils has been assessed using the Seed and Idriss (1971) method[1].

Using this method, it was determined that the foundation soils are not in danger of liquefaction.

14.3 Retaining Wall Dynamic Earth Pressures

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

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

ϕ = 35° for OPSS Granular A or Granular B Type II

ϕ = 32° for OPSS Granular B Type I

ϕ = 42° for rock fill

δ = 50% of ϕ

Where ϕ = the angle of internal friction of the backfill and δ = the angle of friction between the wall and the backfill.

The seismic earth pressure coefficients to be used in design at this site are shown in Table 14.1 at the end of the text.

15 CONSTRUCTION CONCERNS

Based on the limited geotechnical information obtained in the course of the preliminary investigation, construction concerns may include:

- The variability of pile lengths if driven piles are used
- The impact of construction on the ONR track, particularly the risk of track settlement induced by pile driving, embankment loading or by improperly supported excavation.

[1] Seed, H.B. and Idriss, I.M. 1971, "Simplified Procedure for Evaluating Soil Liquefaction Potential" *Journal of Soil Mechanics and Foundations Division,* ASCE, Vol. 101, No. SM9, September, pp. 1249 – 1273.

16 SCOPE OF FUTURE INVESTIGATION

More detailed investigation and analysis of the soil stratigraphy and rock profile under the site is required during the detail design phase of the project.

As a minimum, the investigation and analysis must include, though not necessarily be limited to:

1. Drilling and sampling additional boreholes at each foundation element in accordance with MTO requirements for foundation investigations:
   - At each deep foundation, at least two boreholes to bedrock, with one cored for 3 m into rock
   - At each shallow foundation, at least five boreholes to bedrock with three cored 3 m into bedrock
2. Drilling and sampling one borehole in each approach fill.
3. Where abrupt changes in soil stratigraphy or bedrock profile are encountered, additional boreholes or probes should be advanced to supplement the basic borehole pattern.
4. For the north approach in particular, cone penetration testing with pore pressure dissipation (CPTU) should be considered, especially if it can be carried out in conjunction with CPTU tests for the mainline embankment.
5. Laboratory testing should include, in addition to routine testing, consolidation tests and tri-axial shear tests.
6. The foundation design recommendations must be re-assessed when more complete geotechnical information is available, particularly the depth to bedrock at each foundation element.
7. Issues relating to approach embankment settlement and stability, including stability during construction, may prove critical to the overall design and construction and must be carefully addressed during the detail design phase.
8. The analysis and design recommendations for the immediate approach embankments within 20 m of the abutments must be coordinated with the analysis and design recommendations for the mainline embankments. The requirements for the following items must be coordinated:
   - Surcharging
   - Lateral and forward stability berms
   - Construction sequence and timing

17 CLOSURE

Engineering analysis and preparation of the report were carried out by Mr. Alastair E. Gorman, P.Eng.

The report was reviewed by Dr. Paulo J. Branco, P.Eng., a Designated Principal Contact for MTO Foundations Projects.

Thurber Engineering Ltd.

Alastair E. Gorman, P.Eng., M.Sc.
Senior Foundations Engineer

Paulo J. Branco, P.Eng., Ph.D.
Review Principal

Table 14.1

Earth pressure Coefficients for Seismic Design

Condition	Earth Pressure Coefficient (K) for Earthquake Loading					
	Granular A or Granular B Type II $\phi = 35°$, $\delta = 17°$		OPSS Granular B Type I $\phi = 32°$, $\delta = 16°$		Rock Fill $\phi = 42°$, $\delta = 21°$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (K_{AE})*	0.28	0.46	0.31	0.58	0.21	0.30
Passive (K_{PE})*	7.0	-	5.5	-	14.1	-
At Rest (K_{OE})**	0.53		0.58		0.44	

* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.
** After Woods

Appendix A

Record of Borehole Sheets

SYMBOLS, ABBREVIATIONS AND TERMS USED ON RECORDS OF BOREHOLES

1. TEXTURAL CLASSIFICATION OF SOILS

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

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

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

3. TERMS DESCRIBING CONSISTENCY (COHESIVE SOILS ONLY)

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

NOTE: Hierarchy of Soil Strength Prediction

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

4. TERMS DESCRIBING DENSITY (COHESIONLESS SOILS ONLY)

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

5. LEGEND FOR RECORDS OF BOREHOLES

SYMBOLS AND ABBREVIATIONS FOR SAMPLE TYPE

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

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

▽ Water Level

C_{pen} Shear Strength Determination by Pocket Penetrometer

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

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

UNIFIED SOILS CLASSIFICATION

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

EXPLANATION OF ROCK LOGGING TERMS

ROCK WEATHERING CLASSIFICATION

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

SYMBOLS

CLAYSTONE

SILTSTONE

SANDSTONE

COAL

Bedrock (general)

