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REPORT ON

FOUNDATION INVESTIGATION AND DESIGN
CN RAIL OVERPASS WIDENING
HIGHWAY 417 EASTBOUND
STRUCTURE SITE 27-213/1
W.P. 258-98-00
CASSELMAN, ONTARIO

Submitted to:

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TABLE OF CONTENTS

TABLE OF CONTENTS - continued

LIST OF TABLES

LIST OF DRAWINGS

LIST OF FIGURES

TABLE OF CONTENTS - continued

LIST OF APPENDICES

PART A

FOUNDATION INVESTIGATION AND DESIGN
CN RAIL OVERPASS WIDENING
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield (MH) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a foundation investigation as part of the detailed design for the upgrading of Highway 417 between the Limoges Road and Casselman Road interchanges.

The terms of reference for the scope of work are outlined in Golder's proposal P31-2107 dated August 2003, that forms part of the Consultant's Agreement (Number P.O. 4005-A-000316) for this project. This report addresses widening of the existing overpass bridge structure which carries the eastbound lanes of Highway 417 over the CN Rail line near Casselman, Ontario. The work was carried out in accordance with the Quality Control Plan for this project dated February 2004.

2.0 SITE DESCRIPTION

The existing CN Rail overpass structure is located on Highway 417 approximately 600 m east of Country Road No. 7 (St. Albert Street) in the Village of Casselman, Ontario. The site is about 1.0 km south east of the center of the Village of Casselman and approximately 2.3 km southeast of the CN Rail crossing over the South Nation River. Through this section, Highway 417 consists of two eastbound lanes (EBL) and two westbound lanes (WBL) divided by about a 30 m to 40 m inside median. The eastbound and westbound lanes are carried across the CN Rail line on separate structures. The existing eastbound structure is designated as MTO's Structure Site 27-213/1. The CN Rail tracks are carried on an embankment which is about 1.5 to 1.8 m high with shallow drainage ditches on each side.

The existing bridges for both the eastbound and westbound lanes consist of a concrete deck on precast concrete girders supported on concrete abutments and piers. The bridges consist of three spans of approximately 22, 26 and 22 m. The foundation investigations for the design of these two bridges were carried out in 1970 and the results of those investigations are summarized in the Ministry of Transportation, Ontario GEOCRES No. 31G - 48, *Foundation Investigation Report for Proposed Crossing at Hwy. 417 Eastbound and Westbound Lanes and the Canadian National Railway, Twp. Of Cambridge - Co. of Russell, District No. 9 (Ottawa), W.J. 70-F-7 – W.P. 35-66-17.* The Department of Highways Ontario Bridge Division drawing D6839-1, *Canadian National Railways Overhead – E.B.L.* dated December 1970 indicates the EBL abutment structures to be supported on 12 battered steel 'H' piles while the two (2) piers are supported on shallow foundations. The bridge deck is relatively level at about Elev. 74.7 m.

It is currently proposed to widen the eastbound lanes of Highway 417 in this area to three lanes to accommodate ramp improvements at the nearby interchange to Casselman. The existing EBL structure is too narrow and is proposed to be widened by about 5.6 m to accommodate the additional traffic lane and wider shoulders. Widening of the existing approach embankment by about 5 m with new embankment fill up to about 9.0 to 10.0 m high, is also required.

The widening will be accommodated by widening the existing structure and replacing the existing concrete bridge deck with a thicker concrete deck. The existing pier and abutment foundations will be widened and structurally connected to the existing structures. The new bridge deck elevations are expected to remain at about Elev. 74.7 m.

The results of the 1970 foundation investigations indicate that the overburden at the CN Rail overpass structures consists of a relatively thin layer of clay over a glacial till deposit of variable thickness. The glacial till is underlain by limestone bedrock at depths ranging from 4.0 to 7.0 m.

Regional geologic maps indicate that the clay is an offshore marine deposit of the Champlain Sea Sediments and that the bedrock is of the Ottawa Formation. The regional geologic maps also indicate that evidence of ancient landsliding and erosional terraces is present about 300 metres northwest of the site.

A foundation investigation has also been carried out for the design of the widening of the approach embankments to this structure. The results of that investigation are provided in a report titled "Foundation Investigation and Design, EB CN Rail Overpass East and West Approach Embankment Widening, Highway 417, W.P. 258-98-00, Casselman, Ontario" (Golder report number 04-1120-013-7000).

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out for the widening of the CN Rail overpass structure between May 6 and May 12, 2004. During this time, a total of eight (8) sampled boreholes were advanced within the area of the proposed structure and approach embankment widening. Two boreholes (04-101 and 04-102) were drilled through the existing piers, two boreholes (04-103 and 04-104) were drilled at the location of the proposed pier widenings, two boreholes (04-105 and 04-106) were drilled at the closest accessible points to the abutment widenings, and two boreholes (04-107 and 04-108) were advanced at the toes of the existing embankments.

Borehole 04-103 and boreholes 04-105 to 04-108 were advanced using 108 mm inside diameter (I.D.) continuous flight hollow stem augers on a track-mounted drill rig, supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario. Boreholes 04-101, 04-102 and 04-104 were advanced using a manually operated portable drill rig supplied and operated by Marathon Drilling Ltd.

Borehole 04-101 was advanced through the existing west pier footing to a depth of 3.8 m below the existing ground surface. Borehole 04-102 was advanced through the existing east pier footing to refusal at a depth of 5.7 m below the existing ground surface and then continued into the bedrock to a final depth of 6.3 m using rotary diamond drilling techniques. Samples of bedrock in this borehole were obtained using a 'BQ' size rock core barrel. Samples of the overburden in these two boreholes were obtained at 0.6 m to 0.75 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure, where possible. Rotary diamond drilling techniques were required to penetrate the overburden beneath the pier footings, due to the presence of cobbles and boulders, which limited the sampling that could be performed.

Boreholes 04-103 and 04-104, at the pier widenings, were advanced to auger refusal at depths of 6.6 m and 5.0 m, respectively. Boreholes 04-105 to 04-108 were advanced to auger refusal at depths ranging from 4.7 m to 5.9 m. In these six boreholes, samples of the overburden were obtained at 0.6 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure, where possible. Rotary diamond drilling techniques were required to penetrate the overburden for short intervals of depth in boreholes 04-103 and 04-104 due to the presence of cobbles and boulders. Boreholes 04-103 to 04-106 were advanced past auger refusal to final depths of 8.3 m to 10.1 m into the bedrock using rotary diamond drilling techniques. Samples of bedrock were obtained using a 'BQ' size rock core barrel in Borehole 04-104 and by using an 'NQ' size rock core barrel in boreholes 04-103, 04-105 and 04-106. In-situ vane testing (N vanes) were carried out within the cohesive deposits encountered in Borehole 04-106.

The water levels in the open boreholes was observed throughout the drilling operations and a total of three standpipe piezometers were installed in boreholes 04-104, 04-105 and 04-106 to monitor the groundwater levels at the site. The screened portion of each standpipe was installed within the overburden soils immediately above the bedrock surface at depths of about 5 m. The standpipes consist of 20 mm diameter rigid PVC pipe with a 1.5 m long slotted screen section, installed within silica sand backfill and sealed below minimum 0.3 m long sections of bentonite pellet backfill. The water level in the standpipe piezometers was measured on May 20, 2004. The field work was supervised by a senior technician from our staff who located the boreholes and augerholes, observed the drilling operations, logged the borings, directed the in-situ testing and took custody of the soil samples retrieved.

The samples were identified in the field, placed in appropriate containers, labeled and transported to our Ottawa geotechnical laboratory where the samples underwent further detailed visual examination and laboratory testing. All of the laboratory tests were carried out to MTO and/or ASTM standards as appropriate.

In addition to the field investigation and laboratory testing program described above, the previously referenced companion investigation for the design of the approach embankment widening included one borehole (number 05-201) put down in near proximity to Borehole 04-106. A Shelby tube sample was retrieved from the cohesive soils encountered in Borehole 05-201 and laboratory oedometer consolidation testing carried out. Those results are also relevant to the present investigation.

The borehole locations were selected by Golder Associates personnel with input from Morrison Hershfield (MH) at specific locations of interest. The ground surface elevations at the borehole locations were provided by MH and are understood to be referenced to Geodetic datum.

The borehole locations, including MTM NAD83 northing and easting coordinates, and ground surface elevations referenced to geodetic datum are summarized in the following table and are shown on Drawing 1.

Borehole No.	Borehole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
04-101	Existing West Pier	5019200.24	417004.17	66.0
04-102	Existing East Pier	5019204.56	417029.57	65.8
04-103	Proposed West Pier	5019191.01	417016.72	65.9
04-104	Proposed East Pier	5019197.85	417038.36	65.2
04-105	West Abutment	5019175.50	417014.44	65.6
04-106	East Abutment	5019181.70	417072.51	65.5
04-107	West Toe of Embankment	5019168.47	416964.75	65.8
04-108	East Toe of Embankment	5019185.64	417091.69	65.1

The boreholes were backfilled with bentonite pellets, mixed with native soils, and the site conditions restored following completion of the field work.

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

The study area for this assignment lies within the minor physiographic region known as the Ottawa Valley Clay Plain that lies within the major physiographic region of the Ottawa-St. Lawrence Lowland (Chapman and Putnam, 1984). This physiographic region underlain primarily by limestones of the Ottawa Formation that are, in turn, underlain by a series of sedimentary rocks, consisting of sandstones, dolostones, limestones and shales. These sedimentary formations are underlain by igneous and metamorphic bedrock of the Precambrian Shield. The Ottawa Valley Clay Plain region, present along Highway 417 in this area, is characterized by relatively thick deposits of sensitive marine clay, silt and silty clay that were deposited within the Champlain Sea basin. These deposits, known as the Champlain Sea clay or Leda clay, overlie relatively thin, commonly reworked glacial till and glaciofluvial deposits, that in turn overlie bedrock (Chapman and Putnam, 1984). The study area is located within a small surficially discontinuous region of the Ottawa Valley Clay Plain. This area lies within an abandoned channel of the South Nation River and the silty clays have been mostly removed by fluvial erosion to expose a till plain. Thin layers of clay and silt overlie the glacial till in some portions of the study area.

4.2 Site Stratigraphy

As part of the subsurface investigation at this site, six boreholes were advanced within the limits of the foundation elements and two boreholes were advanced within the footprint of the embankment widenings. The borehole locations and ground surface elevations are shown on Drawing 1.

The detailed subsurface soil, bedrock and groundwater conditions as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil samples, are given on the attached Record of Borehole sheets. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests (SPTs). These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole locations.

In summary, the subsurface conditions consist of limited thicknesses of surficial fill material overlying about 0.4 to 1.8 m of generally stiff silty clay (with a localized thin intermediate layer of sandy silt and clayey silt). Portions of the silty clay in Boreholes 04-106, 04-108, and 05-201 have a soft to firm consistency. The silty clay deposit is generally underlain by a layer of sandy silt or clayey silt with a thickness of 0.3 to 0.7 m.

The sandy silt or clayey silt is underlain by a 1.0 to 4.4 m thick layer of sandy silt till which contains occasional cobbles and boulders. The till is underlain by limestone bedrock the surface of which is relatively flat.

A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections. Profiles and sections of this site are shown on Drawing 2.

Although direct reference is not made to the boreholes from the original investigation for this structure, the borehole logs from that investigation are included in Appendix A.

4.2.1 Topsoil

At the embankment toes the topsoil thickness ranges from 0 to 200 mm and is overlain by 600 mm of fill at Borehole 04-107.

4.2.2 Fill

The fill above the underside of the pier footings consists of sand and gravel with some silt present at the east pier location. The Standard Penetration Test (SPT) 'N' values within the fill at the location of the east pier varied from 1 to 6 blows per 0.3 m of penetration indicating that this fill is very loose to loose. A natural water content of 21 percent was measured on a selected sample of this fill.

The fill encountered at the toes of the embankments is highly variable, ranging from silty clay to silty sands and sandy silts. The depth of the fill ranged from 0.4 m to 0.7 m.

4.2.3 Silty Clay to Clay

A layer of silty clay was encountered below the fill at boreholes 04-103 to 04-108, as well as Borehole 05-201 from the companion investigation for the embankment widening. At boreholes 04-104, 04-106 and 04-107 the silty clay was overlain by a thin layer of clayey to sandy silt. The thickness of the silty clay ranges from about 0.4 m to 1.8 m at the toes of the existing embankments. The deposit is generally composed of a weathered brown to grey brown crust containing occasional sand seams.

The results of an in situ vane test carried out in Borehole 04-106 in the lower portion of the weathered silty clay is shown on the Record of Borehole sheets. Based on the in situ vane test, the undrained shear strength of the upper weathered silty clay is 50 kPa or greater. The sensitivity of the weathered silty clay, as estimated from the in situ vane test, is about 7 implying that the silty clay is typically of medium to extra-sensitivity (CFEM, 1992).

The natural water content measured on selected samples of the weathered silty clay ranged from about 53 to 60 percent. The results of a laboratory grain size distribution test carried out on a representative sample of the unweathered clay is presented on Figure 1.

At Borehole 04-106 a 1.6 m thick layer of unweathered grey silty clay underlies the brown-grey weathered silty clay. A Standard Penetration Test (SPT) carried out in the unweathered clay measured an 'N' value of 0 blows per 0.3 m of penetration (static weight of hammer and rods). The results of in-situ vane testing carried out within the grey silty clay gave undrained shear strength values ranging from 32 to 36 kilopascals indicating a firm consistency. In situ vane testing carried out on remoulded grey silty clay gave undrained shear strengths ranging from 6 to 11 kilopascals, implying that the silty clay is typically sensitive to extra-sensitive (CFEM, 1992).

The results of an Atterberg limit test carried out on a sample of the grey silty clay gave a plastic limit value of 25 percent and a liquid limit value of 82 percent, indicating a clay of high plasticity. The measured water content of the grey silty clay in Borehole 04-106 was 80 percent (see Figure 2).

As described above, laboratory oedometer consolidation testing was performed on a sample of the silty clay retrieved from Borehole 05-201 of the investigation for the embankment widening. The results of that testing are summarized below.

Borehole (Sample)	Elevation (Depth) (m)	σ_{vo}' (kPa)	σ_p' (kPa)	OCR	$\sigma_p' - \sigma_{vo}'$ (kPa)	e_o	C_r	C_c
05-201 (3)	61.9 (3.4)	40	105	2.6	65	1.89	0.018	1.35

where : σ_{vo}' is the calculated effective overburden pressure in kPa
σ_p' is the pre-consolidation pressure in kPa
OCR is the overconsolidation ratio (σ_p'/σ_{vo}')
$\sigma_p' - \sigma_{vo}'$ is the available overconsolidation
e_o is the initial void ratio
C_r is the recompression index
C_c is the compression index

A summary of the results of that testing are provided in Appendix B.

A summary of the coefficient of consolidation (c_v) data from the laboratory oedometer consolidation testing for the embankment widening investigation, including the results for Borehole 05-201, is also provided in Appendix B. These results for Borehole 05-201 indicate that the coefficient of consolidation of the deposit at stress levels below the preconsolidation pressure is in the order of 8 x 10^{-3} cm^2/s. Above the deposit's preconsolidation pressure, the coefficient of consolidation is indicated to be about 10^{-3} to 10^{-4} cm^2/s. It is noted however that relatively small load increments (i.e., not a doubling of each load step, as is conventionally the case) were used for the consolidation test since the material is quite sensitive and therefore smaller load increments are needed to properly define the sharp break in the load-deformation curve which identifies the preconsolidation pressure. Those smaller load increments can result in somewhat of an underestimation of the coefficient of consolidation, since the excess pore pressures generated during each load increment are relatively smaller than if conventional load increments are used. Therefore the actual coefficient of consolidation is likely near the higher end of the ranges specified above.

4.2.4 Clayey Silt, Sandy Silt and Sands

Above the weathered silty clay in boreholes 04-104, 04-106 and 04-107 was a thin layer of clayey silt to sandy silt trace to some clay. This layer varied in thickness between 0.4 m and 0.6 m.

Above the weathered silty clay in Borehole 04-108 was a thin layer of silty clay and fine sand with a thickness of 0.5 m.

Underlying the silty clay in Boreholes 04-103 to 04-106 and 04-108 is a thin layer of clayey silt and sandy silt ranging from 0.3 m to 0.7 m in thickness.

A 0.3 m thick layer of fine to medium sand trace gravel was encountered in Borehole 04-101 beneath the footing for the west pier. A Standard Penetration Test (SPT) measured an 'N' value of 4 blows per 0. 3 m of penetration indicating a very loose state of packing.

4.2.5 Sandy Silt Till

A 0.7 m to 4.4 m thick layer of sandy silt till was encountered below the silty clay and clayey to sandy silt stratum in all of the sampled boreholes at the toes of the embankments.

The sandy silt and gravel till was encountered below the east pier foundation and underlying the sands under the west pier foundation. At Borehole 04-102, at the east pier foundation, the thickness of the till layer from footing level to the surface of the bedrock was 1.3 m.

Standard Penetration Tests (SPT) measured 'N' values ranged between 6 and 57 blows for 0.3 m of penetration indicating a loose to very dense state of packing. The results of laboratory grain size distribution tests carried out on representative samples of the sandy silt till are summarized on Figure 3. The natural water contents of this till material ranged from 7 to 10 percent.

4.2.6 Limestone Bedrock

Limestone bedrock underlies the sandy silt till deposit at this site. In the five boreholes where bedrock was proved by coring the surface of the bedrock was encountered between elevations 58.7 m and 60.3 m. The following table summarizes the bedrock surface depth and elevation as encountered at the borehole locations.

Borehole Number	Ground Surface Elevation (m)	Depth to Bedrock (m)	Bedrock Surface Elevation (m)
04-102	65.8	5.7	60.1
04-103	65.9	6.6	59.3
04-104	65.2	5.0	60.2
04-105	65.6	5.2	60.4
04-106	65.5	6.8	58.7

The limestone bedrock at the site is a member of the Ottawa Formation; it is medium strong, thinly- to medium-bedded. Rock Quality Designation (RQD) values measured on recovered bedrock core samples ranged from 49 to 100 percent (with an average of 78 percent) in the upper 0.3 m to 0.6 m of the bedrock, and from 84 to 100 percent in the lower 3 to 5 m of the recovered bedrock core. Typical RQD values indicate that the bedrock is generally of good to excellent quality. The discontinuities observed in the rock core are typically horizontal to sub-horizontal, associated with the bedding planes.

A description of some of the terms used in the description of the bedrock samples from this site is provided on the *Lithological and Geotechnical Rock Description Terminology* sheet that precedes the Record of Borehole sheets included with this report.

4.3 Groundwater Conditions

Three standpipe piezometers were installed within the overburden soil deposits at this site in boreholes 04-104, 04-105 and 04-106. The water levels were measured in the piezometers on May 20, 2004. The observations are summarized in the following table:

Borehole No.	Depth to bottom of screen (m)	Water Level on May 20, 2004	
		Elevation (m)	Depth (m)
04-104	5.0	64.1	1.1
04-105	5.2	64.4	1.2
04-106	5.2	64.3	1.2

It should be noted that groundwater levels are expected to fluctuate seasonally and are expected to rise during wet periods of the year.

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PART B

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CN RAIL OVERPASS WIDENING
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5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides foundation design recommendations for the proposed widening of the Highway 417 bridge structure over the CN Rail line. The recommendations are based on interpretation of the factual geotechnical data obtained from the boreholes advanced during the subsurface investigation at this site.

The interpretation and recommendations provided are intended only to provide the designers with sufficient information to assess the feasible foundation alternatives and to design the proposed structure foundations. As such, where comments are made on construction they are provided only in order to highlight those aspects which could affect the design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

5.2 Bridge and Retaining Wall Foundation Options

It is understood that the proposed EBL widening of the Highway 417 bridge will maintain the three-span pre-cast girder construction of the existing structures. The girders are to be supported on concrete abutments, which are in turn supported on the foundations. The existing abutments are founded on driven steel H-pile foundations while the piers are supported on shallow foundations essentially bearing on till.

The shallow foundations at the existing piers are founded on till (or a thin sand fill overlying the till) at Elev. 62.8 m at the west pier and Elev. 61.2 m at the east pier. At the pier widening locations, the till present at these same elevations would also be suitable to support the piers using shallow foundations.

Alternatives to using shallow foundations on till for the piers include: shallow foundations on bedrock, steel H-piles driven to bedrock, or cast-in-place concrete caissons supported on or within bedrock. All of these alternatives are feasible but would likely be more expensive. In addition, the additional excavation required to place spread footings on bedrock would require the use of temporary shoring (or more extensive shoring) to support the rail line. More groundwater control would also be required versus the more shallow excavation needed for footings on the glacial till. As such, it is proposed that the pier widenings be supported on shallow foundations, founded on the till, consistent with the existing bridge structures.

For the abutment widenings, and from a foundations perspective, either integral or semi-integral abutments could be adopted for the widening; however, the use of integral abutments would require modification of the existing abutment bearings, unless the widening is articulated from the existing structure.

The silty clay found at this site is compressible and would not be suitable to support the abutments directly. It would also not be feasible to support the abutments on perched footings in the embankment fill since the fill is underlain by the compressible clay deposit.

The use of deep foundations which would transfer the loads down to the underlying bedrock is required. Either driven steel H-pile foundations or cast-in-place concrete caissons could be considered for this purpose, although the former option is probably more cost-effective and has less risk associated with construction. In addition, since the abutment widenings will likely be structurally tied to the existing abutments, foundation systems that would have similar deformation characteristics to the existing structure would be preferred. Thus the use of driven steel H-pile foundations is recommended, from a geotechnical perspective, to support the abutment walls.