DISCONTINUITY SPACING

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

TERMS

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

STRENGTH CLASSIFICATION

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

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RECORD OF BOREHOLE No 05-8

METRIC

W.P. 140-88-00
LOCATION N 5 329 117.20 E 375 070.79
ORIGINATED BY MF

HWY 112
BOREHOLE TYPE Hollow Stem Augers/NQ Coring
COMPILED BY WM

DATUM Geodetic
DATE 16.06.05 - 21.06.05
CHECKED BY PJB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%) (Wp / w / WL)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
309.4 / 0.0	Silty **CLAY**, trace sand Stiff to Firm Brown				309		
		1	SS	10			
		2	SS	9	308		
		3	SS	6	307		0 3 49 48
		4	SS	9			
306.0 / 3.4	**SAND**, some gravel, trace silt Brown Wet			FI	306		
305.5 / 3.9	**BEDROCK**, Gneiss (meta- volcanic), dark grey, with white sub-vertical calcite veins, fresh	1	RUN	5 6			RUN 1# TCR=100%, SCR=20%, RQD=0%, UCS=232.2MPa
		2	RUN	4 4 6 4 >25	305 304		RUN 2# TCR=100%, SCR=67%, RQD=48%, UCS=188.1MPa
302.9 / 6.5		3	RUN	4 2			RUN 3# TCR=100%, SCR=70%, RQD=55%, UCS=171.1MPa
	END OF BOREHOLE AT 6.45 m. BOREHOLE BACKFILLED WITH BENTONITE HOLE PLUG TO SURFACE.						

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT, NATURAL MOISTURE CONTENT, LIQUID LIMIT, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³

+³, ×³: Numbers refer to Sensitivity   20 15 5 10 (%) STRAIN AT FAILURE

Ministry of Transportation Ontario

THURBER

RECORD OF BOREHOLE No 05-9

METRIC

W.P. 140-88-00 LOCATION N 5 329 223.74 E 375 117.58 ORIGINATED BY MF

HWY 112 BOREHOLE TYPE Hollow Stem Augers COMPILED BY WM

DATUM Geodetic DATE 20.06.05 - 20.06.05 CHECKED BY PJB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
309.3							
0.0 / 309.0	**TOPSOIL**, some rootlets, Dark Brown						
0.3 / 308.3	Silty **SAND**, some clay, trace gravel, Loose, Brown, Moist	1	SS	5	309		
0.9	Silty **CLAY**, trace sand, Firm to Soft, Brown/Grey	2	SS	6	308		
		3	SS	3			
	Becoming Grey (Brown / Grey)	4	SS	2	307		0 1 50 49
	occasional iron oxide staining	5	SS	2	306		
		6	SS	0	305		
						2.4 (field vane, sensitivity)	
		7	SS	1	304		0 0 46 54
303.2	augers grinding at 6.1 m						
6.1 / 302.9	**SAND**, some gravel, trace silt, Brown, Wet				303		
6.4	END OF BOREHOLE AT 6.40 m. AUGER REFUSAL AT 6.40 m ON PROBABLE BEDROCK OR BOULDER.						

Piezometer installation consists of 19 mm diameter Schedule 40 PVC pipe with a 1.52 m slotted screen.

WATER LEVEL READINGS:

DATE	DEPTH (m)	ELEVATION (m)
20/06/05	2.07	307.2
21/06/05	1.61	307.7

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3; STRAT PLOT; GROUND WATER CONDITIONS.

+ ^{3}, × ^{3}: Numbers refer to Sensitivity   15 20 5 10 (%) STRAIN AT FAILURE

Appendix B

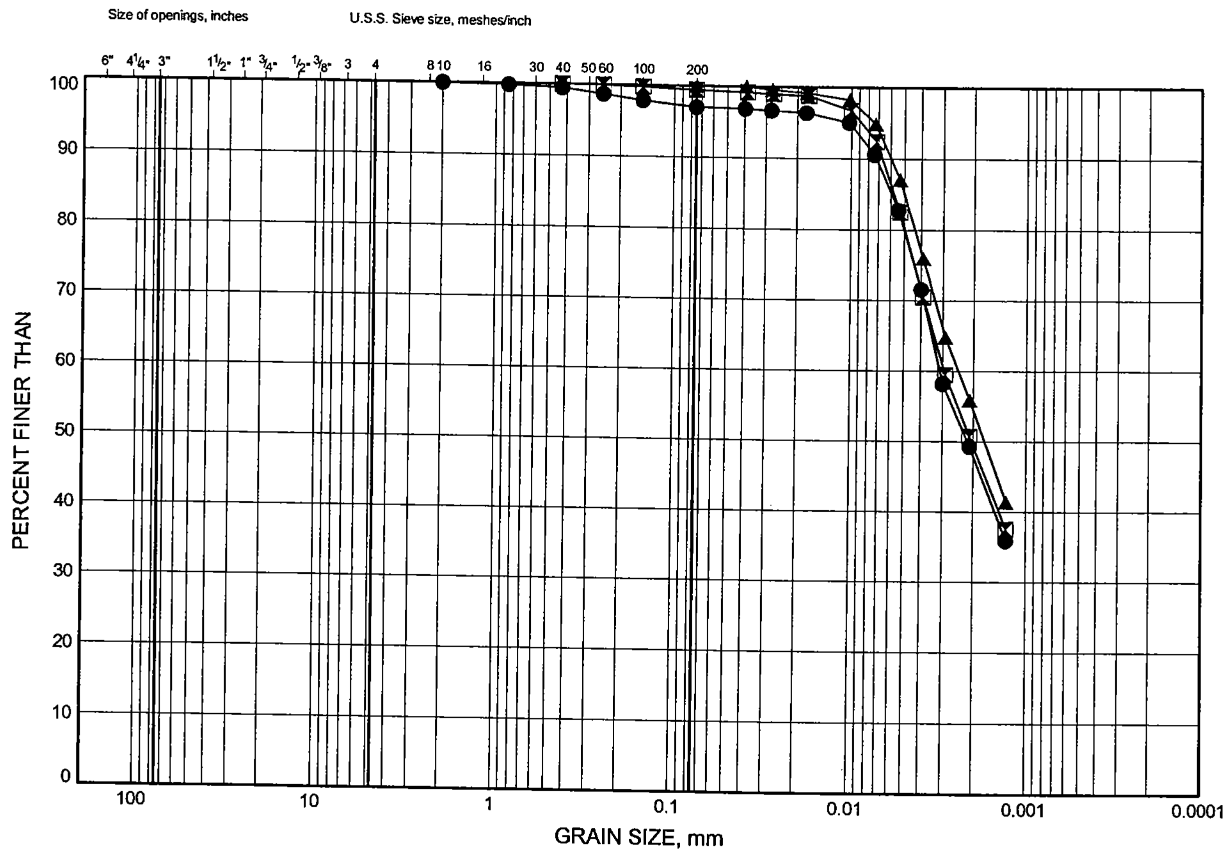
Laboratory Test Results

Hwy 112-ONR Overhead Replacement

GRAIN SIZE DISTRIBUTION

FIGURE B1

SILTY CLAY



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

SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	05-8	2.59	306.81
⊠	05-9	2.59	306.67
▲	05-9	5.79	303.47