RSS walls are not a suitable option for the abutments at this site, as the anticipated primary and secondary settlements could result in the formation of gaps between the RSS wall facing panels. Further, some subexcavation of the existing embankment fill would be required in order to install the RSS wall strips and granular fill; it is expected that fairly extensive and expensive temporary excavation support measures would be required to ensure the stability of the existing embankment side slopes during this removal.

Summaries of the abutment and pier widening options are provided in Tables 2 and 3, along with the advantages, disadvantage, relative costs, and risks/consequences of each. Design guidelines for the feasible deep and shallow foundation options for the bridge abutments and piers are presented in the following sections. In summary, it is recommended that the abutments be supported on driven steel H-piles, founded on bedrock, while the piers be supported on footings on the glacial till.

5.3 Deep Foundations

As discussed above, deep foundations will be required for the abutment widenings. Deep foundations would also be feasible for the pier widenings. Guidelines for two deep foundations options, namely steel H-pile foundations and cast-in-place concrete caissons, are provided in this section.

5.3.1 Steel H-Pile Foundations

Steel H-piles driven to found on the limestone bedrock may be used for support of the abutment widenings. The following table assumes that the abutment widenings will be constructed with the same underside of pile cap elevation as the existing abutments. All elevations have thus been inferred from the General Plan drawing D-6839-1 by MTO for the existing structure and dated December 1970.

Location	Underside of Pile Cap Elevation (m)*	Approximate Pile Tip Elevation (m)*
East Abutment	69.6	60.1
West Abutment	69.6	60.3

5.3.1.1 Axial Geotechnical Resistance

For HP 310 x 110 piles driven to found on the limestone bedrock, a factored axial resistance at Ultimate Limit States (ULS) of 2,000 kN may be assumed for design. This value takes into account the structural capacity limitation of the pile, and potential difficulties that the pile may have seating into the bedrock surface that may be variable and inclined. The pile tips for vertical piles should have the flanges suitably reinforced while battered piles should be provided with rock points such as Titus ejector or equivalent to ensure penetration and adequate seating as per current MTO practice (Standard OPSD 3301.00 and OPSS 903.07.02.05).

A Serviceability Limit States (SLS) value is not provided because the limestone bedrock is considered to be an unyielding material. Under these conditions the SLS resistance (for 25 mm of settlement) is higher than the ULS value.

Pile installation should be in accordance with SP903S01. For this site, the piles will essentially be driven to practical refusal on the bedrock. The drawings should incorporate the appropriate note stating that the piles should be driven to bedrock.

Embankment fill materials which are placed in advance of pile driving should be of 75 mm minus sized material to avoid obstruction to driving of the piles.

5.3.1.2 Downdrag Load (Negative Skin Friction)

It should be noted that the widening of the existing approach embankments will result in some filling of the south shoulder area behind the east and west abutments and an increase in the effective stress level in the silty clay deposit which underlies this site. It is understood that the maximum height of fill, which will be beneath the new south edge-of-roadway, will be about 1.6 metres. That stress increase will lead to some compression of the clay deposit under the east abutment (as inferred from the results of Boreholes 04-106 and 05-201). The magnitude of the resulting consolidation settlements are discussed later in Section 5.7.2 of this report, but is estimated at about 50 millimetres. Additional settlement will occur due to longer term secondary compression (creep) and due to compression of the existing embankment fills themselves.

The consolidation settlement is time-dependent and will not completely occur during the construction period. That is, post-construction settlement of the clay deposit will take place. Since the piles will be end-bearing on bedrock, a small amount of settlement of the clay relative to the pile will result in the development of negative skin friction on the piles. Therefore, negative skin friction or downdrag loads will need to be taken into account during design of the piles supporting the abutments. The effects of negative skin friction or downdrag loads on the existing abutment piles should also be considered.

The magnitude of the downdrag load acting on the pile is a function of the adhesion (skin friction) that develops between the pile and the clay, and the surface area of the pile within the clay deposit. The unit negative skin friction acting on a unit area along a single pile can be calculated using the following equations:

For cohesionless soils (sand, gravel, rockfill, sandy silt)

$$f_{sn} = \beta\sigma_v'$$

where

f_{sn} is the unit negative skin friction (kN)
β is the shaft resistance factor = 0.47
σ_v' is the effective vertical (overburden) pressure (kPa)

For cohesive soils

$$q_n = \alpha\tau_u$$

where

q_n is the unit negative skin friction (kN)
α is the reduction coefficient ranging from 0.5 to 1.0
τ_u is the undrained shear strength (kPa)

For this site σ_v', can be calculated (approximately) for design purposes as:

$$\sigma_v' = \gamma' z$$

where

γ' is the buoyant unit weight of soil (assume 11 kN/m^3)
Z is the depth below final grade elevation (kPa)

For design purposes, the following are the values of τ_u and $\alpha\tau_u$ that can be used to calculate negative skin friction:

East Abutment

Soil Unit	τ_u	$\alpha\tau_u$
Stiff to very stiff weathered silty clay crust from Elev. 64.4 m to 63.1 m	80 kPa	40 kPa
Firm unweathered silty clay from Elev. 63.1 m to 61.5 m	30 kPa	25 kPa

West Abutment

Soil Unit	τ_u	$\alpha\tau_u$
Very stiff weathered silty clay crust from Elev. 64.9 m to 63.8 m	80 kPa	40 kPa

The total downdrag load is a function of the surface areas of the pile within the soil strata and the undrained shear strength mobilized from the top of the embedding layer down to the neutral point (Briaud, 1994). The load calculated in this manner is a nominal (unfactored) load. The structural engineer needs to multiply this load by a load factor of 1.25, as defined in Section 6.8.3 of the CHBDC, and include it as part of the load acting on the pile as described in the CHBDC.

Using the method described above, the estimated downdrag loads acting on a single pile at the abutment foundations are summarised in the following table. The loads given are the estimated nominal (unfactored) downdrag loads acting on HP 310 x 110 steel piles for the structure.

Location	**Nominal (unfactored) Downdrag Load**
East Abutment	500 kN
West Abutment	N/A*

* No downdrag was assigned to the west abutment since stresses imposed do not exceed preconsolidation pressure of the weathered crust.

For this assessment, the neutral plane was assumed to be at the underside of the silty clay deposit.

It should be noted that the structural engineer needs to review the piles within the existing abutment foundation structures to determine whether there is sufficient capacity to carry the downdrag loads. It is anticipated that the downdrag loads given above will affect the single vertical pile under the south wing wall of the east abutment and the most southerly inward battered (1 horizontal to 6 vertical) pile under the abutment wall.

Two options that could be used to eliminate or reduce the downdrag loads at the east embankment are as follows:

- The embankment widening could be pre-loaded (see Section 5.7.2) such that essentially all of the settlement occurs prior to installation of the piles. It should be noted that this option would not reduce the design downdrag loads on the existing piles, just the downdrag loads on the new piles.

- Consideration can be given to using lightweight expanded polystyrene fill to construct the embankment widening behind the east abutment. The lightweight fill would be keyed into the existing embankment fill and extended back from the abutment wall a distance of 15 m. The use of the lightweight fill would have the added benefit of reducing the lateral earth pressures on the new abutment wall and also eliminating downdrag loads on the existing abutment walls.

It should be noted that the guidelines given above in relation to the downdrag loads are not applicable to the pier foundations, only to the abutment foundations. No significant filling is proposed for the pier foundation areas.

5.3.1.3 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered steel H-piles. For vertical piles, the resistance to lateral loading will have to be derived from the soil in front of the piles. Where integral abutments are under consideration, there will also be a requirement for the piles to move sufficiently to accommodate the bridge deck deflections.

The resistance to lateral loading in front of the piles may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h, is based on the following equations:

For cohesionless soils

$$k_h = \frac{n_h z}{B}$$

where
n_h is the constant of subgrade reaction (MPa/m)
z is the depth (m)
B is the pile diameter/width (m)

For cohesive soils

$$k_h = \frac{67 s_u}{b}$$

where
k_h is the coefficient of horizontal subgrade reaction (kPa/m)
s_u is the undrained shear strength of the soil (kPa)
b is the pile width or diameter (m)

The following ranges for the value of n_h and s_u may be assumed in the structural analysis, using the stratigraphic sections provided on Drawing 2.

East Abutment

SOIL DEPOSIT	n_h	s_u
Compact sand and gravel fill from Elev. 69.6 to 64.4 m	16.0 MPa/m	--
Stiff to very stiff weathered silty clay crust from Elev. 64.4 to 63.1 m	--	80 kPa
Firm unweathered silty clay from Elevation 63.1 to 61.1 m	--	30 kPa
Compact sandy silt from Elev. 61.1 to 60.1	11.0 MPa/m	--

West Abutment

SOIL DEPOSIT	n_h	s_u
Compact sand and gravel fill from Elevation 69.6 to 64.9 m	16.0 MPa/m	--
Very stiff weathered silty clay crust from Elevation 64.9 to 63.5 m	--	80 kPa
Compact sandy silt till from Elev. 63.5 to 60.3 m	11.0 MPa/m	--

The above tables have been prepared based on the assumption that the pile cap base elevation for the abutment widenings will nearly match that of the existing abutments, at an estimated elevation of 69.6 m.

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. No reduction factor need be considered for pile spacing perpendicular to the direction of loading. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor as follows:

Pile Spacing in Direction of Loading (d = Pile Diameter)	Reduction Factor
8d	1.0
6d	0.7
4d	0.4
3d	0.25

5.3.2 Caisson Foundations

Cast-in-place concrete caissons could feasible for both the abutment or pier widenings. Caisson foundations could conceivably be founded on or socketted into the limestone bedrock.

The use of a temporary liner or casing will be required in order to advance the caissons with minimal loss of ground.

The limestone bedrock at the site is moderately strong. Rock sockets will have to be advanced by rock coring or churn drilling.

5.3.2.1 Axial Geotechnical Resistance

Caissons founded on the surface of the limestone bedrock, or socketted nominally (less than 1 m) into the bedrock, should be designed based on end-bearing resistance and a factored geotechnical resistance at ULS of 4 MPa. Serviceability Limit States resistances do not apply to caissons founded on the limestone bedrock since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

As described previously in relation to the design of piled foundations, downdrag forces should be considered in the design of caisson foundations for the east abutment due to consolidation of the silty clay deposit. The unfactored downdrag load acting on a single 1.5 m diameter caisson over its length is estimated to be 2,000 kN. The structural capacity of the caissons must be checked for the factored dead and downdrag loads in accordance with Section 6.8.4 of the *CHBDC*. The assumptions and methods used in assessing that downdrag force are the same as those described in Section 5.3.1.2 of this report. The guidelines provided in that section of the report for reducing or eliminating the downdrag forces are equally applicable to this foundation option.

5.3.2.2 Resistance to Lateral Loads

The resistance to lateral loading developed by the soils in front of the caissons, and the reductions due to group effects, may be determined as per Section 5.3.1.3.

5.3.3 Frost Protection

The pile caps should be provided with a minimum of 1.8 m of soil cover for frost protection.

5.4 Shallow Foundations

Shallow foundations may be considered for the support of the pier widenings. Shallow foundations are not feasible for the abutment widenings.

For the pier widenings, two shallow foundation options could be considered. The footings could be supported on the glacial till or alternatively on the underlying limestone bedrock.

As noted in Section 5.4.3, a minimum of 1.8 m of soil cover must be provided above the footing level to ensure adequate protection against frost penetration. Based on the borehole results, a founding level of Elev. 61.9 m (or lower)may be taken for the design of spread footings at the east pier and Elev. 63.7 m (or lower) for the west pier.

Variations in the thickness of the silty clay deposit that overlies the glacial till should be anticipated and provision should be made in the contract for extending the footing excavation deeper as may be needed to reach the till founding stratum.

For footings on bedrock, the borehole information indicates founding levels for the east and west piers of 60.2m and 59.3m, respectively.

For all footing construction, it should be noted that the above design founding elevations are provided based on the borehole data available. It must be confirmed during construction that the soils at the base of the excavation are consistent with those anticipated. Provision should be made in the Contract documents for subexcavation and replacement with mass concrete should softer zones be encountered at the design founding level.

5.4.1 Limits States Factored Geotechnical Resistance and Reaction

Spread footings placed on undisturbed till, at or below the design elevations given above, may be designed based on a factored geotechnical resistance at Ultimate Limit States (ULS) of 600 kPa. The geotechnical resistance at Serviceability Limit States (SLS) may be taken as 300 kPa.

Footings on the bedrock surface may be designed using a factored geotechnical resistance at Ultimate Limit States (ULS) of 2,000 kPa. Serviceability Limit States resistances do not apply to design of footings on the limestone bedrock since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

The geotechnical resistances provided herein are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with the *Canadian Highway Bridge Design Code (CHBDC).*

Due to the proximity of the existing and proposed piers, it is recommended that the piers be kept structurally separate. In addition, to reduce the risk of undermining the existing piers where the new piers are at a lower elevation, the new pier footings should be separated from the existing piers by a distance at least equal to the difference in founding level, but no less than 0.5 m.

5.4.2 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings and subsoils should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The coefficient of friction, tan δ, may be taken as 0.40 for cast-in-place concrete footings constructed on undisturbed, generally compact sandy silt till. For footings on bedrock, a coefficient of friction of 0.7 may be used. These represent unfactored values; in accordance with the *CHBDC*, a factor of 0.8 is to be applied in calculating the horizontal resistance.

5.4.3 Frost Protection

The footings should be provided with a minimum of 1.8 m of soil cover for frost protection.

5.5 Site Coefficient

For seismic design purposes, the Site Coefficient, S, for this site in accordance with Section 4.4.6 of the CHBDC may be taken as 1.2, consistent with Soil Profile Type II.

5.6 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment stems and any associated wing walls / retaining walls will depend on the type and method of placement of the backfill materials, on the nature of the soils behind the backfill, on the magnitude of surcharge including construction loadings, on the freedom of lateral movement of the structure, and on the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

The following recommendations are made concerning the design of the walls. It should be noted that these design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. This fill should be placed and compacted in accordance with MTO's Special Provision (SP) 105S10.
- Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost tapers should be in accordance with OPSD 3501.00 and 3504.00.

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with *CHBDC* Section 6.9.3 and Figure 6.9.3. Compaction equipment should be used in accordance with the requirements set out in Sections 501.06 and 501.07 of MTO's SP105S10. Other surcharge loadings should be accounted for in the design, as required.

- The granular fill may be placed either in a zone with width equal to at least 1.8 m behind the back of the wall stem (Case i in Figure C6.9.1(l) of the *Commentary to the CHBDC*) or within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (Case ii in Figure C6.9.1(l) of the *Commentary to the CHBDC*).

- For Case i, the pressures are based on the proposed embankment fill materials and the following parameters (unfactored) may be used assuming the use of Select Subgrade material:

Soil unit weight:	20 kN/m^3
Coefficients of static lateral earth pressure:	
Active, K_a	0.35
At rest, K_o	0.50

- For Case ii, the pressures are based on the granular fill as placed and the following parameters (unfactored) may be assumed:

	Granular 'A'	**Granular 'B' Type II**
Soil unit weight:	22 kN/m^3	21 kN/m^3
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.27
At rest, K_o	0.43	0.43

- If the wall support and superstructure allow lateral yielding of the stem, active earth pressures may be used in the geotechnical design of the structure. If the abutment support does not allow lateral yielding, at-rest earth pressures should be assumed for geotechnical design.

- Seismic loading will result in increased lateral earth pressures acting on the abutment stem and retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to the National Building Code of Canada, this site is located in Seismic Zone 4. The site-specific zonal acceleration ratio for Ottawa is 0.2 which is also applicable for Casselman. Based on experience, for the subsurface conditions at this site, a 15 per cent amplification of the ground motion will occur, resulting in an increase in the ground surface acceleration from 0.2g to 0.23g. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of A = 0.23.

- In accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its Commentary, for structures which do not allow lateral yielding, the horizontal seismic coefficient, k_h, used in the calculation of the seismic active pressure coefficient, is taken as 1.5 times the zonal acceleration ratio (i.e. k_h = 0. 34). For structures which allow lateral yielding, k_h is taken as 0.5 times the zonal acceleration ratio (i.e. k_h = 0.12). The seismic active earth pressure coefficient is also dependent on the vertical component of the earthquake acceleration, k_v. Three discrete values of vertical acceleration are typically selected for analysis, corresponding to $k_v = +2/3\ k_h$, $k_v = 0$, and $k_v = -2/3\ k_h$.

- The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case I and Case ii) may be used in design; these coefficients reflect the maximum K_{AE} obtained using the k_h and three values of k_v as described above. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat.

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case i	Case ii	
		Granular A	Granular B Type II
Yielding wall	0.39	0.33	0.33
Non-yielding wall	0.82	0.68	0.68

Note: These CHBDC seismic K_{AE} values include the effect of wall friction ($\delta=\phi'/2$) and are less than the static values of K_a and K_o reported above for the very low zonal acceleration ratio of this site.

- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A (mm), where A is the design zonal acceleration ratio of 0.23. This corresponds to displacements of up to 58 mm at this site.

- The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

$$\sigma_h(d) = K\,\gamma\,d + (K_{AE} - K)\,\gamma\,(H-d)$$

where
$\sigma_h(d)$ is the lateral earth pressure at depth, d, (kPa)
K is either the static active earth pressure coefficient (K_a) or the static at rest earth pressure coefficient (K_o);
K_{AE} is the seismic active earth pressure coefficient;
γ is the unit weight of the backfill soil (kN/m^3), as given previously;
d is the depth below the top of the wall (m); and
H is the total height of the wall (m).

5.7 Approach Embankment Design and Construction

As described previously, a separate foundation investigation has been carried out for the design of the overall embankment widening (Golder report number 04-1120-013-7000). Adjacent to the bridge, the widening of the existing approach embankments will require filling of up to 1.6 m over the existing embankment fill, at the point of maximum filling beneath the new edge-of-roadway

Based on the borehole results, the embankment subgrade soils consist of a firm to very stiff deposit of sensitive marine clay overlying glacial till.

5.7.1 Approach Embankment Stability

Stability analyses were performed on the critical sections of the proposed embankment widening. For this report, critical sections are assumed to correspond to the area where filling are greatest.

Limit equilibrium slope stability analyses were performed using the commercially available program SLOPE/W (version 5.17), produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the factor of safety of numerous potential failure surfaces was computed in order to establish the minimum factor of safety. The factor of safety is defined as the ratio of the forces tending to resist the failure to the driving forces tending to cause failure. A target factor of safety of 1.3 is normally used for the design of embankment slopes under static conditions. This factor of safety is considered adequate for the embankments at this site considering the design requirements and the field data available. The stability analyses were performed to check that the target minimum factor of safety was achieved for the proposed embankment widening.

For cohesive soils, total stress parameters were employed in the analyses. The total stress parameters (i.e. average mobilized undrained shear strength – s_u) for the cohesive soils were assessed based on the results of field vane testing.

Soil Deposit	Unit Weight (kN/m^3)	Effective Friction Angle	Undrained Shear Strength (kPa)
Embankment Fill	20	30°	--
Sand	18	30	--
Weathered Silty Clay Crust	17	--	80
Unweathered Grey Silty Clay	17	--	30
Glacial Till	21	32.5	--

The stability for the proposed embankment fill widening was assessed based on precedent experience in similar soil conditions. The results of the static and seismic stability analyses for the approach embankments are shown on Figures 4 to 11. The factor of safety against instability under static conditions would be in excess of 1.3 assuming 2 to 1 side slopes and that all of the surface and near surface organic layers have be removed prior to construction. Stability analyses undertaken using the above parameters and the assumed filling geometry indicate a factor of safety against instability under seismic condition in excess of 1.1, which is considered acceptable.

Based on the elevation of the ground water table, and assumed geometry of the slope and the type of fill being used, the liquefaction potential of the soils under seismic loading is considered low but it is possible that surficial sloughing could occur in localized areas.

Where the approach embankment height is equal to or greater than 8 m, a mid-height berm at least 2 m in width is required for maintenance purposes. To reduce surface water erosion on the embankment side slopes, placement of topsoil and seeding or pegged sod is recommended.

5.7.2 Approach Embankment Settlement

Settlement of the approach embankments will occur due to compression of the new embankment fill itself, as well as consolidation of the underlying silty clay deposit. Provided that the new embankment fill material consists of select subgrade material or clean earth fill, the settlement of the embankment fill itself is expected to be less than about 25 mm. The use of granular fill for the new embankment construction would reduce this magnitude, since the majority of settlement of granular fills will occur during construction.

At and immediately behind the *west* abutment, where no unweathered silty clay is present, the coefficient of consolidation of the weathered silty clay crust, typically being a fissured soil and being stressed within its re-compression limits, is relatively high. Therefore the subgrade settlements resulting from compression of the weathered silty clay crust, which should be modest in magnitude, would be expected to occur quite rapidly, likely entirely during embankment construction, such that the post-construction settlements of the embankment surface would not be expected to noticeably exceed the compression of the embankment fill itself.

However, at and behind the *east* abutment, the available information indicates that up to about 1.7 m of compressible un-weathered silty clay may be present. The critical location in terms of the embankment settlement is considered to be the edge of the new embankment (i.e., essentially the edge of the new ramp lane) since this is the location with the greatest filling (about 1.6 to 1.7 m thickness) and greatest stress increase on the underlying subgrade.

For the east abutment, the existing effective stress profile within the silty clay beneath the future embankment edge, and the resulting stress increase from the widening, were calculated using a closed form solution based on elastic stress distribution theory for a 2 dimensional embankment loading. The calculated effective stress level in the unweathered silty clay deposit generally exceeds the preconsolidation pressure of this deposit as indicated by the laboratory oedometer consolidation testing; the silty clay beneath the embankment slope has been locally pre-loaded above its original preconsolidation pressure. Therefore, for the purposes of the settlement analyses for the edge of the new embankment, the preconsolidation pressure of the deposit was taken as the existing effective stress level (i.e., the clay is normally consolidated). Therefore all of the calculated settlements occur within the 'virgin' compression range, with no contribution from re-compression of the deposit. The coefficient of consolidation of the deposit is also significantly lower at stress levels exceeding the preconsolidation pressure and therefore the calculated settlements will take longer to occur and should largely be manifested after construction of the embankment.