Date August 2005

Project 140-88-00

THURBER

Prep'd HS

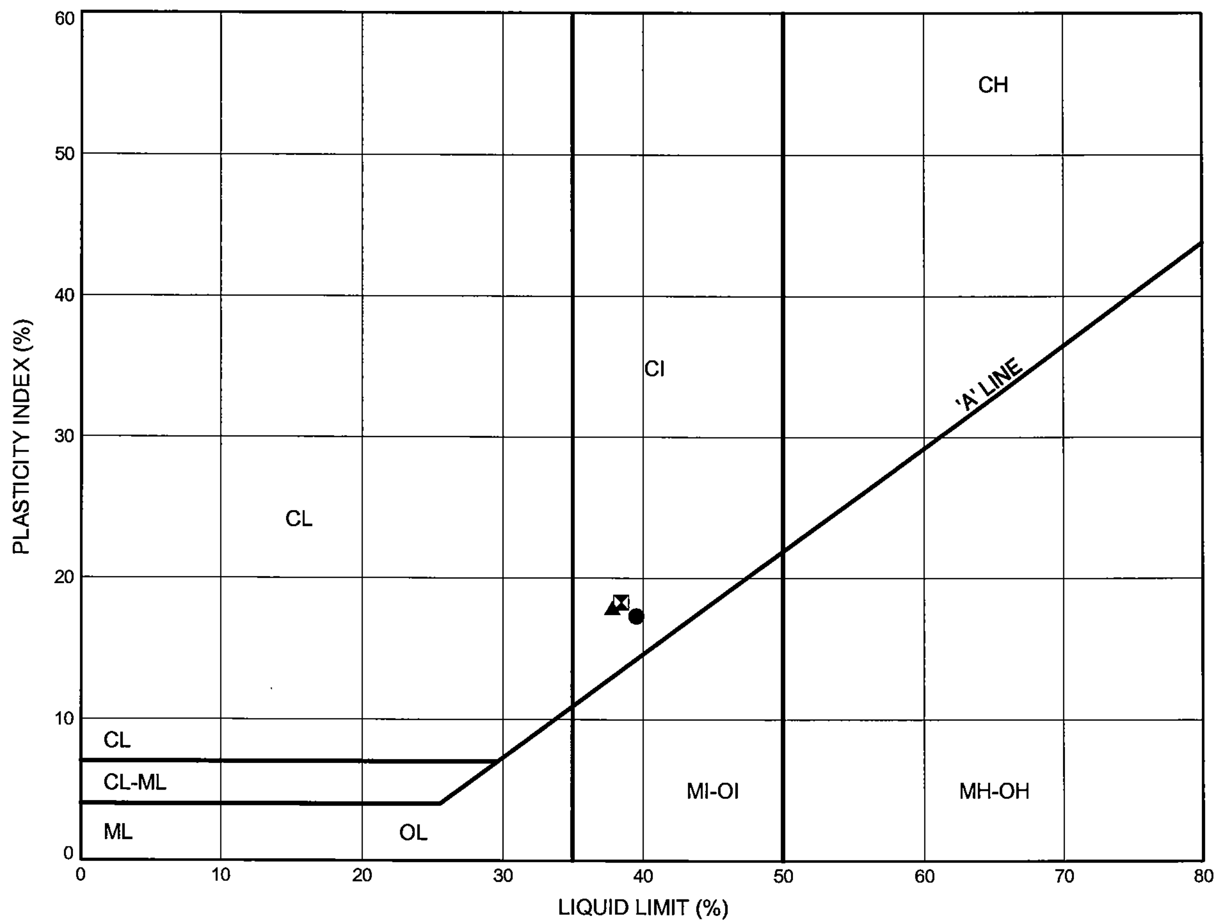
Chkd. AEG

Hwy 112-ONR Overhead Replacement

ATTERBERG LIMITS TEST RESULTS

FIGURE B2

SILTY CLAY



SYMBOL	BH	DEPTH (m)	ELEV. (m)
●	05-8	2.59	306.81
⊠	05-9	2.59	306.67
▲	05-9	5.79	303.47

THURBALT 6416.GPJ 15/09/05

Date September 2005

Project 140-88-00

THURBER

Prep'd HS

Chkd. AEG

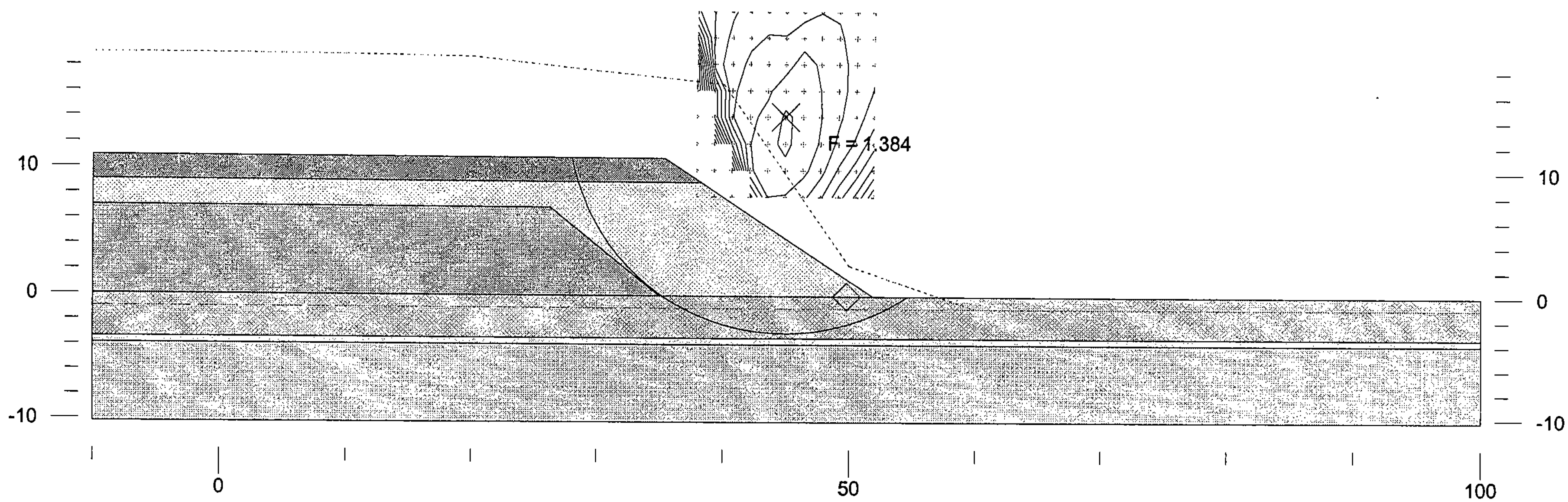
Appendix C

Selected Slope Stability Output

Thurber Engineering Ltd. - Toronto
15-64-16
Hwy 112 - ONR Overpass
August 2005
South Abutment - Short Term - Granular B & Rock Fill
BH05-8 - Height = 11m - One Construction Stage

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.	Area above constr. line
Gran. Surcharge	22	0	35	0	0	92.4
Granular B	22	0	35	0	0	
Rock Fill	19	0	42	0	0	
Brown Clay	19	50	0	.2	2	
Sand	22	0	35	0	1	
Hard Bottom	(Infinitely Strong)					