The calculated settlement resulting from primary consolidation of the deposit beneath the east abutment is 50 mm.

It should be noted that, due to the limited thickness of the clay deposit the time required for these settlements to occur is fairly limited. It is expected that only about 3 months time will be required to achieve more than 90 percent of this settlement. It should be noted that settlements of the embankment fill itself would be in addition to this value. Further, in the longer term, these settlements would increase due to secondary compression (creep) of the deposit. It is expected that over a period of 10 years following construction (the likely approximate time until the next repaving) secondary compression could increase these settlements by about 25 percent.

Considering that the structure will be pile supported and therefore un-settling, it is considered that this calculated post-construction embankment settlement of 50 mm is somewhat in excess of what is generally acceptable. Distortion of the roadway could be excessive and could impact on the serviceability and/or safety of the roadway.

The foundation investigation report for the widening of the approach embankments (Golder report number 04-1120-013-7000) discusses several options for reducing the post-construction settlements, which are also applicable to the embankment area behind the east abutment. The most-feasible of these options which include:

1. Allowing the embankment to settle,
2. Pre-loading
3. Light weight fill

These options are discussed below.

Option 1 – Allow Embankments to Settle.

The embankments could be allowed to settle, with the expectation that it would be necessary to mill and re-pave the new lanes is the near future (say, 1 year after construction) to return the lane to an acceptable profile and cross fall. It should be noted however that, for the period prior to re-paving, the settlements could have a detrimental impact on the serviceability and safety of the roadway.

Option 2 – Pre-Loading.

The embankment could be pre-loaded and allowed to settle prior to paving. Although a pre-load time of 3 months should be sufficient for the area behind the east abutment, for the overall ramp/embankment a pre-load time of about 6 months is expected to be necessary to allow essentially all of the primary consolidation settlements to occur. However these times are estimates only, and the actual pre-load time would need to be confirmed by monitoring of the settlements.

For this option, an instrumentation monitoring program, with plans, details, and specifications is required.

Option 3 – Lightweight Fill

Lightweight EPS fill could be used for the embankment construction in the area behind the east abutment, and thereby reduce the stress increase on the compressible clay deposit to a level such that the embankment settlements will be within acceptable tolerances. As a preliminary guideline, the full embankment widening construction (except for the pavement structure) for a distance of 15 metres back from the abutment should be constructed with light weight fill. A more detailed assessment of the limits and thickness of EPS fill can be provided, if this option is selected.

It should also noted that suitable frost tapers would need to be provided at the ends of the EPS fill treatment to avoid differential frost heaving of the overlying pavement surface.

Since all of the settlements are to result from compression in the 'virgin' compression zone, it is not considered feasible to consider the use of other heavier light weight fills (e.g., slag) for this application; the magnitude of the settlement will be directly proportional to the magnitude of the stress increase, which must therefore be minimized.

All three of the above options for reducing the embankment settlements are considered to be technically feasible.

A summary comparison of the advantages, disadvantages, relative costs, and risks associated with the above options, from a geotechnical perspective, is presented in Table 4 following the text of this report.

Based on the understanding that the construction schedule for this project would permit the embankment widening to be constructed up to about one year before the ramp needs to be in-service, it is considered that Option 2 (preloading) is preferred, in that it has probably the lowest cost and potentially little or no impact on the overall construction schedule.

5.7.3 Subgrade Preparation and Approach Embankment Construction

Any topsoil, organic matter and softened / loosened soils should be stripped from within the limits of the widening. All subgrade soils should be proof-rolled prior to fill placement.

Embankment fill should be placed in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 per cent of the material's Standard Proctor maximum dry density.

The final lift prior to placement of the granular subbase and base courses should be compacted to 100 per cent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified personnel during placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

The new embankment fills should be benched into the existing embankment in accordance with OPSD 208.010.

5.8 Design and Construction Considerations

5.8.1 Excavation

It is assumed that the abutment widenings will be constructed with the same underside of pile cap level as the existing abutments. The excavations will extend through the existing embankment fills for the abutment widening. Excavation for the pier widenings will extend through the weathered silty clay, and through the glacial till if footings on bedrock is the selected foundation option.

Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities. These soils are classified as Type 3 soils according to the OHSA and therefore excavations for the east abutment, west abutment and the east pier should be made with side slope no steeper than 1 horizontal to 1 vertical.

The excavation for the west bridge pier will be up at least about 3 metres deep and will be located less than 2 metres from the rail line. Railtrack protection (temporary shoring) will therefore be required for this excavation. Roadway protection will also be required for the excavations needed to construct the abutment widening.

For the east pier widening, the nearest rail will be about 5 metres from the excavation. For the case of this pier being supported on a footing on the glacial till, where the excavation will be about 3.5 m deep, stability analyses were undertaken for this excavation, considering also the loading from the rail traffic, and these analyses are shown on Figures 12 and 13. The surcharge loading due to the rail traffic is estimated from the Cooper E90 locomotive axle loading. The loading considered is based on a line load of 40,820 kg (90,000 lbs) per axle, for 4 axles paced 1.5 m apart. For analysis purposes, it is assumed that the axle loads are spread uniformly across 2.5 m wide rail ties and are treated as a uniform surcharge of 65 kPa. Assuming 1 to 1 side sloped open cut excavations, the factor of safety against instability under static condition is in excess of 1.3, which is acceptable. As a result, the use of temporary shoring for these excavations is not anticipated, unless the excavation will be made deeper than about 4 metres.

5.8.2 Temporary Shoring

As discussed above, railtrack protection will be required for the construction of the west pier and roadway protection will be required for construction of the bridge abutment widenings.

The temporary excavation support should be in accordance with MTO Special Provision 539S01. The temporary system for the roadway protection at the abutment widening should be designed to Performance Level 2 as defined in SP 539S01. For the railtrack protection at the pier widenings, the requirements for the excavation support and the performance level should be determined based on consultation with the railway, in view of the tolerance of the railway to accept movement.

It is understood that the design of the shoring will be entirely the responsibility of the contractor. To the expected depths of excavation, it is not expected that basal heaving or basal instability will be a concern. The shoring will have to be designed to resist lateral earth pressures that are controlled by the flexibility of the shoring and its method of support.

5.8.3 Groundwater and Surface Water Control

Given that the abutment pile cap elevation is above the measured groundwater level, it is anticipated that the excavations at the east and west abutments will not experience significant groundwater inflow. However, a modest amount of groundwater flow is expected for the east and west pier excavations and it is anticipated that adequate groundwater control can be affected through the use of pumping from properly filtered sumps in the excavation. As indicated earlier, deeper subexcavation to reach bedrock could result is the need for more extensive groundwater control systems.

Surficial drainage may be also required around the perimeter of the west pier excavation due to the cutting of the existing drainage ditch.

5.8.4 Obstructions

Although not encountered during field investigation, it is possible that cobbles and boulders may be present at the abutment area and pre-augering through the earth fill may be required prior to installation of piles.

The presence of such obstructions will affect the installation of the driven steel H-piles. Provision will have to be made in the Contract Documents to ensure that the Contractor is equipped to handle such obstructions.

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PROVINCE OF ONTARIO

MSS:FJH:cr:al
n:\active\2004\1120\geotechnical\04-1120-013 mh hwy 417 limoges to casselman\phase 5000 foundations\04-1120-013 rpt-007 05july15 final report cnr overpass.doc

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LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO	Drive open
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open Sampler for a distance of 300 mm (12 in.)

Dynamic Penetration Resistance; N_d:
The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive Uncased a 50 mm (2 in.) diameter, 60^0 cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

Peizo-Cone Penetration Test (CPT):
An electronic cone penetrometer with a 60^0 conical tip and a projected end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded Electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	**N** Blows/300 mm Or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	**$C_{u2}S_u$** Kpa	Psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	Over 200	Over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limited
w_l	liquid limit
C	consolidaiton (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	modified Proctor compaction test
SPC	standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane test (LV-laboratory vane test)
γ	unit weight

Note:
1. Tests which are anisotropically consolidated prior shear are shown as CAD, CAU.

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	= 3.1416
ln x, natural logarithm of x	
$\log_{10}$ x or log x, logarithm of x to base 10	
g	Acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\sigma'$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma''$-u)
σ'_{vo}	initial effective overburden stress
$\sigma_1\sigma_2\sigma_3$	principal stresses (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1+\sigma_2+\sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma'=\gamma-\gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = p_s/p_w$) formerly (G_s)
e	void ratio
n	porosity
S	degree of saturation

* Density symbol is p. Unit weight symbol is γ where γ=pg(i.e. mass density x acceleration due to gravity)

(a) Index Properties (cont'd.)

w	water content
w_l	liquid limit
w_p	plastic limit
I_p	plasticity Index=(w_l-w_p)
w_s	shrinkage limit
I_L	liquidity index=(w-w_p)/I_p
I_c	consistency index=(w_l-w)/I_p
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index-(e_{max}-e)/(e_{max}-e_{min}) (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (overconsolidated range)
C_s	swelling index
C_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	Overconsolidation ratio=σ'_p/σ'_{vo}

(d) Shear Strength

$\tau_p\tau_r$	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction=tan δ
c'	effective cohesion
c_u, S_u	undrained shear strength (ϕ=0 analysis)
p	mean total stress $(\sigma_1+\sigma_3)/2$
p'	mean effective stress $(\sigma'_1+\sigma'_3)/2$
q	$(\sigma_1-\sigma_3)/2$ or $(\sigma'_1-\sigma_3)/2$
q_u	compressive strength $(\sigma_1-\sigma_3)$
S_t	sensitivity

Notes: 1. τ=c'σ' tan □'

2. Shear strength=(Compressive strength)/2

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering

Faintly Weathered: weathering limited to the surface of major discontinuities.

Slightly weathered: penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

Moderately weathered: weathering extends throughout the rock mass but the rock material is not friable

Highly weathered: weathering extends throughout rock mass and the rock material is partly friable.

Completely weathered: rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved.

BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	>2 m
Thickly bedded	0.6 m to 2m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	<6 mm

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	>3 m
Wide	1 – 3 m
Moderately close	0.3 – 1 m
Close	50 – 300 mm
Very close	<50 mm

GRAIN SIZE

Term	Size*
Very Coarse Grained	>60 mm
Coarse Grained	2 – 60 mm
Medium Grained	60 microns - 2mm
Fine Grained	2 – 60 microns
Very Fine Grained	<2 microns

Note: *Grains >60 microns diameter are visible to the naked eye.

CORE CONDITION

Total Core Recovery

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varies from 0% for completely broken core 100% for core in solid sticks.

DISCONTINUITY DATA

Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

Dip with Respect to (W.R.T.) Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90^0 angle is horizontal.

Description and Notes

An abbreviated description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature information concerning the nature of fracture surfaces and infillings are also noted.

Abbreviations

B –	Bedding	Ca-	Calcite
FO-	Foliation/Schistosity	P-	Polished
CL -	Cleavage	S-	Slickensided
SH -	Shear Plane/Zone	SM-	Smooth
VN-	Vein	R-	Ridged/Rough
F -	Fault	ST-	Stepped
CO-	Contact	PL-	Planar
J -	Joint	FL-	Flexured
FR-	Fracture	UE-	Uneven
MF -	Mechanical	W-	Wavy
A-	Angular	C-	Curved
BP-	Bedding Plane	H-	Hackly
BL-	Blast Induced	SL-	Sludge Coated
II	Parallel To	TCA-	To Core Axis
	Perpendicular To	STR-	Stress Induced

Golder Associates

RECORD OF BOREHOLE No 04-101

METRIC

PROJECT 04-1120-013 Phase 5000

W.P. 258-98-00

LOCATION N 5019200.2 ;E 417004.2

ORIGINATED BY D.J.S.

DIST ______ HWY 417

BOREHOLE TYPE Portable Drill

COMPILED BY M.I.C.

DATUM Geodetic

DATE May 11, 2004

CHECKED BY M.I.C.

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)
66.0	GROUND SURFACE					
0.0	Sand and gravel, trace wood and metal (FILL) Brown					
64.9 / 1.1	CONCRETE	1	BQ RC	DD	65	
		2	BQ RC	DD		
		3	BQ RC	DD	64	
		4	BQ RC	DD		
62.8					63	
62.5	Fine to medium sand, trace gravel (FILL) Loose Brown Wet	5	SS	4		~22
62.2 / 3.8	Sandy SILT, some gravel and clay with cobbles (TILL) Dense Grey Wet	6	BQ RC	DD		
	End of Borehole					

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25, 50, 75; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

$O^{3\%}$ STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 04-102

METRIC

PROJECT 04-1120-013 Phase 5000
W.P. 258-98-00
LOCATION N 5019204.6 ;E 417029.6
ORIGINATED BY D.J.S.
DIST HWY 417
BOREHOLE TYPE Portable Drill
COMPILED BY M.I.C.
DATUM Geodetic
DATE May 10-11, 2004
CHECKED BY M.I.C.

ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
65.8	GROUND SURFACE										
0.0	Sand and gravel, some silt (FILL) Very loose to loose Brown Moist to Wet						65				
			1	SS	6		64				
			2	SS	2				o (approx. 11)		
							63				
62.3			3	SS	1						
3.5	CONCRETE		4	BQ RC	DD		62				
			5	BQ RC	DD						
61.2 / 4.6	Sandy SILT, some gravel and clay, occasional boulder (TILL) Grey		6	BQ RC	DD		61				
			7	BQ RC	DD						
60.2 / 5.7	LIMESTONE BEDROCK Medium bedded Medium strong Grey Fresh						60				
59.5 / 6.3	For bedrock coring details refer to Record of Drillhole 04-102 End of Borehole										

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75

$+^3, \times^3$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

PROJECT: 04-1120-013 Phase 5000

RECORD OF DRILLHOLE: 04-102

SHEET 1 OF 1

LOCATION: N 5019204.6 ; E 417029.6

DRILLING DATE: 5/11/2004

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Portable Drill

DRILLING CONTRACTOR: Marathon Drilling

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED, BC-BROKEN CORE, MB-MECH. BREAK, B-BEDDING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Borehole continued from previous page		60.13												
6	Rotary Drill, BQ Core	LIMESTONE BEDROCK Medium bedded Medium strong Fresh Grey		5.67	1											
				59.46 6.34												
		End of Drillhole														

Depth scale: 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20

DEPTH SCALE
1 : 75

Golder Associates

LOGGED: D.J.S.
CHECKED: M.I.C.

RECORD OF BOREHOLE No 04-103

METRIC

PROJECT 04-1120-013 Phase 5000			
W.P. 258-98-00	LOCATION N 5019191.0 ;E 417016.7		ORIGINATED BY D.J.S.
DIST	HWY 417	BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger	COMPILED BY M.I.C.
DATUM Geodetic		DATE May 6, 2004	CHECKED BY M.I.C.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
65.9	GROUND SURFACE				
0.0	Silty clay, trace gravel and metal (FILL) Grey brown and dark brown				
65.5 / 0.4	Silty CLAY (Weathered Crust) Very stiff to stiff Grey brown Wet	1	SS	4	
		2	SS	1	
64.1	Sandy SILT and clayey SILT, layered Very stiff Grey brown Wet				
63.7 / 2.2	Sandy SILT, some gravel and clay with cobbles and boulders (TILL) Compact to very dense Brown to grey Wet	3	SS	11	
		4	SS	13	29 33 31 7
		5	SS	37	
		6	NQRC	DD	
		7	SS	57	
		8	SS	40	
	Dark grey fine to medium sand layer from 5.8 m to 6.1 m depth	9	SS	20	
59.3 / 6.6	LIMESTONE BEDROCK Thickly bedded Medium strong Fresh Grey	10	NQRC	DD	
	For bedrock coring details refer to Record of Drillhole 04-103	11	NQRC	DD	
		12	NQRC	DD	
55.9 / 10.1	End of Borehole				

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (65, 64, 63, 62, 61, 60, 59, 58, 57, 56); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75; UNIT WEIGHT γ kN/m³

$+^3, \times^3$: Numbers refer to Sensitivity ○$^{3\%}$ STRAIN AT FAILURE

PROJECT: 04-1120-013 Phase 5000

RECORD OF DRILLHOLE: 04-103

SHEET 1 OF 1

LOCATION: N 5019191.0 ; E 417016.7

DRILLING DATE: 5/6/2004

DATUM: Geodetic

DRILL RIG: CME 55

INCLINATION: -90° AZIMUTH: —

DRILLING CONTRACTOR: Marathon Drilling

FR-FRACTURE, CL-CLEAVAGE, SH-SHEAR, VN-VEIN, F-FAULT, J-JOINT, P-POLISHED, S-SLICKENSIDED, SM-SMOOTH, R-ROUGH, ST-STEPPED, PL-PLANAR, FL-FLEXURED, UE-UNEVEN, W-WAVY, C-CURVED, BC-BROKEN CORE, MB-MECH. BREAK, B-BEDDING

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	FLUSH COLOUR % RETURN	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Borehole continued from previous page		59.29												
7	Rotary Drill HQ Core	LIMESTONE BEDROCK Thickly bedded Medium strong Fresh Grey		6.61	1											
8					2											
9					3											
10		End of Drillhole		55.84 10.06												

DEPTH SCALE 1 : 75

Golder Associates

LOGGED: D.J.S.

CHECKED: M.I.C.

DRILLHOLE 04-1120-013-5000ROCKMTO.GPJ GLDR_CAN.GDT 11/8/04

Golder Associates

RECORD OF BOREHOLE No 04-104

METRIC

PROJECT 04-1120-013 Phase 5000

W.P. 258-98-00

LOCATION N 5019197.9 ;E 417038.4

ORIGINATED BY J.S.

DIST ___ HWY 417

BOREHOLE TYPE Portable Drill

COMPILED BY M.I.C.

DATUM Geodetic

DATE May 12, 2004

CHECKED BY M.I.C.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES
65.2	GROUND SURFACE			
0.0	Silty sand with organic matter (FILL), Very loose, Brown, Wet	1	SS	2
64.8 / 0.5	Sandy SILT, some clay, Very loose, Grey brown, Wet	2	SS	3
64.1 / 1.1	Silty CLAY (Weathered Crust), Stiff, Grey brown, Wet	3	SS	2
		4	SS	2
62.4 / 2.8	Sandy SILT and Clayey SILT, Loose, Grey brown, Wet	5	SS	3
		6	SS	12
61.9 / 3.4	Sandy SILT, some gravel and clay with cobbles (TILL), Compact, Grey, Wet	7	SS	13
		8	NQ RC	DD
60.2 / 5.0	LIMESTONE BEDROCK, Very thinly bedded to medium bedded, Medium strong, Fresh, Grey	9	NQ RC	DD
		10	NQ RC	DD
		11	NQ RC	DD
58.1 / 7.2	LIMESTONE BEDROCK, Thinly bedded to medium bedded, Medium strong, Fresh, Grey	12	NQ RC	DD
		13	NQ RC	DD
56.9 / 8.3	For bedrock coring details refer to Record of Drillhole 04-104. End of Borehole			

Note:
Water level in standpipe at 1.1 m depth below ground surface on May 20, 2004

Other column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (65 to 57); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa — O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L — WATER CONTENT (%) (25, 50, 75); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

$O^{3\%}$ STRAIN AT FAILURE

MISS_MTO 04-1120-013-5000.GPJ ON_MOT.GDT 11/8/04

PROJECT: 04-1120-013 Phase 5000

RECORD OF DRILLHOLE: 04-104

LOCATION: N 5019197.9 ; E 417038.4

DRILLING DATE: 5/12/2004

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG: Portable Drill

DRILLING CONTRACTOR: Marathon Drilling

FR-FRACTURE F-FAULT SM-SMOOTH FL-FLEXURED BC-BROKEN CORE
CL-CLEAVAGE J-JOINT R-ROUGH UE-UNEVEN MB-MECH. BREAK
SH-SHEAR P-POLISHED ST-STEPPED W-WAVY B-BEDDING
VN-VEIN S-SLICKENSIDED PL-PLANAR C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	COLOUR % RETURN FLUSH	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Borehole continued from previous page		60.20												
5	Rotary Drill BQ Core	LIMESTONE BEDROCK Very thinly bedded to medium bedded Medium strong Fresh Grey		5.00	2											
6					3											
					4											
7		LIMESTONE BEDROCK Thinly bedded to medium bedded Medium strong Fresh Grey		58.04 7.16	5											
8					6											
		End of Drillhole		56.88 8.32												

9 10 11 12 13 14 15 16 17 18 19 20

DRILLHOLE 04-1120-013-5000ROCKMTO.GPJ GLDR_CAN.GDT 11/8/04

DEPTH SCALE
1 : 75

Golder Associates

LOGGED: J.S.

CHECKED: M.I.C.

Golder Associates

RECORD OF BOREHOLE No 04-105

METRIC

PROJECT 04-1120-013 Phase 5000
W.P. 258-98-00
LOCATION N 5019175.5 ;E 417014.4
ORIGINATED BY J.S.
DIST
HWY 417
BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
COMPILED BY M.I.C.
DATUM Geodetic
DATE May 7, 2004
CHECKED BY M.I.C.

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	NATURAL MOISTURE CONTENT	GR	SA	SI	CL
65.6	GROUND SURFACE								
0.0	TOPSOIL								
0.2	Silty fine sand (FILL) Brown								
64.9 / 0.7	Silty CLAY, occasional fine sand seam (Weathered Crust) Very stiff Grey to brown Moist to wet	1	SS	7					
63.8	Sandy SILT with occasional clayey silt layer Grey to brown Moist to wet	2	SS	3					
63.5 / 2.1	Sandy SILT, some gravel and clay, with cobbles (TILL) Loose to compact Brown to grey Wet	3	SS	8					
		4	SS	8	o	16	34	43	7
	Fine to coarse sand seam at 5.2 m depth	5	SS	17					
		6	SS	6					
60.3 / 5.2	LIMESTONE BEDROCK Thickly bedded Medium strong Fresh Grey	7	NQRC	DD					
		8	NQRC	DD					
	For bedrock coring details refer to Record of Drillhole 04-105	9	NQRC	DD					
57.2 / 8.4	End of Borehole								

Note:
Water level in standpipe at 1.2 m depth below ground surface on May 20, 2004

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (65, 64, 63, 62, 61, 60, 59, 58); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

+ 3, × 3: Numbers refer to Sensitivity   O 3% STRAIN AT FAILURE

RECORD OF DRILLHOLE: 04-105

PROJECT: 04-1120-013 Phase 5000
SHEET 1 OF 1

LOCATION: N 5019175.5 ; E 417014.4
DRILLING DATE: 5/7/2004
DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---
DRILL RIG: CME 55
DRILLING CONTRACTOR: Marathon Drilling

FR-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE
CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH. BREAK
SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING
VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	COLOUR % RETURN / FLUSH	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Borehole continued from previous page		60.36												
	Rotary Drill, NW / NQ Core	LIMESTONE BEDROCK Thickly bedded Medium strong Fresh grey		5.24	1											
6–7					2											
8				57.22 8.38	3											
		End of Drillhole														

DEPTH SCALE
1 : 75

Golder Associates

LOGGED: J.S.
CHECKED: M.I.C.

DRILLHOLE 04-1120-013-5000ROCKMTO.GPJ GLDR_CAN.GDT 11/8/04

Golder Associates

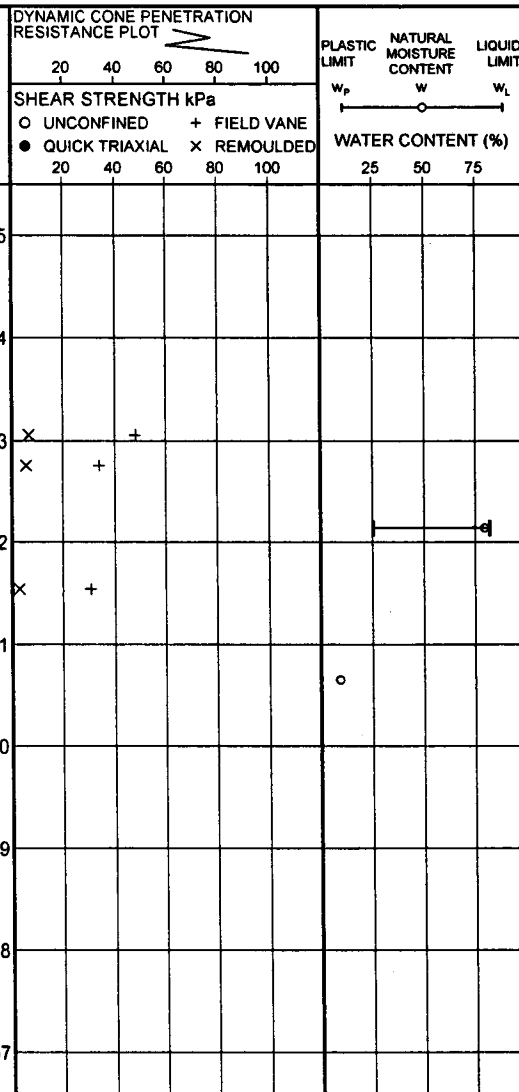
RECORD OF BOREHOLE No 04-106

METRIC

PROJECT 04-1120-013 Phase 5000			
W.P. 258-98-00	LOCATION N 5019181.7 ;E 417072.5	ORIGINATED BY J.S.
DIST ___ HWY 417	BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger	COMPILED BY M.I.C.
DATUM Geodetic	DATE May 10, 2004	CHECKED BY M.I.C.

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
65.5	GROUND SURFACE					
0.0	Limestone rockfill with topsoil (FILL) Grey				65	
64.9 / 0.6	Sandy SILT Light brown Moist					
64.4 / 1.1	Silty CLAY, occasional fine sand seam (Weathered Crust) Very stiff to stiff Grey brown Wet	1	SS	7		
		2	SS	2	64	
63.1 / 2.4	Silty CLAY Firm Grey Wet				63	
		3	SS	WH	62	0 2 23 75
61.5	Sandy SILT and Clayey SILT, layered Loose Grey Wet	4	SS	8		
61.1 / 4.4	Sandy SILT, some gravel, trace clay (TILL) Loose to compact Grey Wet	5	SS	10	61	50 24 20 6
60.1 / 5.4	LIMESTONE BEDROCK Thin to medium bedded Medium strong Fresh Grey	6	NQRC	DD	60	
		7	NQRC	DD	59	
58.7 / 6.8	LIMESTONE BEDROCK Thickly bedded Medium strong Fresh Grey For bedrock coring details refer to Record of Drillhole 04-106	8	NQRC	DD	58, 57	
56.6 / 8.9	End of Borehole					

Note:
Water level in standpipe
at 1.2 m depth below ground surface
on May 20, 2004



+ 3, × 3: Numbers refer to Sensitivity   O 3% STRAIN AT FAILURE

RECORD OF DRILLHOLE: 04-106

PROJECT: 04-1120-013 Phase 5000

LOCATION: N 5019181.7 ; E 417072.5

INCLINATION: -90° AZIMUTH: —

DRILLING DATE: 5/10/2004

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

DATUM: Geodetic

FR-FRACTURE, F-FAULT, SM-SMOOTH, FL-FLEXURED, BC-BROKEN CORE, CL-CLEAVAGE, J-JOINT, R-ROUGH, UE-UNEVEN, MB-MECH. BREAK, SH-SHEAR, P-POLISHED, ST-STEPPED, W-WAVY, B-BEDDING, VN-VEIN, S-SLICKENSIDED, PL-PLANAR, C-CURVED

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	COLOUR % RETURN	FLUSH	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	HYDRAULIC CONDUCTIVITY k, cm/sec	DIAMETRAL POINT LOAD INDEX (MPa)	NOTES WATER LEVELS INSTRUMENTATION
		Borehole continued from previous page		60.11													
	Rotary Drill HQ Core	LIMESTONE BEDROCK Thin to medium bedded Medium strong Grey		5.39	1												
6					2												
				58.70													
7		LIMESTONE BEDROCK Thickly bedded Medium strong Fresh Grey		6.80													
8					3												
				56.57													
9		End of Drillhole		8.93													
10																	
11																	
12																	
13																	
14																	
15																	
16																	
17																	
18																	
19																	
20																	

DEPTH SCALE

1 : 75

LOGGED: J.S.

CHECKED: M.I.C.

Golder Associates

RECORD OF BOREHOLE No 04-107

METRIC

PROJECT 04-1120-013 Phase 5000
W.P. 258-98-00
LOCATION N 5019168.5 ;E 416964.8
ORIGINATED BY J.S.
DIST
HWY 417
BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
COMPILED BY M.I.C.
DATUM Geodetic
DATE May 11, 2004
CHECKED BY M.I.C.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	NATURAL MOISTURE CONTENT
65.8	GROUND SURFACE					
0.0 / 65.2	Sandy silt (FILL) Loose Brown to grey					
	TOPSOIL				65	
64.7 / 1.1	Sandy SILT, trace clay Loose Brown to grey Moist	1	SS	7		~45
64.3 / 1.5	Silty CLAY (Weathered Crust) Grey to brown					
	Sandy SILT, some gravel, trace clay, with cobbles and boulder (TILL) Compact to very dense Grey Wet	2	SS	16	64	
		3	SS	10	63	
		4	SS	14		
		5	SS	34	62	
		6	SS	13	61	
59.9		7	SS	60	60	
5.9	End of Borehole Auger Refusal					

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED, 20 40 60 80 100; PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 25 50 75; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

+3, ×3: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

MISS_MTO 04-1120-013-5000.GPJ ON_MOT.GDT 11/8/04

RECORD OF BOREHOLE No 04-108

1 OF 1 **METRIC**

PROJECT 04-1120-013 Phase 5000
W.P. 258-98-00 LOCATION N 5019185.6 ;E 417091.7 ORIGINATED BY J.S.
DIST HWY 417 BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger COMPILED BY M.I.C.
DATUM Geodetic DATE May 10, 2004 CHECKED BY M.I.C.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
65.1	GROUND SURFACE						
0.0 / 0.1	TOPSOIL; Silty CLAY and fine SAND, Brown				65		
64.5 / 0.6	Silty CLAY (Weathered Crust), Firm, Grey to brown, Wet	1	SS	4	64		
		2	SS	1			
63.1 / 2.0	Sandy SILT and clayey SILT, layered, Loose, Grey, Wet				63		
62.4 / 2.7	Sandy SILT with clay layers and some gravel (TILL), Compact to dense, Grey, Wet	3	SS	7	62		
		4	SS	13		o	29 33 33 5
		5	SS	48	61		
60.4 / 4.7	End of Borehole, Auger Refusal						

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75; UNIT WEIGHT γ kN/m³; STRAT PLOT; GROUND WATER CONDITIONS.

$+^{3}, \times^{3}$: Numbers refer to Sensitivity $O^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No 05-201

METRIC

PROJECT 04-1120-013-7000

W.P. 258-98-00

LOCATION N 5019183.46 ; E 417080.81

ORIGINATED BY P.A.H.

DIST   HWY 417

BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger

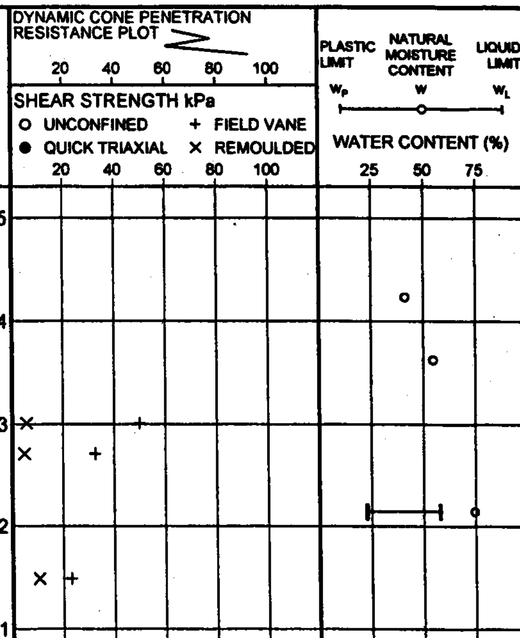
COMPILED BY M.I.C.

DATUM Geodetic

DATE February 21, 2005

CHECKED BY M.I.C.

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³
65.3	Ground Surface				
0.0	TOPSOIL				
65.0	Dark brown				
64.7 / 0.6	Sandy SILT, Grey brown, Moist				
	Silty CLAY, Very stiff to stiff, Grey brown, Wet	1	SS	3	
		2	SS	2	
63.0 / 2.3	Silty CLAY, Soft to firm, Grey, Wet	3	TP	PH	15.7
61.3 / 4.0	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL), Dense, Grey, Wet	4	SS	42	
60.9 / 4.4	End of Borehole, Auger Refusal				



REMARKS & GRAIN SIZE DISTRIBUTION (%): GR SA SI CL

GROUND WATER CONDITIONS

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

$O^{3\%}$ STRAIN AT FAILURE

TABLE 1

SUMMARY OF WATER CONTENT AND ATTERBURG LIMIT DETERMINATIONS

PROJECT NUMBER 04-1120-013
PROJECT NAME MTO/ HWY 417 CNR BRIDGE WIDENING / CASSELMAN
DATE TESTED

Borehole No.	Sample No.	Depth (m)	Water Content (%)	Atterberg Limits			
				W_L	W_P	LI	PI
04-101	5a	3.2-3.6	19.0%				
04-102	2	2.1-2.7	20.5%				
04-103	1	1.4-2	44.0%				
04-103	4	2.9-3.5	8.1%				
04-104	4	2-2.6	60.3%				
04-105	4	3.0-3.7	9.7%				
04-106	3	3.0-3.7	79.6%	81.8	25.3	1	56.5
04-106	5	4.6-5	9.1%				
04-107	1D	0.8-1.4	42.1%				
04-108	4	3-3.7	7.3%				

TABLE 2

COMPARISON OF FOUNDATION ALTERNATIVES
ABUTMENTS
EB CNR OVERPASS
HIGHWAY 417
STRUCTURE SITE 27-213/1

Embankment Option	***Feasibility***	***Advantages***	***Disadvantages***	***Relative Costs***	***Risks/Consequences***
Option 1 Piled Foundations	• Feasible	• High bearing resistance • Negligible settlements	• Possibility of encountering obstructions during driving	• Less expensive than caissons	• Risk of encountering obstructions during driving
Option 2 Cast-in-place concrete caissons founded on or socketed into bedrock	• Feasible	• High capacity • Negligible settlements	• Need to install a liner through embankment fill and native overburden • Need to inspect sockets. Typically required dewatering of socket.	• Likely more expensive than driven piles	• Possible ground loss or construction difficulties associated with liner installation and socket construction
Option 3 Spread footing foundations on embankment fill	• Not feasible	• N/A	• N/A	• N/A	• N/A

TABLE 3

COMPARISON OF FOUNDATION ALTERNATIVES PIERS
EB CNR OVERPASS
HIGHWAY 417
STRUCTURE SITE 27-213/1

Embankment Option	*Feasibility*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks/Consequences*
Option 1 Spread footing foundations on glacial till	• Feasible	• Limited excavation depth • Relatively simple construction • Consistent with existing foundations	• Lower bearing capacity, compared to footings on bedrock • Potential for minor settlement relative to existing structure • Requires excavating to about 3.5 metres depth, adjacent to railway line.	• Likely least expensive option.	• Potential for minor differential settlement relative to existing structure • Potential for need to sub-excavate founding surface, if becomes disturbed. • Potential for excavation to impact on railway line.
Option 2 Spread footing foundations on bedrock	• Feasible	• Higher bearing capacity • Settlement negligible, therefore very limited differential settlement relative to existing structure	• Requires deeper excavation, adjacent to railway line. Shoring will be required. • Excavation could undermine existing footings. Underpinning could be required.	• Likely more expensive than footings on glacial till.	• Potential for deeper excavation to impact on railway line • Potential for excavation to undermine existing footings and cause settlements • Potential construction problems with making excavation (e.g., groundwater control)
Option 3 Piled Foundations	• Feasible	• High bearing resistance • Negligible settlements	• Possibility of encountering obstructions during driving • Piles will be short (approx. 4m) and therefore have limited lateral resistance.	• Probably more expensive than footing options	• Risk of encountering obstructions during driving
Option 2 Cast-in-place concrete caissons founded on or socketed into bedrock	• Feasible	• High capacity • Negligible settlements	• Need to install a liner • Require access by larger equipment, adjacent to railway line • Need to inspect sockets. Typically required dewatering of socket.	• Likely most expensive option	• Possible ground loss or construction difficulties associated with liner installation and socket construction

TABLE 4

COMPARISON OF EMBANKMENT ALTERNATIVES
EB CNR OVERPASS EMBANKMENTS

Embankment Option	*Feasibility*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks/Consequences*
Option 1 Allow embankments to settle	• Feasible, if can accept settlements	• No impact on construction schedule or costs	• Requires post-construction maintenance • Possible safety issue due to settlement	• Relatively low costs, but must consider short term post-construction maintenance costs	• Possible excessive roadway settlement of widened area and edge of adjacent lane.
Option 2 Pre-load	• Feasible	• Minimum post-construction maintenance required, depending on construction schedule	• Delays paving and use of ramp.	• Similar cost as Option 1	• Some uncertainty about schedule, since can not start construction until monitoring indicates sufficient settlement has occurred.
Option 3 Light weight fill	• Feasible	• No post-construction maintenance. • Minimal impact on schedule • If used near east abutment, would reduce settlement in that area.	• Expensive	• Expensive	• Low risk option, but contractor may successfully propose one of other options as change order

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

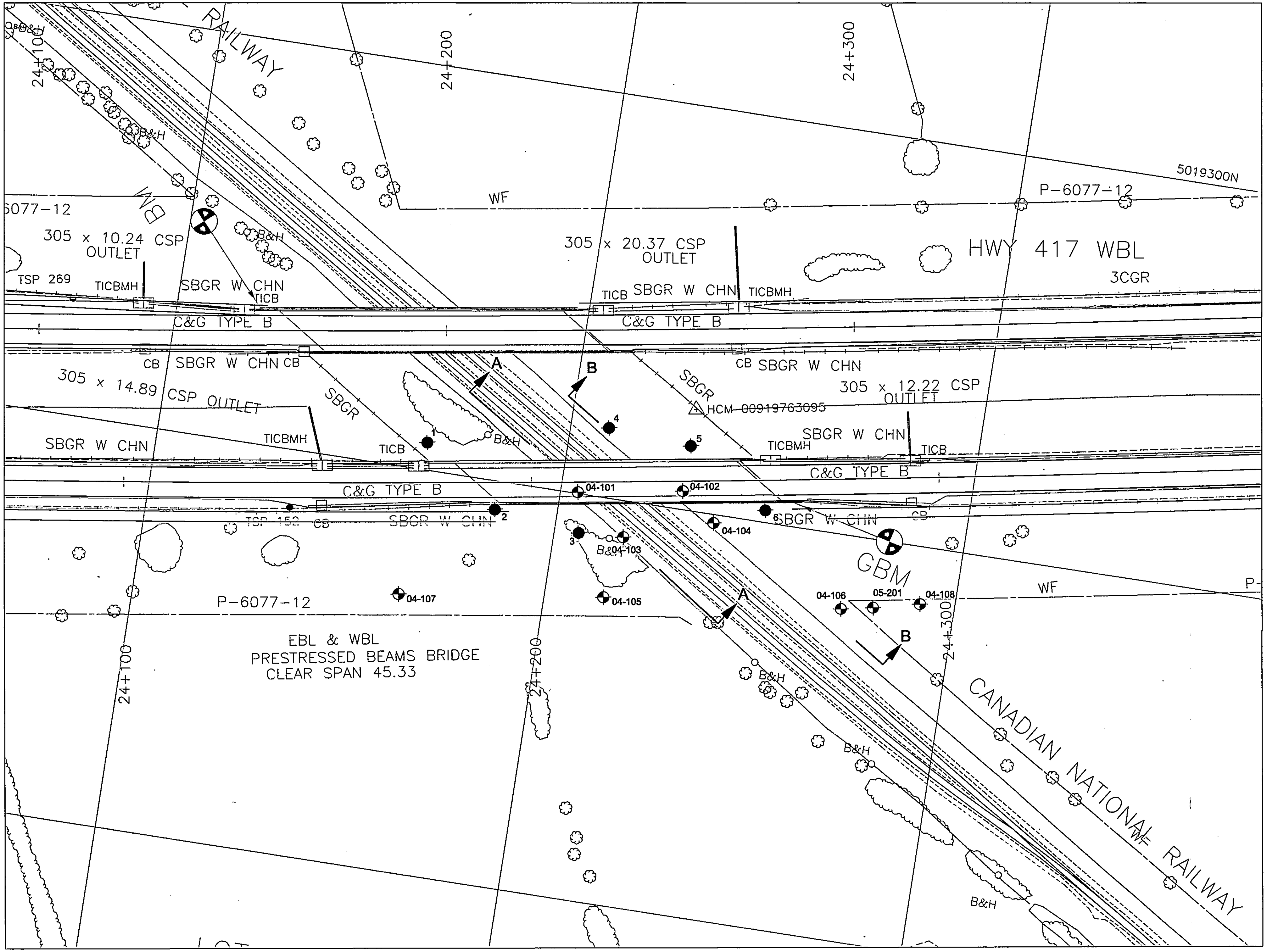
DIST No. 42
CONT No.
WP No. 258-98-00

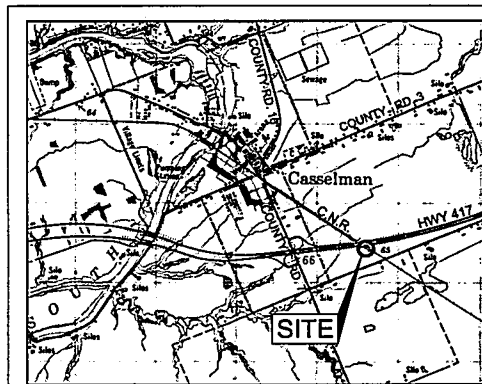
SHEET

C.N.R. OVERHEAD EBL BRIDGE
STRUCTURE REHABILITATION
BOREHOLE LOCATIONS
AND SOIL STRATA

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA





KEY PLAN

LEGEND

Borehole – Current Golder Associates Ltd. Investigation

Borehole – Previous MTO Investigation Geocres No. 31GA48

Seal

Piezometer

N Standard Penetration Test value

16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)

100% Rock Quality Designation (RQD)

WL in piezometer (May 20, 2004)

1:250

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
04–101	66.0	5019200.24	417004.17
04–102	65.8	5019204.56	417029.57
04–103	65.9	5019191.01	417016.72
04–104	65.2	5019197.85	417038.36
04–105	65.6	5019175.50	417014.44
04–106	65.5	5019181.70	417072.51
04–107	65.8	5019168.47	416964.75
04–108	65.1	5019185.64	417091.69
05–201	65.3	5019183.46	417080.81
1 (MTO)	65.1	5019206.9	416965.8
2 (MTO)	65.0	5019193.0	416984.8
3 (MTO)	64.7	5019190.3	417006.1
4 (MTO)	64.9	5019217.4	417009.4
5 (MTO)	65.0	5019215.9	417029.9
6 (MTO)	65.1	5019202.9	417050.4