92.4 = Total Area



8/5/2005 4:28:34 PM C:\THURBE~1\15\MTO\64-16H~1\STABIL~1\BH06-H1.GSL Thurber Engineering Ltd. - Toronto F = 1.384

FIGURE C1

Thurber Engineering Ltd. - Toronto
15-64-16
Hwy 112 - ONR Overpass
August 2005
South Abutment - Long Term - Granular & Rock Fill
BH05-8 - Height = 11m - One Construction Stage

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.	Area above constr. line
Granular B	22	0	35	0	0	92.4
Rock Fill	19	0	42	0	0	
Brown Clay	19	0	28	0	1	
Sand	22	0	35	0	1	
Hard Bottom	(Infinitely Strong)					

92.4 = Total Area

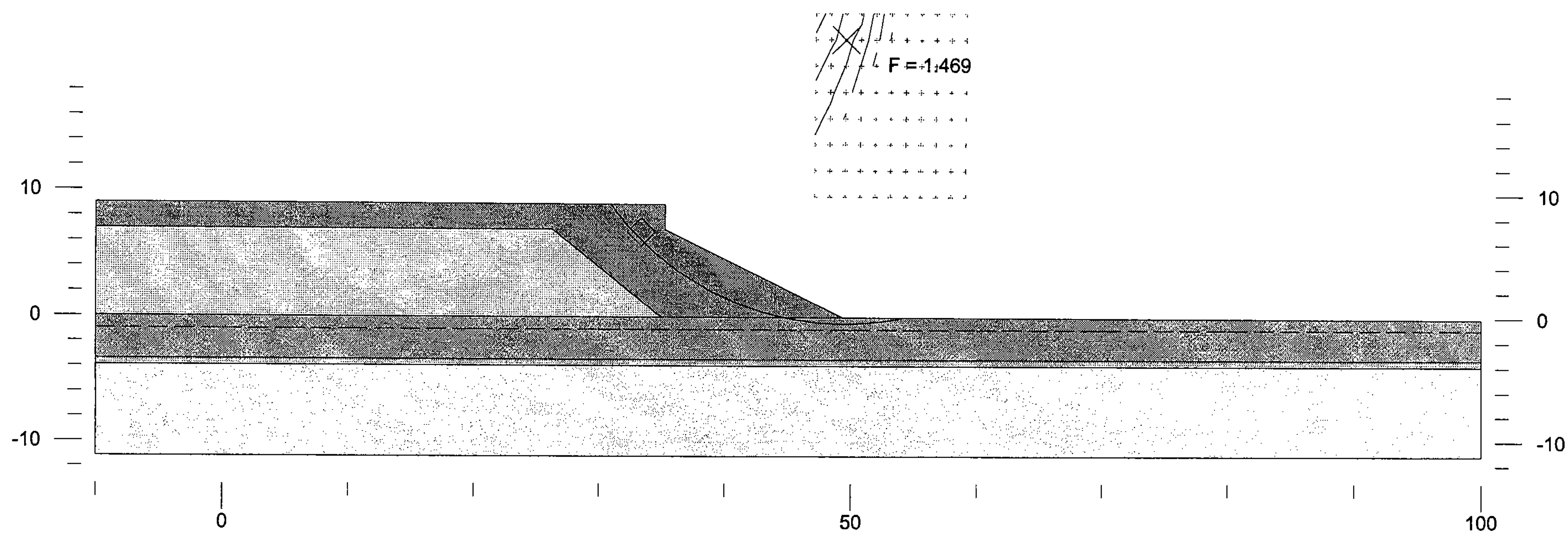


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FIGURE C2

Thurber Engineering Ltd. - Toronto
15-64-16
Hwy 112 - ONR Overpass
August 2005
North Abutment - Short Term - Rock Fill
BH05-9 - Height = 9m to 11m; 15m x 4m high Berm - Second and Last Construction Stage

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.	Area above constr. line
Berm	22	0	35	0	0	92.4
Gran. Surcharge	22	0	35	0	0	
Granular B	22	0	35	0	0	
Rock Fill	19	0	42	0	0	
Silty Sand	19	0	30	0	0	
Brown Clay	19	50	0	.2	2	
Grey Clay	18	25	0	.2	3	
Hard Bottom	(Infinitely Strong)					