NOTES

The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

Base plan provided in electronic format by Morrison Hershfield Limited

SCALE 10 0 10 20 METRES

REVISIONS DESCRIPTION

DESIGN	CHK	CODE 041120013-5000-01	LOAD	DATE July 2005
DRAWN S.L.	CHK M.I.C	SITE 27-213/1	CL-625-ONT.	DWG 1

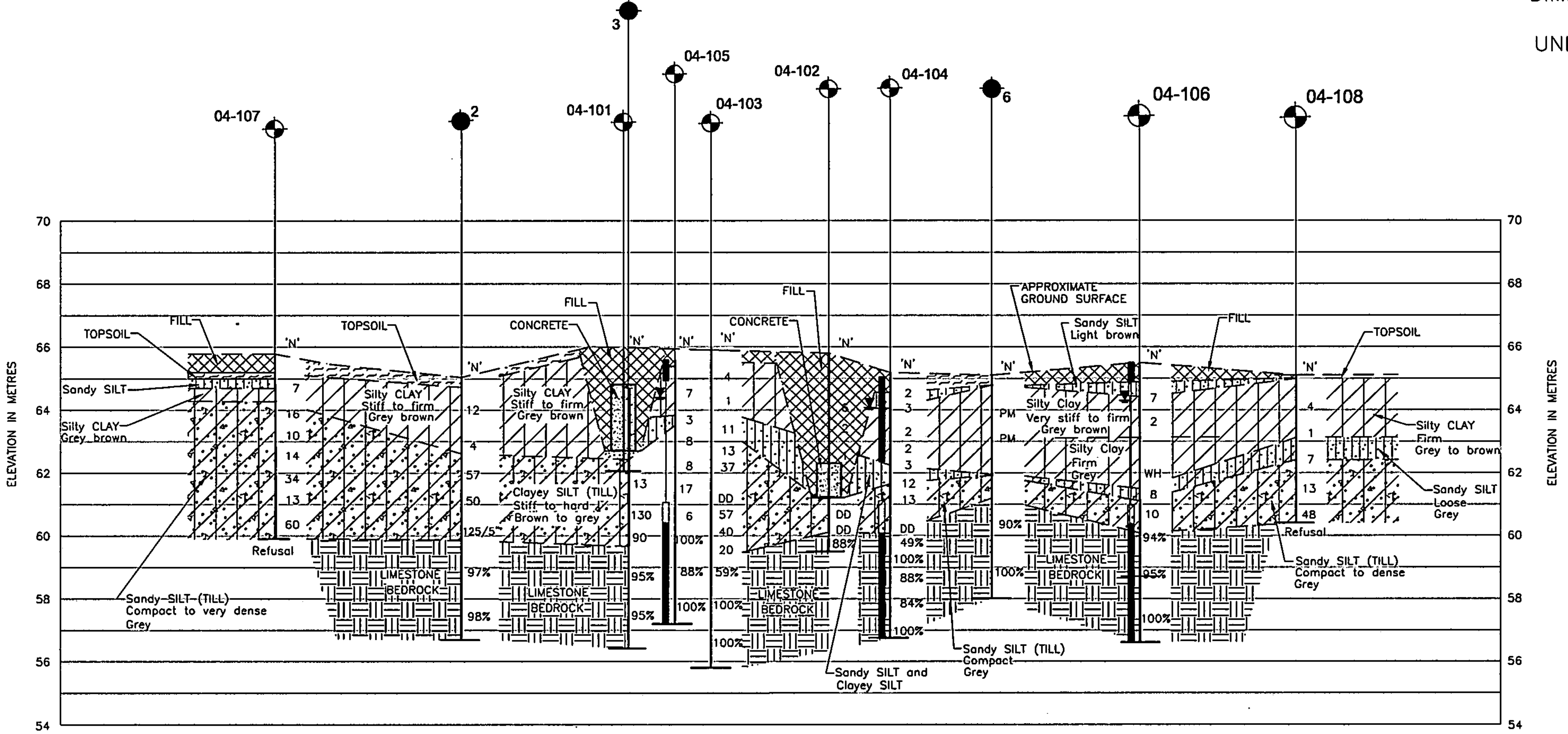
DRAWING NAME: 041120013-5000-203A MODIFIED: 2005/05/16 10:09:09
CREATED: S.L.

MINISTRY OF TRANSPORTATION, ONTARIO PH-D-707 88-05

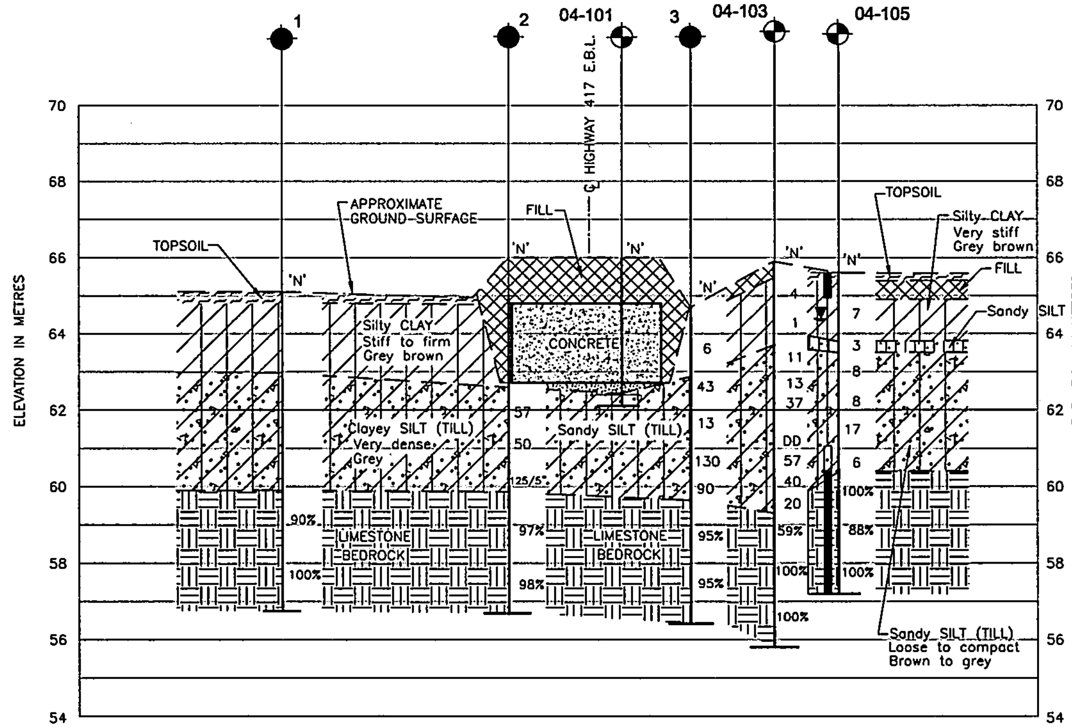
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

DIST No. 42 CONT No. WP No. 258-98-00	
C.N.R. OVERHEAD EBL BRIDGE STRUCTURE REHABILITATION BOREHOLE LOCATIONS AND SOIL STRATA	SHEET

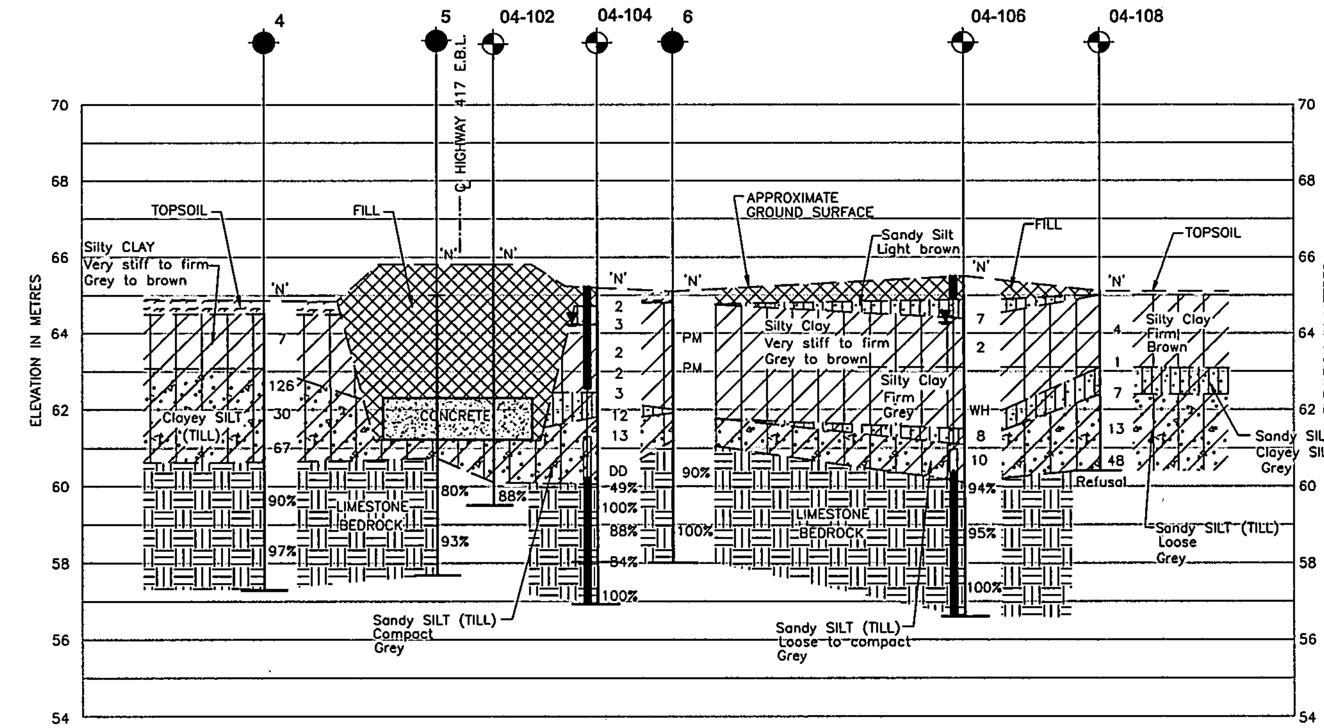
Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PROFILE ALONG HIGHWAY 417 E.B.L.

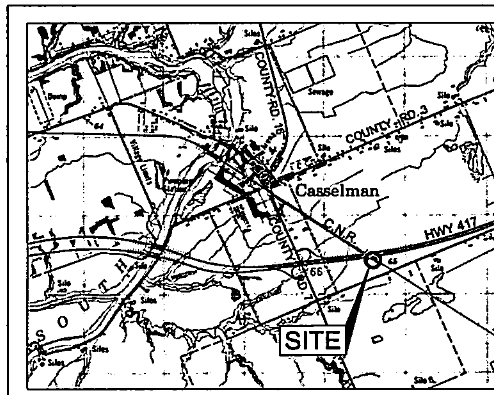


SECTION A-A



SECTION B-B

SCALE HORIZONTAL 8 0 8 16 METRES
SCALE VERTICAL 2 0 2 4 METRES



KEY PLAN

LEGEND

Symbol	Description
(borehole symbol)	Borehole – Current Golder Associates Ltd. Investigation
(filled borehole symbol)	Borehole – Previous MTO Investigation Geocres No. 31GA48
	Seal
	Piezometer
N	Standard Penetration Test value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
100%	Rock Quality Designation (RQD)
▼	WL in piezometer (May 20, 2004)

1:250

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
04-101	66.0	5019200.24	417004.17
04-102	65.8	5019204.56	417029.57
04-103	65.9	5019191.01	417016.72
04-104	65.2	5019197.85	417038.36
04-105	65.6	5019175.50	417014.44
04-106	65.5	5019181.70	417072.51
04-107	65.8	5019168.47	416964.75
04-108	65.1	5019185.64	417091.69
1 (MTO)	65.1	5019206.9	416965.8
2 (MTO)	65.0	5019193.0	416984.8
3 (MTO)	64.7	5019190.3	417006.1
4 (MTO)	64.9	5019217.4	417009.4
5 (MTO)	65.0	5019215.9	417029.9
6 (MTO)	65.1	5019202.9	417050.4

NOTES

The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

Base plan provided in electronic format by Morrison Hershfield Limited

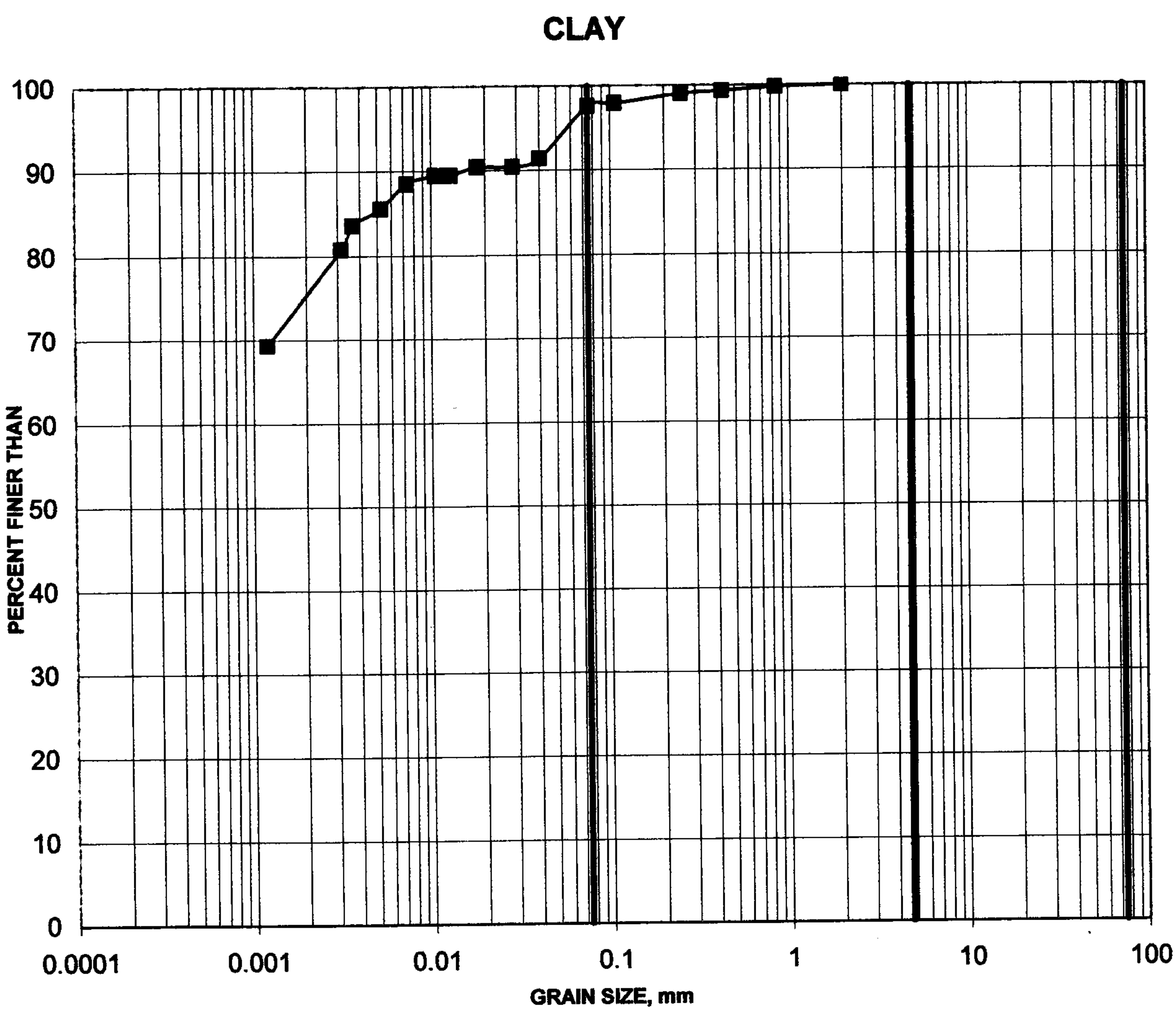
REVISIONS | DESCRIPTION

DESIGN	CHK	CODE 041120013-5000-02	LOAD	DATE July 2005	
DRAWN S.L.	CHK M.I.C	SITE 27-213/1	CL-625-ONT.	DWG 2	

MINISTRY OF TRANSPORTATION, ONTARIO PH-O-707 88-05

DRAWING NAME: 041120013-5000-203B MODIFIED: 2005/05/16 10:09:09
CREATED: S.L.

GRAIN SIZE DISTRIBUTION	FIGURE 1

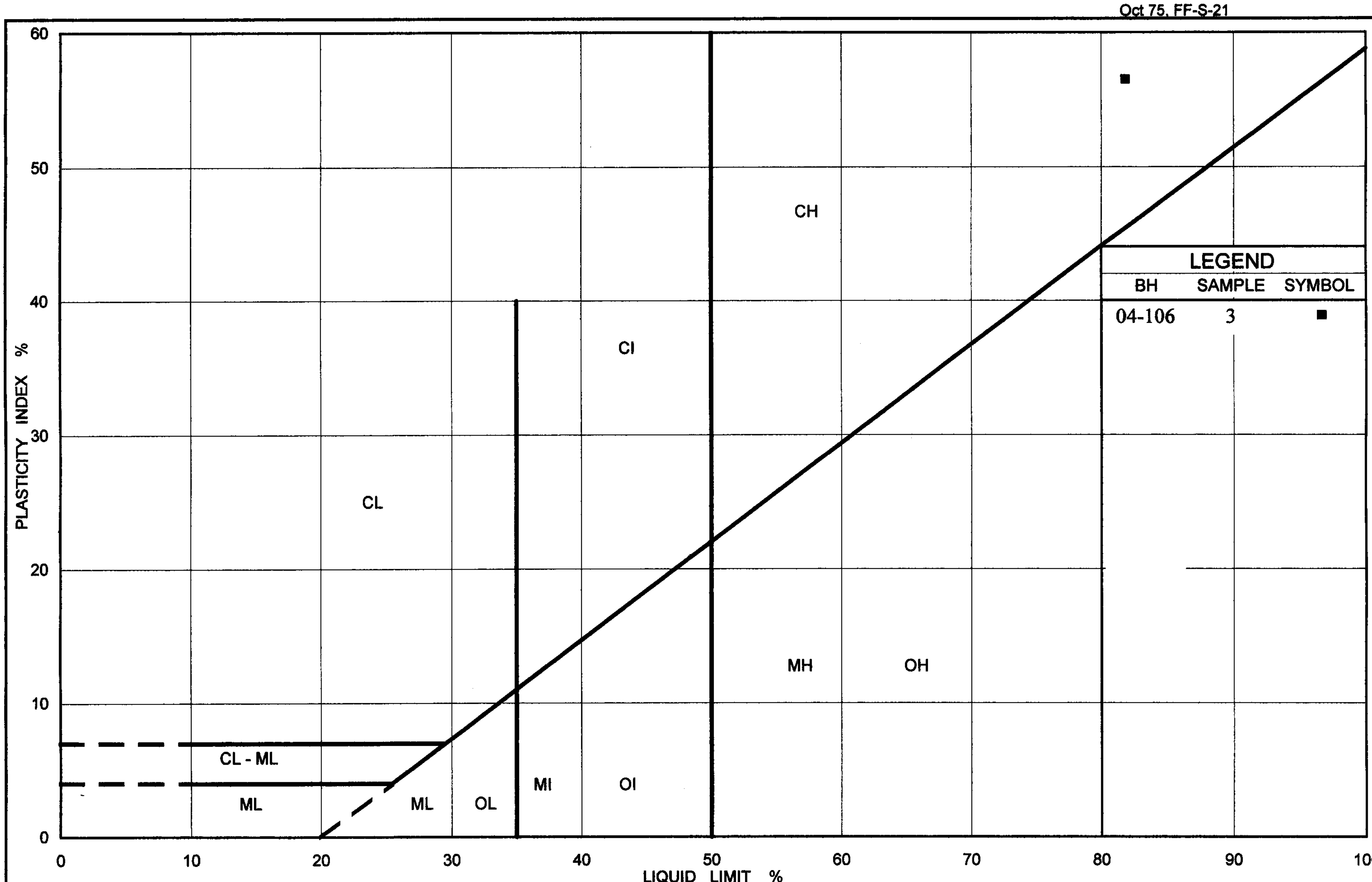


SILT AND CLAY	SAND SIZE			GRAVEL SIZE		COBBLE SIZE
	FINE	MEDIUM	COARSE	FINE	COARSE	

	Borehole	Sample	Depth (m)
—■—	04-106	3	3.0-3.7

Created by: JML
Checked by: EWK

Project: 04-1120-013 5000 **Golder Associates**



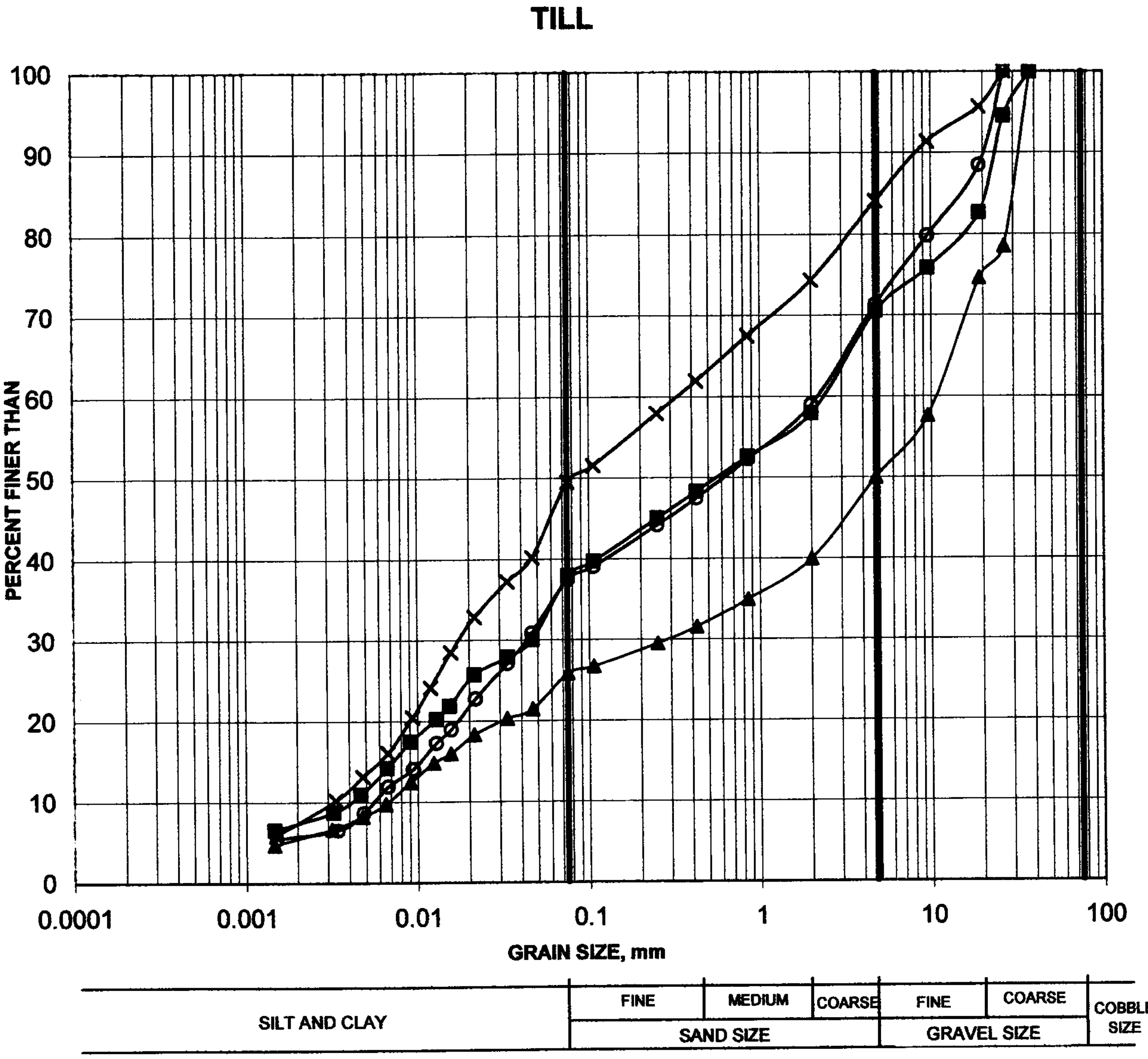
Ministry of Transportation
Ontario

Highly Plastic Clay
(Borehole 04-106)