92.4 = Total Area

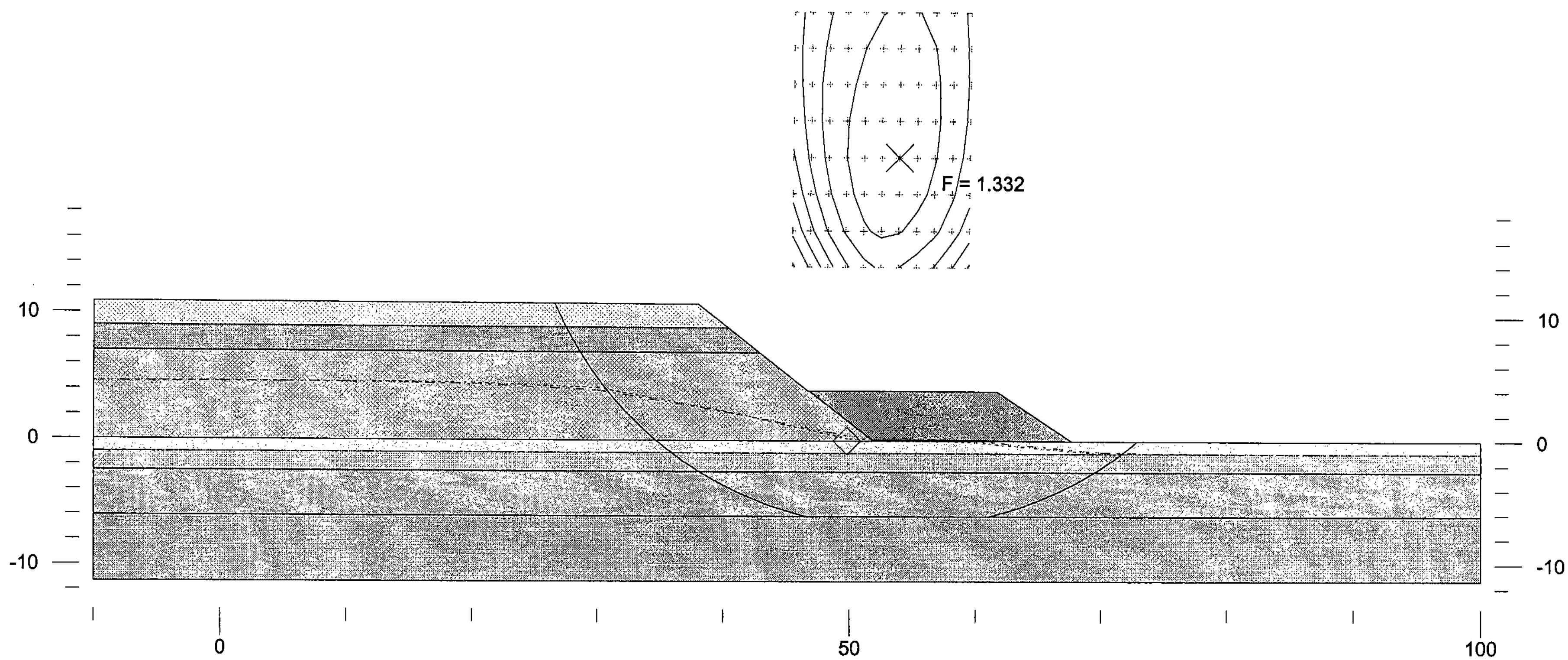

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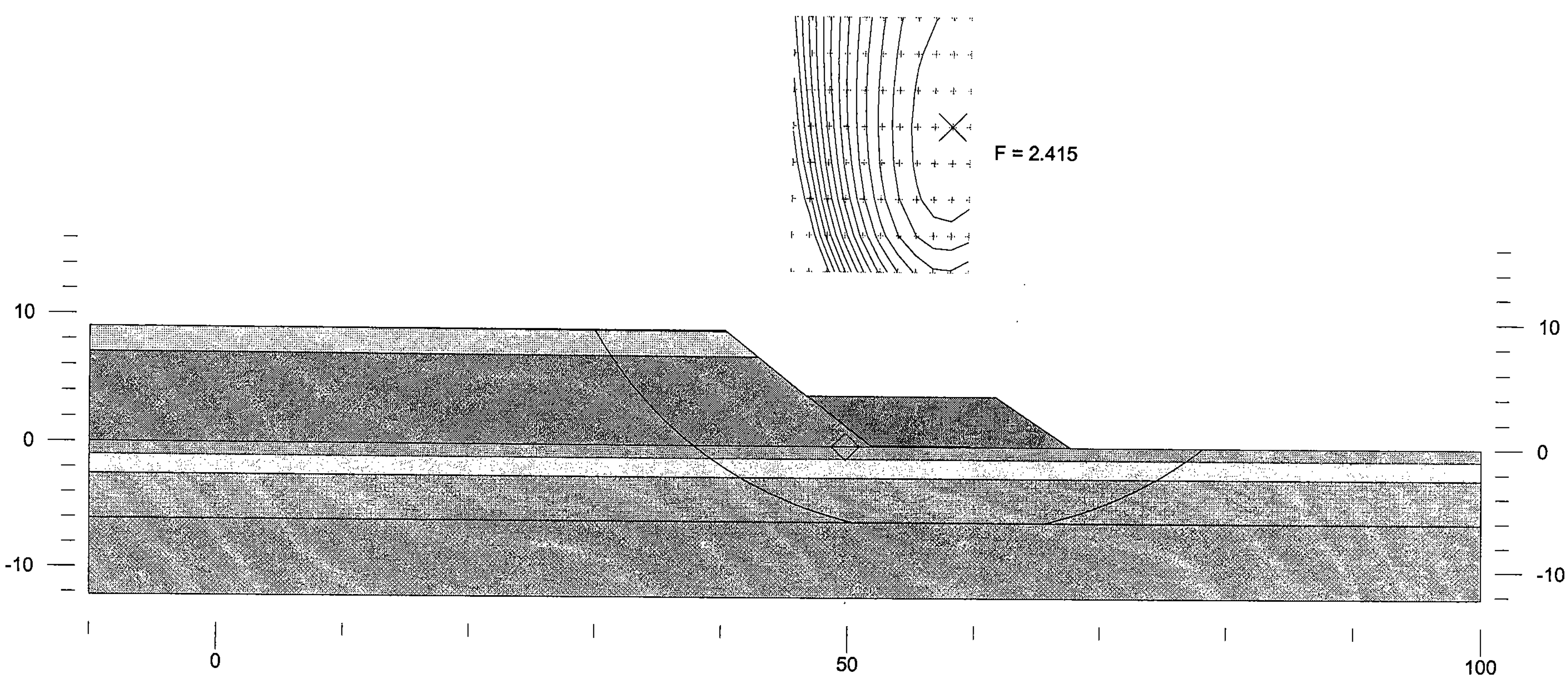
FIGURE C3

Thurber Engineering Ltd. - Toronto
15-64-16
Hwy 112 - ONR Overpass
August 2005
North Abutment - Long Term - Rock Fill
BH05-9 - Height = 11m; 15m x 4m high Berm

	Gamma kN/m3	C kPa	Phi deg	Min c/p	Piezo Surf.	Area above constr. line
Berm	22	0	35	0	0	92.4
Granular B	22	0	35	0	0	
Rock Fill	19	0	42	0	0	
Silty Sand	19	0	30	0	0	
Brown Clay	19	0	28	0	1	
Grey Clay	18	0	25	0	1	
Hard Bottom	(Infinitely Strong)					

92.4 = Total Area



8/5/2005 4:30:18 PM C:\THURBE~1\15\MTO\64-16H~1\STABIL~1\BH06-G6L.GSL Thurber Engineering Ltd. - Toronto F = 2.415

FIGURE C4

Appendix D

Foundation Comparison

COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

Driven Piles	Footing on Native Soil	Footing on Engineered Fill	Caisson
Advantages: i. Piles will develop high geotechnical resistance on the bedrock. ii. Allows choice of conventional, integral or semi-integral abutment design. iii. Readily installed. ***Disadvantages:*** i. Higher unit costs than footings. ii. Bedrock may not be deep enough at south abutment to allow integral abutment, depending on final bridge arrangement.	***Advantages:*** i. Ease of construction. ii. Allows choice of conventional or semi-integral abutment. iii. Lower cost than deep foundations. ***Disadvantages:*** i. Low geotechnical resistance available at this site. ii. Potential for unacceptable magnitude of settlement. **NOT RECOMMENDED**	***Advantages:*** i. Would permit use of higher geotechnical resistance than is available on the native soil. ii. Allows choice of conventional or semi-integral abutment. iii. Allows use of perched abutments. iv. Lower cost than deep foundations. ***Disadvantages:*** iii. Cost of constructing engineered fill. iv. Low geotechnical resistance available at this site. v. Potential for unacceptable magnitude of settlement. **NOT RECOMMENDED**	***Advantages:*** i. High resistance is available for caissons founded on bedrock. ii. Choice of conventional or semi-integral abutment design. ***Disadvantages:*** i. Possibly the most costly alternative since soil conditions will require either drilling a socket or sealing a liner at the bedrock surface.

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Appendix E

Special Provisions

The following Special provisions are referenced in this report:

- Amendment to OPSS 206, December 1993 – dated November 2002
- Special Provision No. 902S01
- Special Provision No. 903S01
- Special Provision No. 105S10
- OPSS 572