FIG No. 2

Project No. 04-1120-013

GRAIN SIZE DISTRIBUTION	FIGURE 3

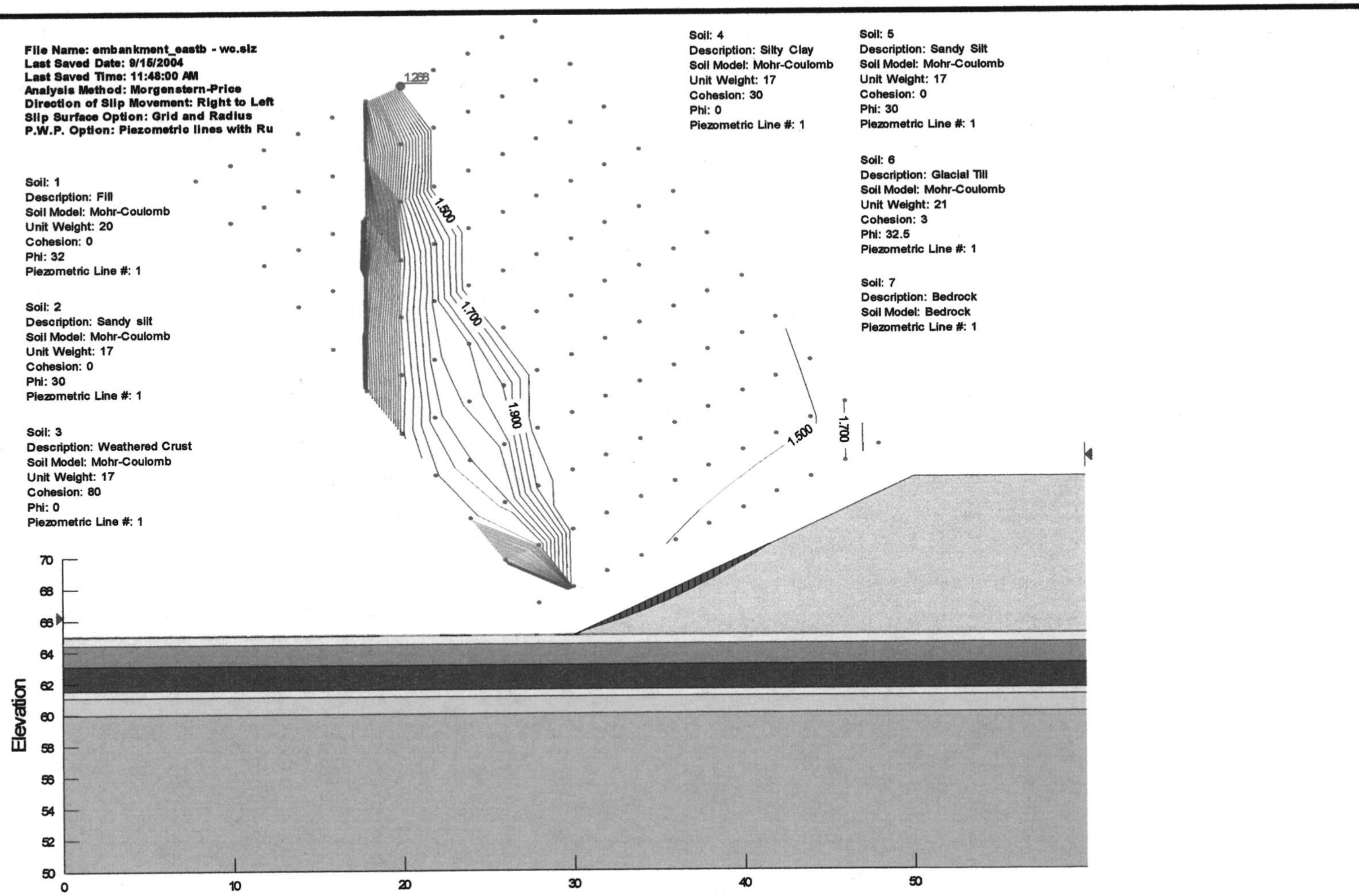


Borehole	Sample	Depth (m)
04-103	4	2.9-3.5
04-105	4	3.0-3.7
04-106	5	4.6-5.0
04-108	4	3.0-3.7

Project: 04-1120-013 5000

Golder Associates

Created by: JML
Checked by: EWK



PROJECT No.	04-1120-013
FILE No.	04-1120-013
REV. 0	NOT TO SCALE
DESIGN	WC 09/15/04
CADD	
CHECK	
REVIEW	

FIGURE 4

TITLE

East Embankment Surficial Failure

PROJECT

G.W.P. 258-98-00

Golder Associates

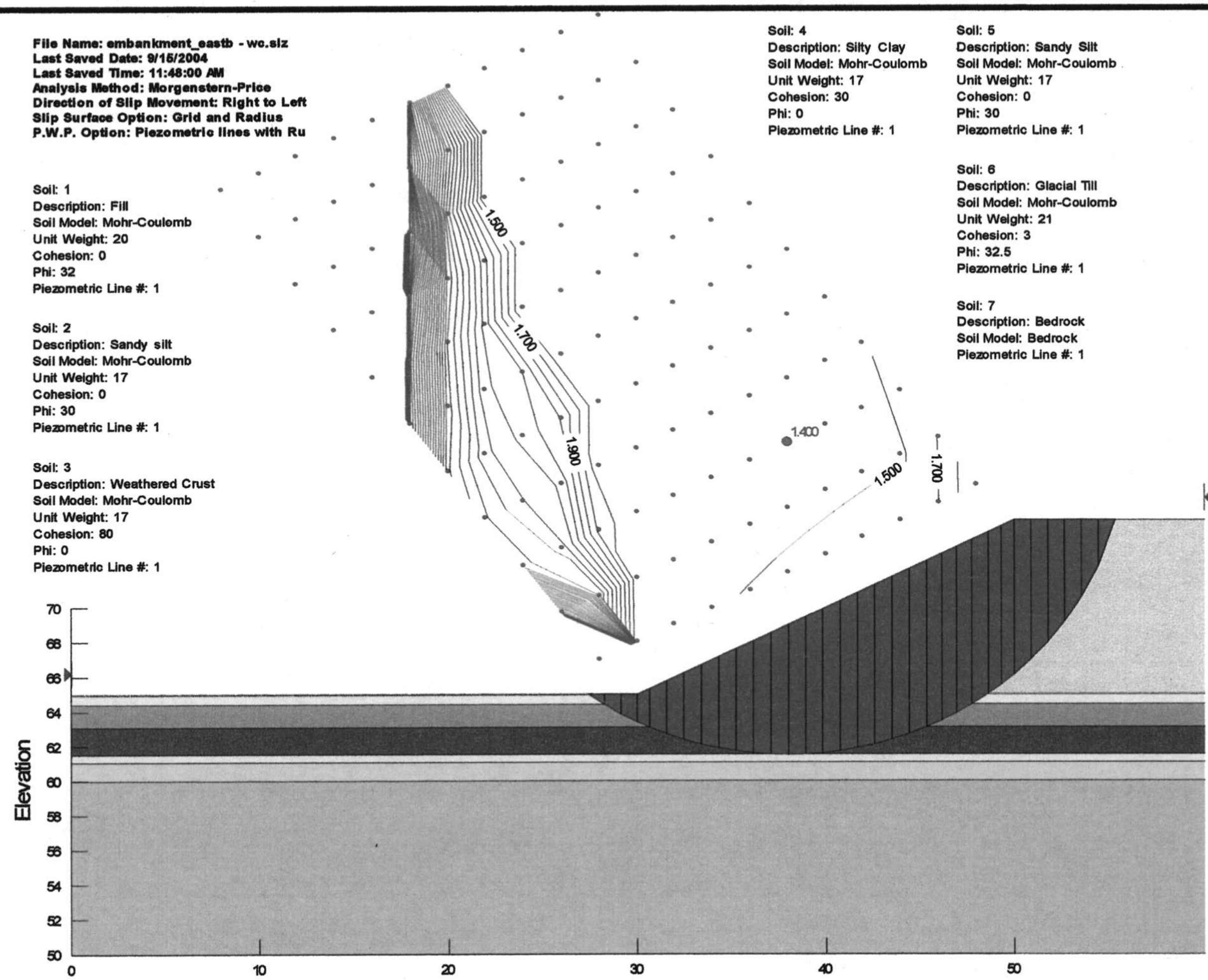


FIGURE 5		
PROJECT No.	04-1120-013	
FILE No.	04-1120-013	
REV. 0	NOT TO SCALE	
DESIGN	WC	09/15/04
CADD		
CHECK		
REVIEW		

TITLE

East Embankment Global Failure

PROJECT

G.W.P. 258-98-00

Golder Associates

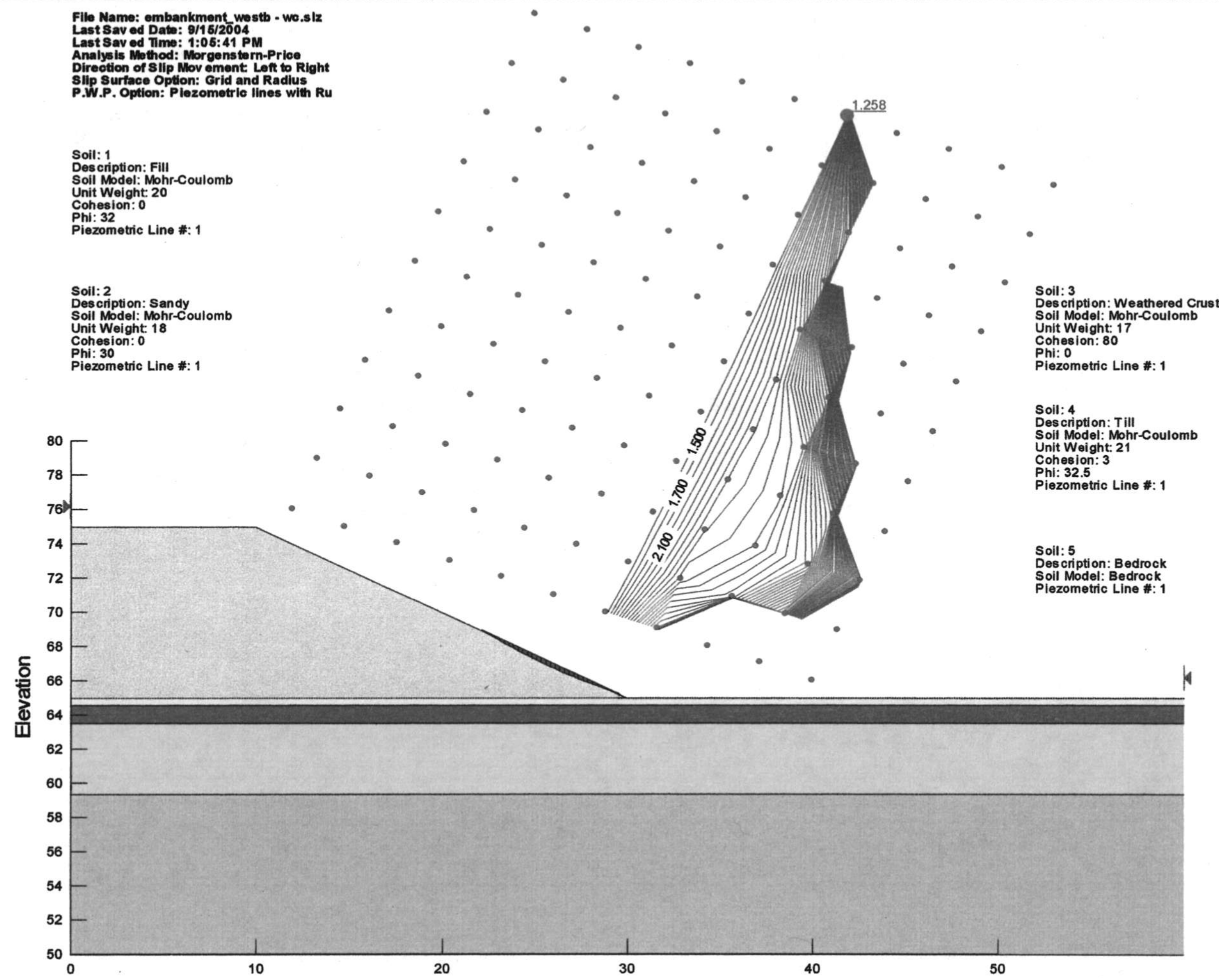


FIGURE 6

PROJECT No.	04-1120-013
FILE No.	04-1120-013
REV. 0	NOT TO SCALE
DESIGN	WC 09/15/04
CADD	
CHECK	
REVIEW	

TITLE

West Embankment Surficial Failure

PROJECT

G.W.P. 258-98-00

Golder Associates

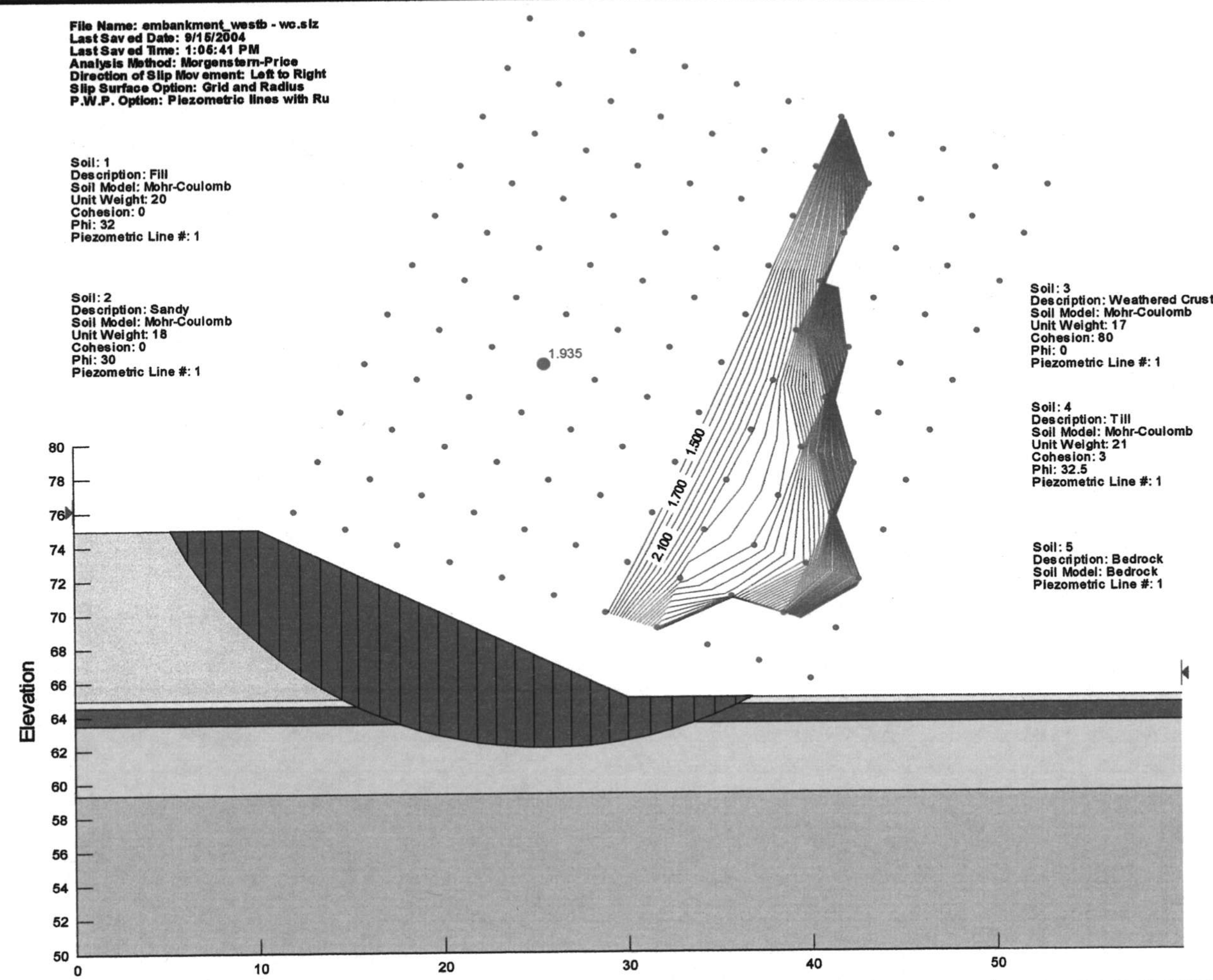


FIGURE 7	PROJECT No. 04-1120-013 FILE No. 04-1120-013 REV. 0 — NOT TO SCALE DESIGN WC 09/15/04 CADD CHECK REVIEW	
TITLE **West Embankment Global Failure**	PROJECT G.W.P. 258-98-00	Golder Associates

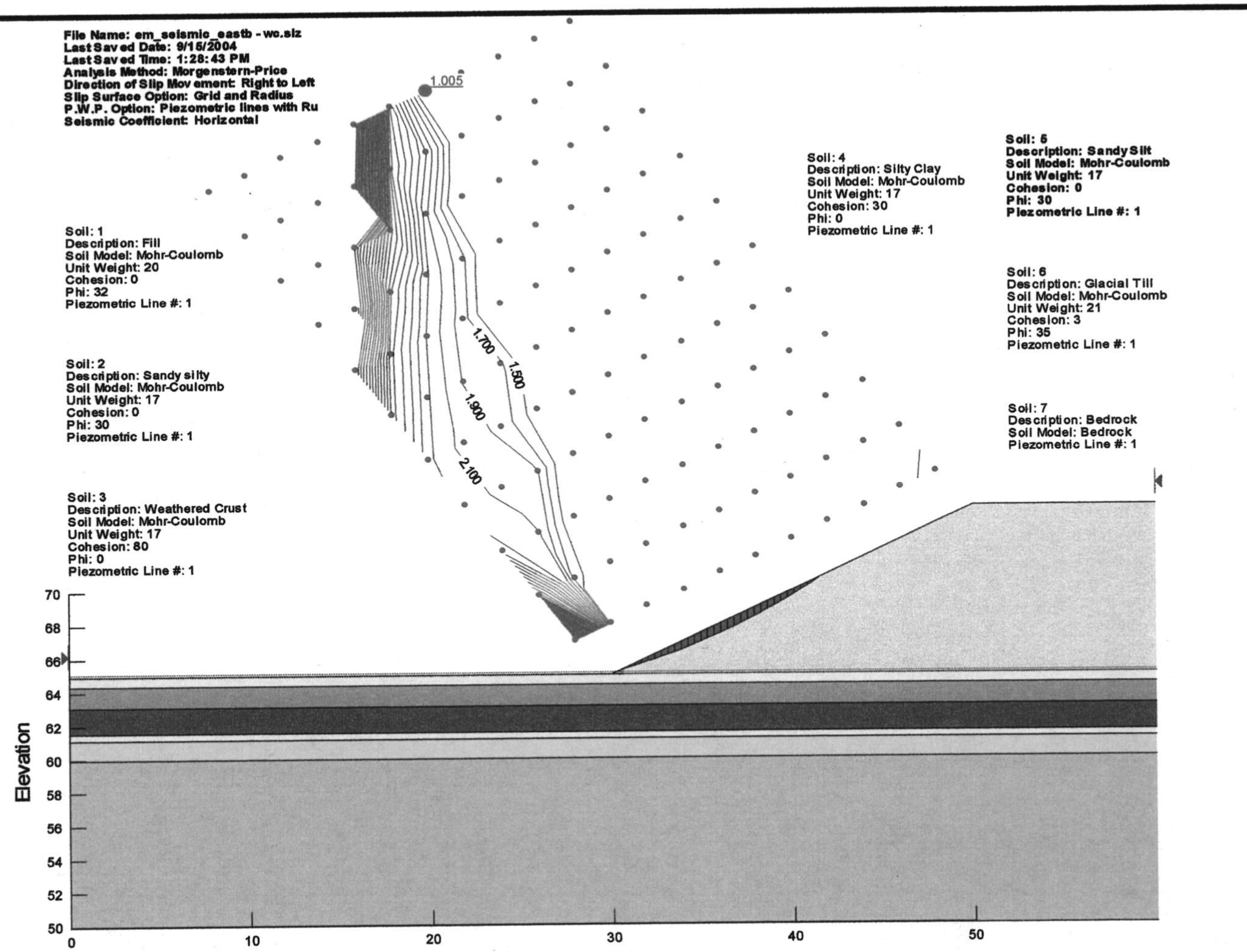


FIGURE 8	REVIEW	CHECK	CADD	DESIGN	REV. 0	FILE No.	PROJECT No.
				WC	NOT TO SCALE	04-1120-013	04-1120-013
				09/15/04			

TITLE

East Embankment Seismic Surficial Failure

PROJECT

G.W.P. 258-98-00

Golder Associates

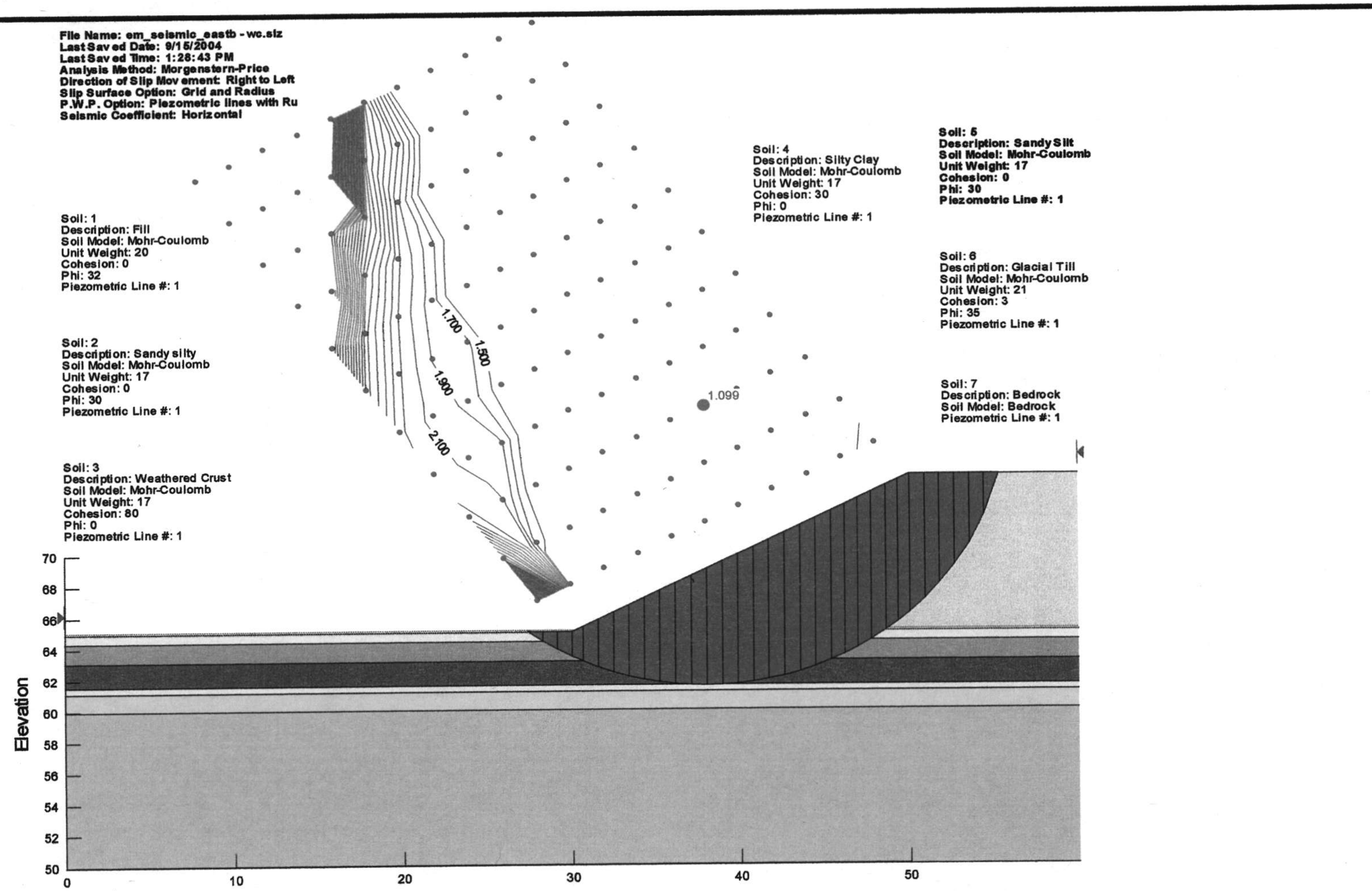


FIGURE 9	
REVIEW	
CHECK	
CADD	
DESIGN	WC 09/15/04
REV. 0	NOT TO SCALE
FILE No.	04-1120-013
PROJECT No.	04-1120-013

TITLE

East Embankment
Seismic Global Failure

PROJECT

G.W.P. 258-98-00

Golder Associates

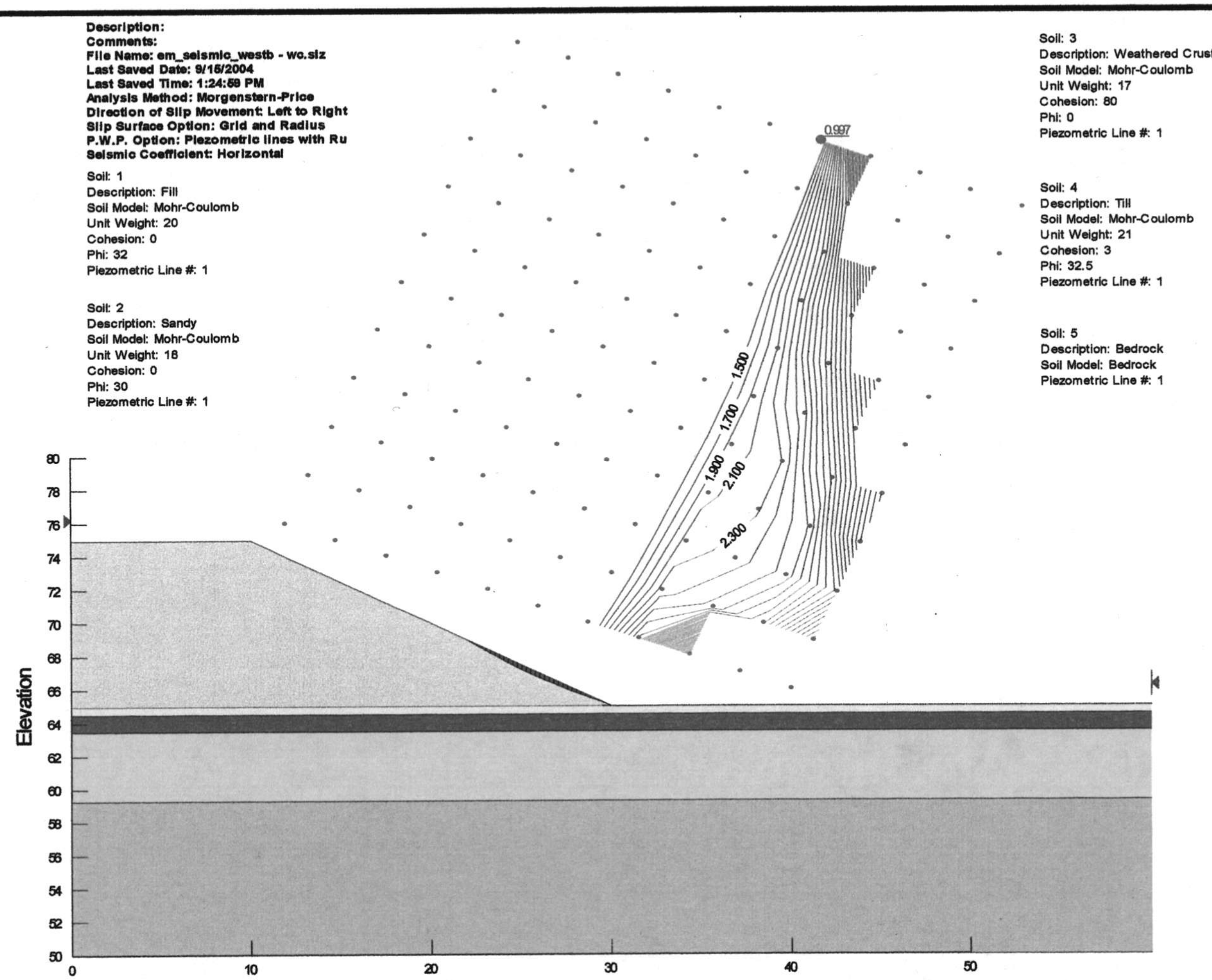


FIGURE 10		
PROJECT No.	04-1120-013	
FILE No.	04-1120-013	
REV. 0	NOT TO SCALE	
DESIGN	WC	09/15/04
CADD		
CHECK		
REVIEW		

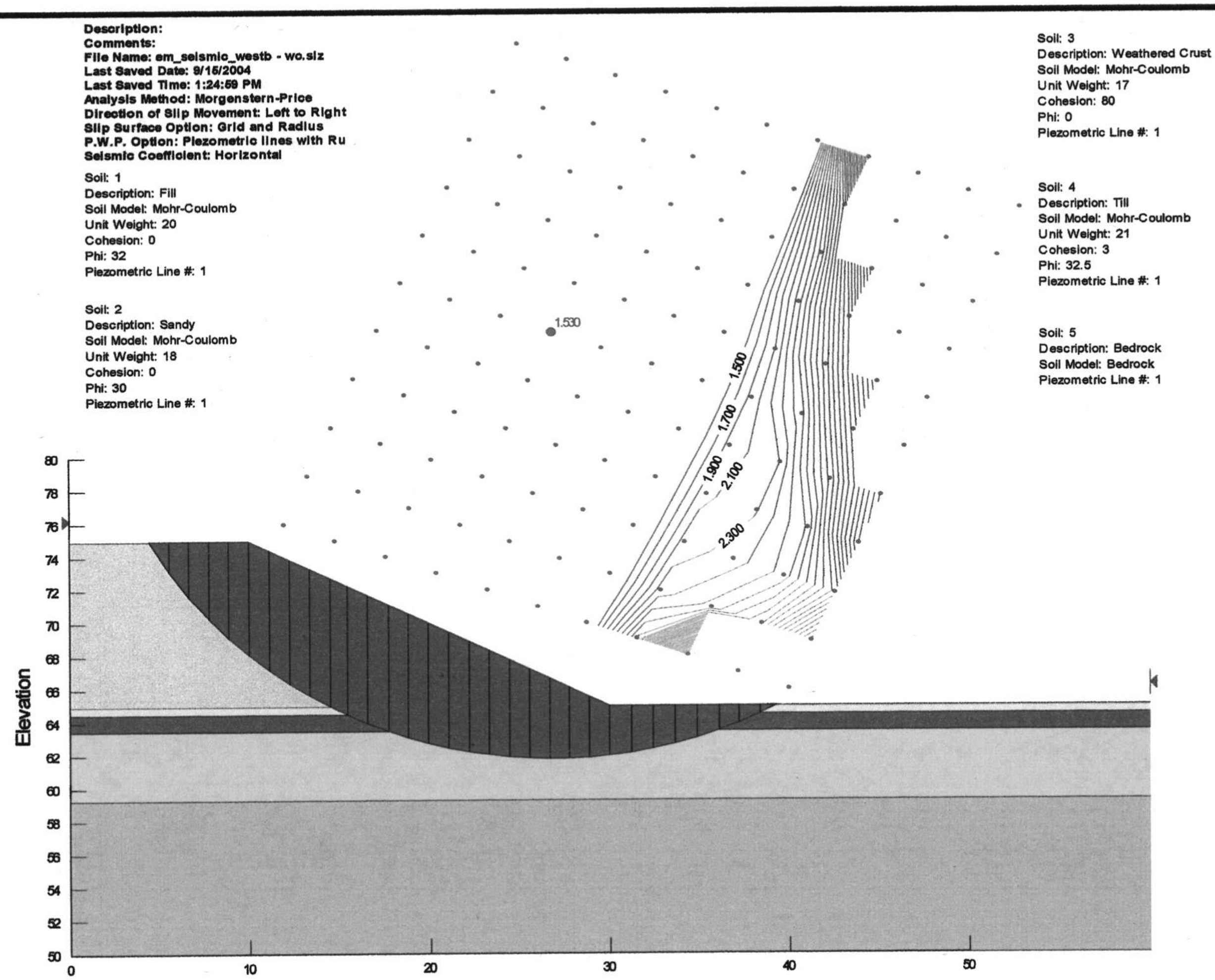
TITLE

West Embankment
Seismic Surficial Failure

PROJECT

G.W.P. 258-98-00

Golder Associates



PROJECT No.		04-1120-013
FILE No.		04-1120-013
REV. 0		NOT TO SCALE
DESIGN	WC	09/15/04
CADD		
CHECK		
REVIEW		

FIGURE 11

TITLE

West Embankment Sesimic Global Failure

PROJECT

G.W.P. 258-98-00

Golder Associates

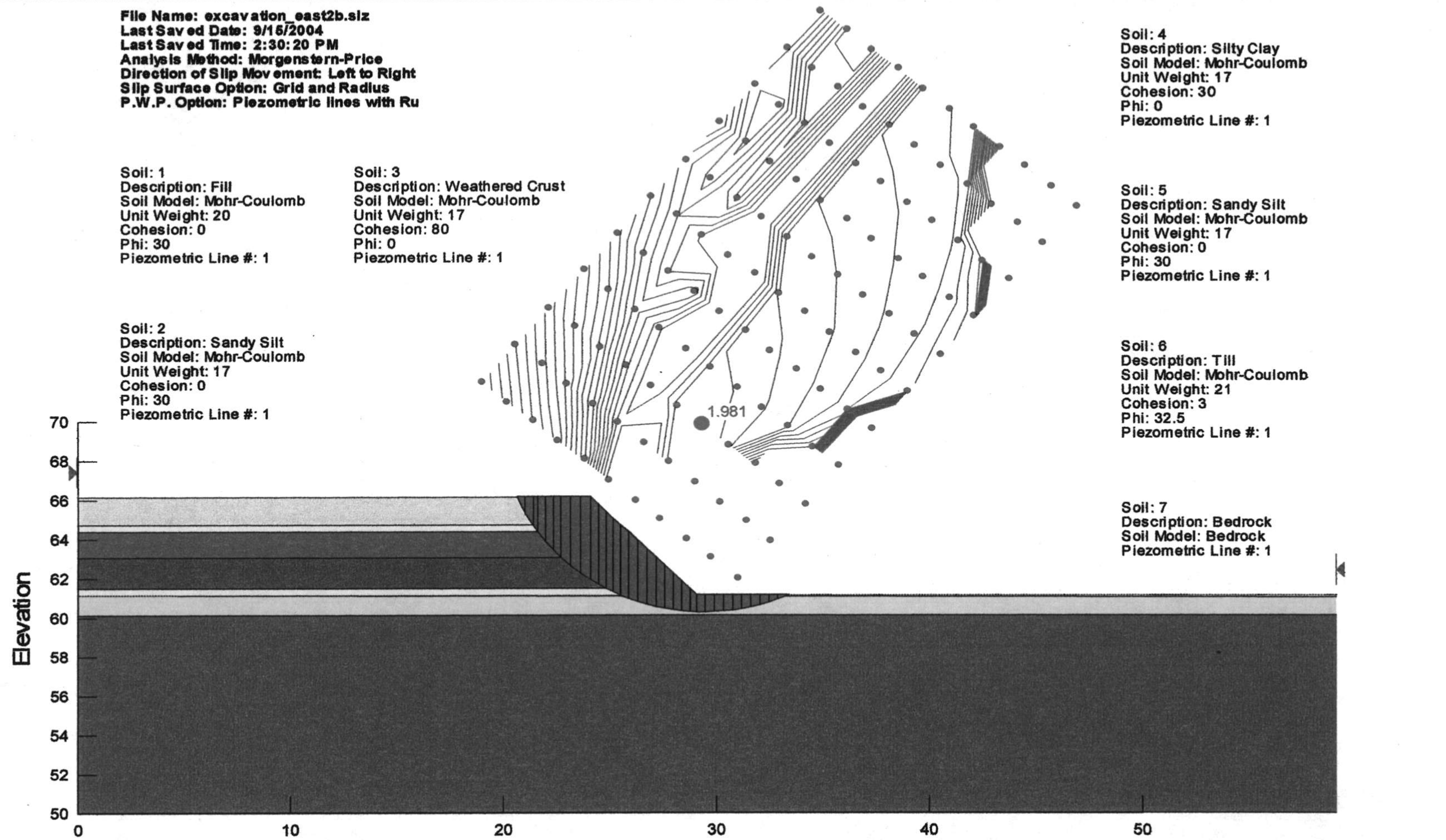


FIGURE 12	REVIEW	CHECK	CADD	DESIGN: WC, 09/15/04	REV. 0: NOT TO SCALE	FILE No.: 04-1120-013	PROJECT No.: 04-1120-013	TITLE: East Excavation	PROJECT: G.W.P. 258-98-00	Golder Associates

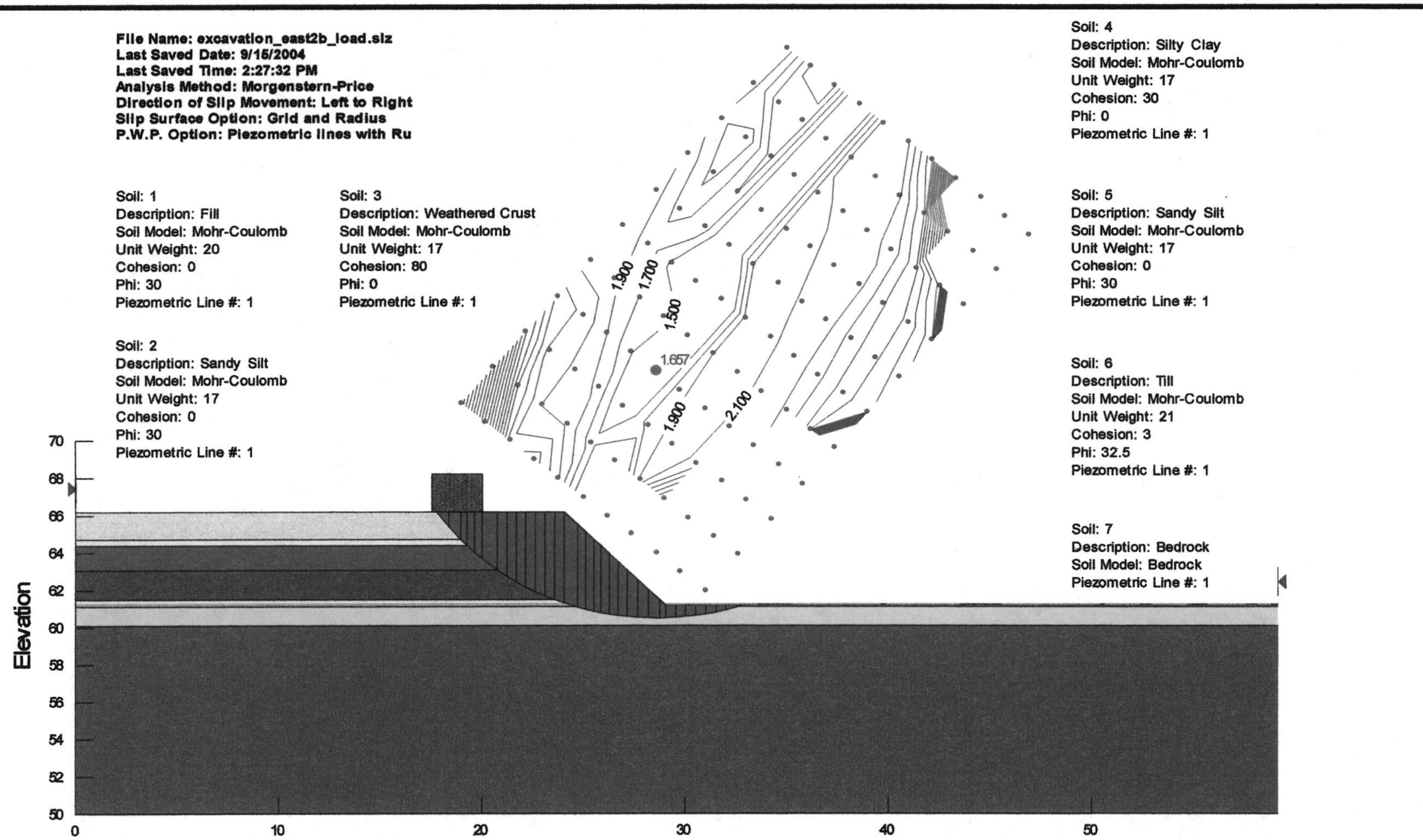


FIGURE 13		
REVIEW		
CHECK		
CADD		
DESIGN	WC	09/15/04
REV. 0	NOT TO SCALE	
FILE No.	04-1120-013	
PROJECT No.	04-1120-013	

TITLE

East Excavation (with surcharge)

PROJECT

G.W.P. 258-98-00

Golder Associates

APPENDIX A

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 1

JOB 70-F-7 LOCATION Sta. 465 + 09 ℄ Prop. Hwy.417 EBL o/s 30' Lt. ORIG

W.P. 35-66-17 BORING DATE February 6 & 25, 1970 COM

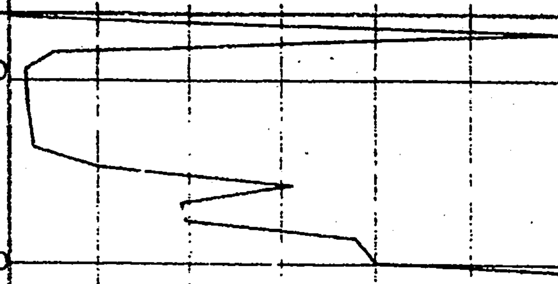
DATUM Geodetic BOREHOLE TYPE Washboring-BX Casing; Cone CHE

SOIL PROFILE			SAMPLES			
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE
213.5	Ground Level					
0.0	Topsoil					
1.0	Silty clay to clay					210
206.5	Grey - Brown					
7.0	Glacial Till					
	Brown to Grey Brown					200
196.4						
17.1	Limestone Bedrock		1	AXT	90%	
	Sound		2	AXT	100%	190
186.1						
27.4	End of Borehole					
						180

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100

SHEAR STRENGTH P.S.F.

O UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB. VANE



LIQUID LIMIT –
PLASTIC LIMIT
WATER CONTE
w_p w
WATER CC

FORM OB-MT-126 (REV 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 2

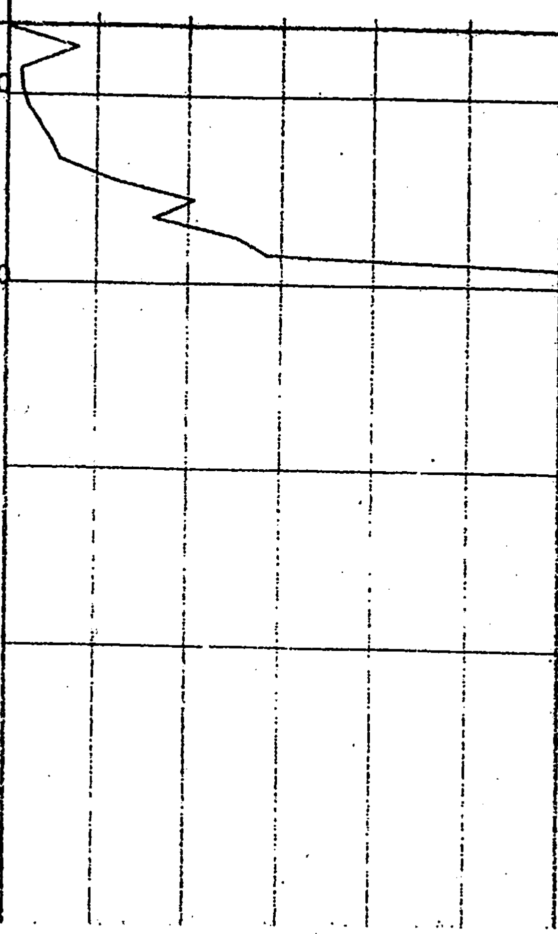
JOB 70-F-7
W.P. 35-66-17
DATUM Geodetic

LOCATION Sta. 465 + 62 ℄ Prop. Hwy. 417 EBL o/s 18' Rt.
BORING DATE February 17 - 18, 1970
BOREHOLE TYPE Washboring-BX & BX Casing; Cone

ELEV. DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FOOT
213.4	Ground Level			
0.0	Topsoil			
1.0	Clay with trace of sand. Stiff - Firm	1	SS	12
	Mottled Brown - Grey	2	SS	4
205.4 / 8.0	Het.mix.of clayey silt sand & gravel (Glacial Till) Hard	3	SS	57
	occ. non-cohesive zones. Very dense.Grey	4	SS	50
		5	SS	125/5"
196.4 / 17.0	Limestone Bedrock	6	AXT	97%
	Sound	7	AXT	98%
186.0 / 27.4	End of Borehole			

ELEV. SCALE: 210, 200, 190, 180

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F.
o UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB. VANE



LIQUID LIMIT
PLASTIC LIM
WATER CON
W_P W
WATER C
25 50

FORM OB-MT-126 (REV. 1969)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO
MATERIALS & TESTING OFFICE

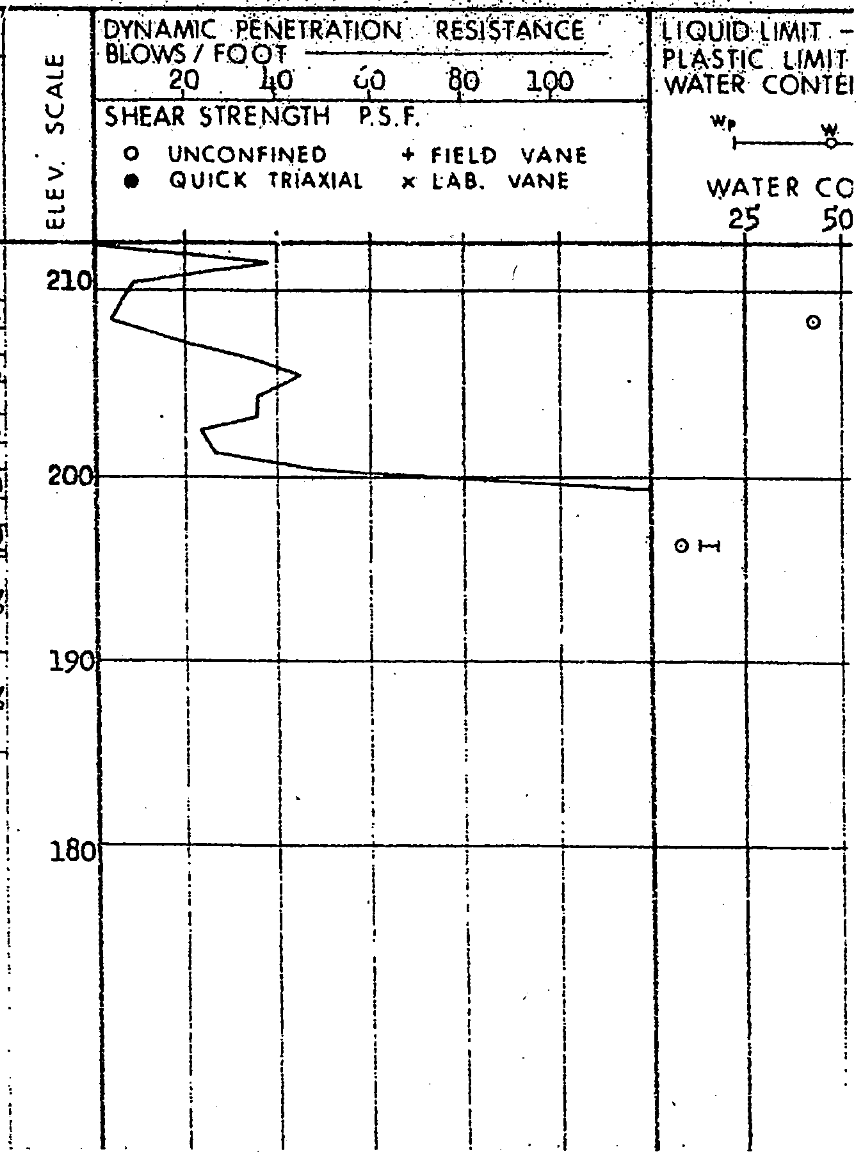
RECORD OF BOREHOLE No. 3

JOB 70-F-7
W.P 35-66-17
DATUM Geodetic

LOCATION Sta. 466 + 25 Ø Prop Hwy.417 EBL o/s 37' Rt.
BORING DATE February 16-17, 1970
BOREHOLE TYPE Washboring-NX & BX Casing; Cone

ORIG
COM
CHE

ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FOOT
212.4	Ground Level			
0.0	Topsoil			
1.0	Clay with trace of sand & gravel.	1	SS	6
206.4	Firm. Brown-Grey.			
6.0	Het.mix.of clayey silt sand & gravel(Glac.Till) Stiff to hard. Brown-Grey occ. non-cohesive seams.	2	SS	43
		3	SS	13
		4	SS	130
195.8	Very dense. Grey.	5	SS	90
16.6	Limestone Bedrock	6	AXT	95%
	Sound	7	AXT	95%
185.2				
27.2	End of Borehole			



DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 4

JOB 70-F-7
W.P. 35-66-17
DATUM Geodetic
LOCATION Sta. 466 + 55 ¢ Prop.Hwy.417 EBL o/s 42' Lt.
BORING DATE February 24, 1970
BOREHOLE Washboring-BX Casing; Cone
OR
COM
CH

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	BLOWS / FOOT	ELEV SCALE
212.8	Ground Level					
0.0	Topsoil					
1.2	Clay with trace sand & silt seams.					210
			1	SS	7	
206.8	Firm. Grey					
6.0	Het.mix.of clayey silt, sand & gravel (Glacial Till)		2	SS	126	
			3	SS	30	
198.8	Hard. Grey-Brown.		4	SS	67	200
14.0	Limestone Bedrock		5	AXT	90%	
	Sound		6	AXT	97%	190
188.0						
24.8	End of Borehole					180

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT: 20 40 60 80 100
SHEAR STRENGTH PSF
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB. VANE

LIQUID LIMIT
PLASTIC LIMI
WATER CONT
W_P
WATER C
25 50

FORM OB-MT-[illegible] (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS- ONTARIO

MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 5

JOB 70-F-7 LOCATION Sta. 467 + 24 ⌀ Prop. Hwy.417 EBL o/s 30! Lt. ORIG

W.P. 35-66-17 BORING DATE February 20 & 24, 1967 COMP

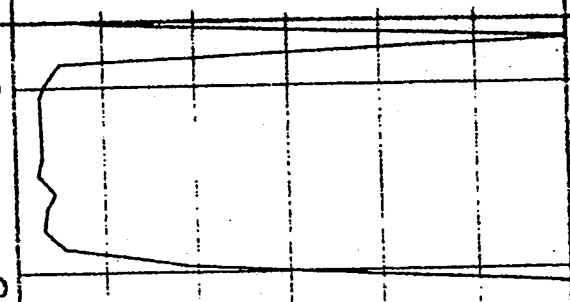
DATUM Geodetic BOREHOLE TYPE Washboring-BX Casing; Cone CHEC

SOIL PROFILE			SAMPLES				
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100; SHEAR STRENGTH P.S.F. o UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE
213.3	Ground Level						
0.0	Topsoil						
1.0	Clay Brown to Grey					210	
202.3 / 11.0	Glacial Till Grey-Brown						
199.3 / 14.0	Limestone Bedrock		1	AXT	80%	200	
	Sound		2	AXT	93%		
189.3 / 24.0	End of Borehole					190	
						180	

LIQUID LIMIT —
PLASTIC LIMIT -
WATER CONTEN[T]

W_P W

WATER CO[NTENT]



FORM OB-MT-126 (REV. 1969) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No. 6

JOB 70-F-7 LOCATION Sta. 467 + 79 ℄ Prop.Hwy.417 EBL o/s 18' Rt. ORI

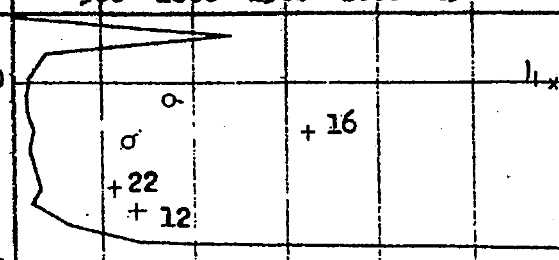
W.P. 35-66-17 BORING DATE February 20, 1970 COM

DATUM Geodetic BOREHOLE TYPE Washboring-NX & BX Casing; Cone CHE

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	STRAT. PLOT	SAMPLES – NUMBER	SAMPLES – TYPE	SAMPLES – BLOWS/FOOT	ELEV. SCALE
213.5	Ground Level					
0.0	Topsoil					
1.0	Clay with trace sand & occ. silt seams. Very stiff to firm Brown & Grey		1	TW	PM	210
			2	TW	PM	
203.0 / 10.5	Glacial Till					
200.7 / 12.8	Limestone Bedrock		3	AXT	90%	200
	Sound		4	AXT	100%	
190.2 / 23.3	End of Borehole					190
						180

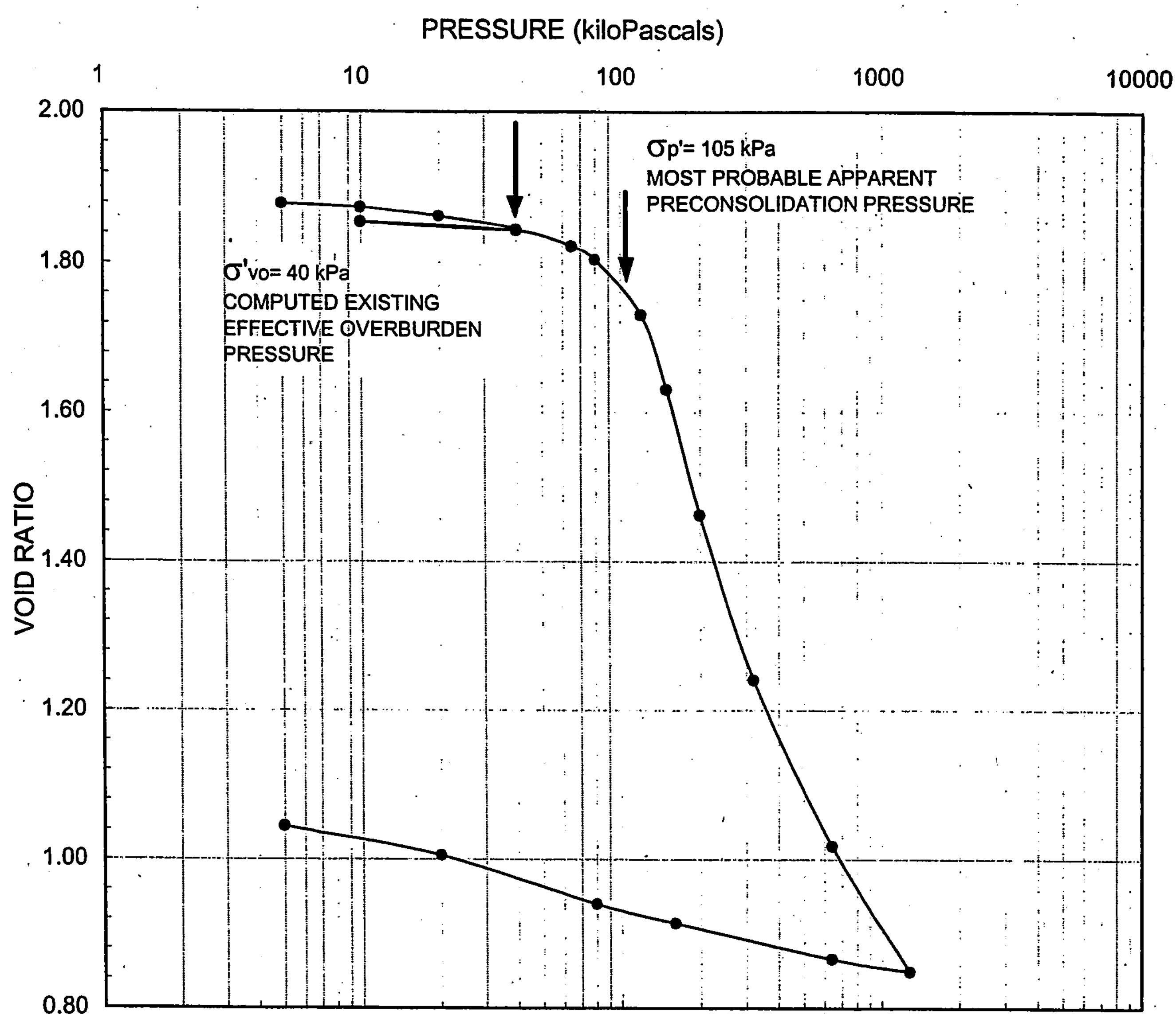
DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100

SHEAR STRENGTH P.S.F.
o UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB. VANE
500 1000 1500 2000 2500



LIQUID LIMIT
PLASTIC LIMIT
WATER CONTE
W_P W
WATER C
25

APPENDIX B

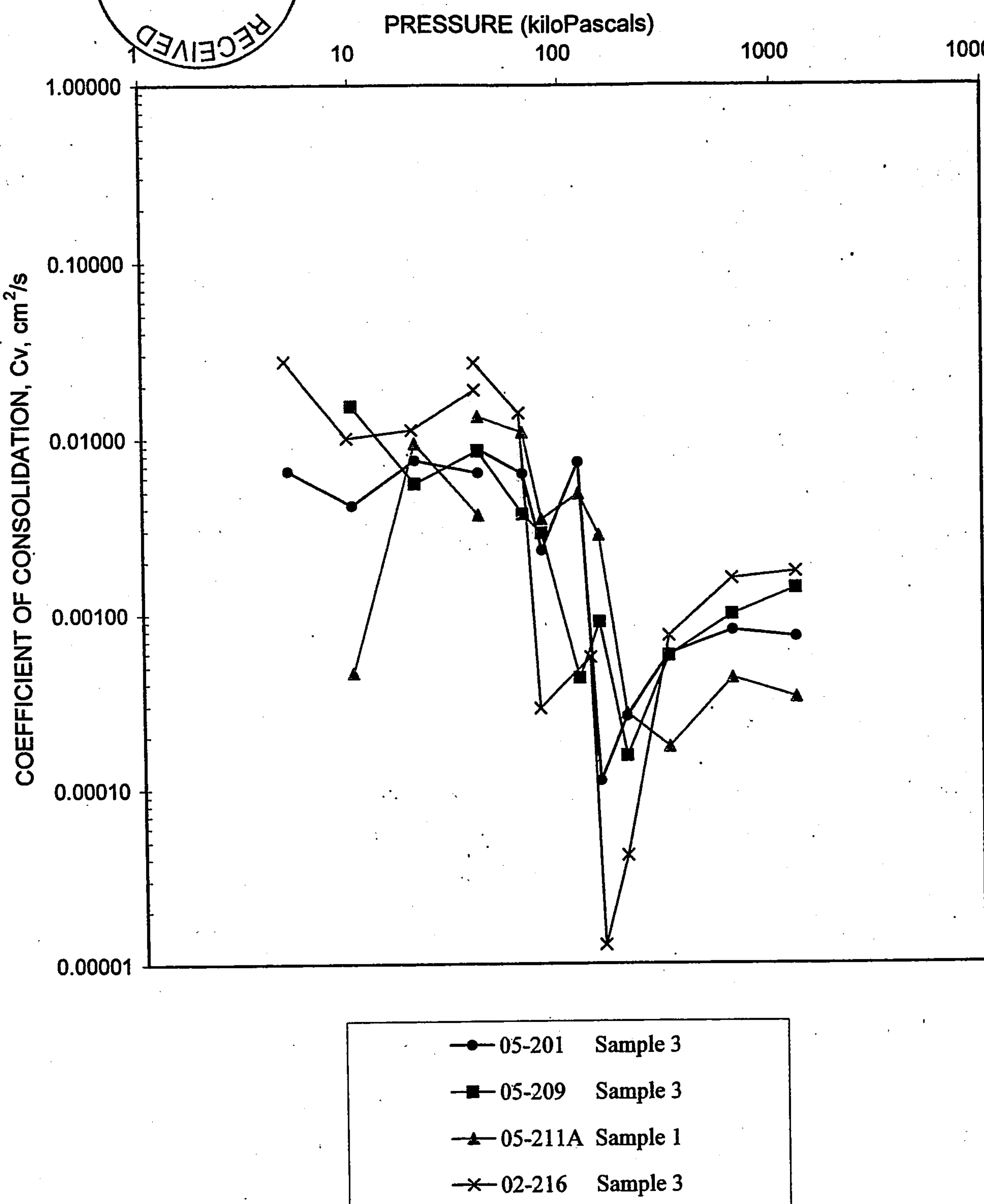


LEGEND

Borehole: 05-201	w_i = 67.5%	S_o = 98%
Sample: 3	w_f = 40.0%	C_c = 1.35
Depth (m): 3.40	w_l = 58	C_r = 0.018
	w_p = 23	

Golder Associates

SCALE	AS SHOWN	TITLE: CONSOLIDATION TEST RESULTS
DATE	05/03/05	
DESIGN	NA	
CADD	NA	
FILE No. Consolidation summary	CHECK EWK	
PROJECT No. 04-1120-013 REV. 0	REVIEW MIC	FIGURE 4



Golder Associates	SCALE: AS SHOWN	TITLE	
	DATE: 05/03/05	**Summary of Coefficient of Consolidation**	
	DESIGN: NA		
	CADD: NA		
FILE No.: Consolidation summary	CHECK: EWK		
PROJECT No.: 04-1120-013 · REV. 0	REVIEW: MIC		FIGURE 11