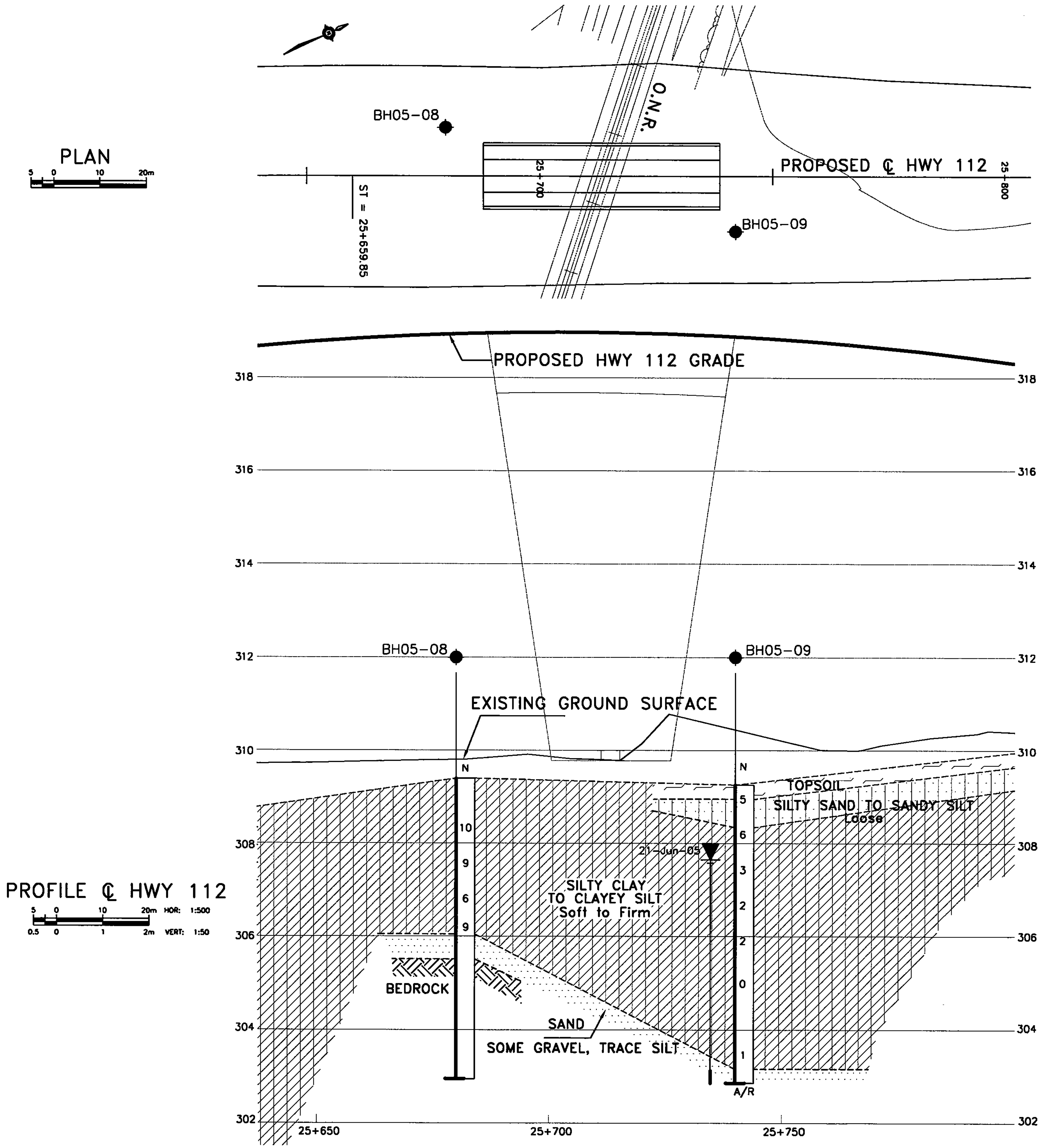
Appendix F

Site Photographs

Appendix G

Drawings

THURBER



METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

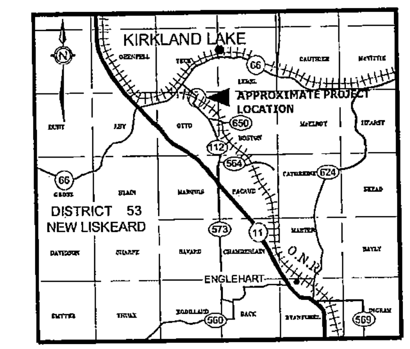
HWY 112

GWP NO. 140-88-00

HIGHWAY 112
ONR OVERHEAD REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

THURBER ENGINEERING LTD.



KEYPLAN

	LEGEND
●	Bore Hole
⊕	Dynamic Cone Penetration Test (cone) or Probe Hole
◆	Bore Hole & Cone
N	Blows/0.3m (Std pen Test, 475J/blow)
CONE	Blows/0.3m (60° Cone, 475J/blow)
PH	Pressure, Hydraulic
▼	WL in Piezometer at Time of Investigation (Date)
⊻	Head Artesian Water
	Piezometer
▼	WL in Open Borehole Upon Completion of Drilling
90%	Rock Quality Designation (RQD)
A/R	Auger Refusal
C/R	Cone Refusal

NO	ELEVATION	NORTH	EAST
BH05-08	309.4	5329177.2	375070.8
BH05-09	309.3	5329223.7	375117.6

— NOTE —

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

DRAWING NOT TO BE SCALED
100 mm ON ORIGINAL DRAWING

REVISIONS DATE	BY	DESCRIPTION			
DESIGN AEG	CHK PJB	CODE CHBDC	LOAD		DATE AUG . 2005
DRAWN HS	CHK AEG	SITE	STRUCT	SCHEME	DWG 1