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FOUNDATION INVESTIGATION AND DESIGN REPORT

REHABILITATION OF NORTH EMBANKMENT AND APPROACH
HIGHWAY 140 / CNR OVERPASS
PORT COLBORNE, ONTARIO

MINISTRY OF TRANSPORTATION, ONTARIO

Submitted to:
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REPORT

GEOCRES No. 30L14-50

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Table of Contents

LIST OF DRAWINGS

LIST OF TABLES

LIST OF APPENDICES

PART A

FOUNDATION INVESTIGATION REPORT
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August 2009
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Ministry of Transportation Ontario (MTO) to provide foundation engineering services for the investigation and design of remedial measures to address the historical distress and poor performance experienced on the existing embankment of Highway 140 north of the CNR overpass structure, in Port Colborne, Ontario. The section of roadway experiencing distress includes the immediate approach and the embankment section extending approximately 300 m north of the bridge.

The terms of reference for the scope of work are outlined in Golder's proposal P81-1416 dated July 2008 that forms part of the Consultant's Agreement (Agreement No. 2008-E-0013) for this project. The work was carried out in accordance with the Quality Control Plan for this project dated September 2008.

Geotechnical investigations were carried along the north embankment by the MTO in the 1960's and 1970's prior to and following construction of the existing bridge structure and approach embankments. The purpose of the current investigation by Golder is to complement the previous investigations at the site to provide information on the existing embankment composition as well as on the subsurface foundation stratum to aid in the assessment of the cause(s) of the instabilities and to facilitate the development of embankment stabilization/slope treatment recommendations.

This report addresses only the geotechnical issues associated with the remediation of the north embankment to the Highway 140/CNR overpass structure.

2.0 SITE DESCRIPTION

The site of the Highway 140/CNR overpass structure, approaches and embankments is located approximately 500 m north of Forkes Road East in Port Colborne, Ontario. The key plan on Drawing 1 provides an overview of the site location.

The terrain in the area directly adjacent to Highway 140 in the vicinity of the project site is flat farmland, with poor surficial drainage, with a ground surface elevation of about 177 m, referenced to Geodetic datum. At the location of the CNR overpass structure, Highway 140 is a two-lane road with a posted speed limit of 80 km/h.

Based on our review of the available Geocres information and discussions with MTO, it is our understanding that the existing north approach and embankment were constructed in May 1971 out of locally available material excavated from borrow pits located on the east side of the highway (i.e. in the area of the existing storm water retention pond). At present, at its highest point, the north embankment is approximately 9.5 m tall at the north approach to the bridge, and slopes downwards to the north to a height of less than 2 m at a distance of about 320 m from the bridge. A three-span concrete bridge, with abutments founded on piles driven to bedrock and piers founded on shallow spread footings, crosses the CNR tracks at the south end of the north embankment.

3.0 INVESTIGATION PROCEDURES

The investigation for the rehabilitation of the north approach and embankment to the Highway 140/CNR overpass structure included the following key components:

- Desktop study/review of available background information from MTO Geocres;
- Site visit/field reconnaissance and meeting with MTO Area Maintenance Coordinator; and
- Field borehole drilling and test pit investigation.

The desktop study and review of background information available from the MTO's Geocres library was carried out in mid-September 2008. The details of the previous investigations conducted at the site are summarized in Geocres Reports 30L 14-036 and 30L 14-045 which are provided in Appendix D of this report.

The initial site visit and field reconnaissance was performed by Golder on September 23, 2008. During the site visit, two members of Golder's engineering staff met with Mr. Brian Minor, the MTO Maintenance Coordinator for Central Region Operations, Niagara, to discuss the embankment construction, the history of the ongoing slope stability problems and the type of maintenance/repairs performed at the site. During this time, Golder also examined the current condition of the embankment including zones of surficial sloughing on the side slopes, tension cracks near the slope crests, deformation of the guide rail, padding on the roadway surface and vegetation on the slopes. The record of the locations of these zones and features were mapped onto a sketch plan of the north embankment together with details of the changes in the existing slope geometry, size of berms and general site conditions.

The subsurface drilling and test pitting investigation was carried out by Golder along the north embankment of Highway 140 between September 29 and October 9, 2008. During this time, four (4) boreholes (08-1 to 08-4) were advanced at the site using a truck-mounted CME-75 drill rig supplied and operated by Aardvark Drilling Inc. of Guelph, Ontario. In addition to the boreholes, eight (8) shallow test pits (TP-1 to TP-8) were excavated into the side slopes of the embankment using a CAT, track-mounted mini excavator supplied and operated by Roadside Rentals Inc. of Allenburg, Ontario.

The boreholes were advanced using 108 mm inside diameter continuous flight hollow-stem augers, to depths of between about 11.3 m and 37.2 m below the existing ground surface/top of roadway. Soil samples were obtained at 0.75 m and 1.5 m intervals of depth, using a 50 mm outside diameter (O.D.) split-spoon sampler, in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586-99), and using 76 mm O.D thin-walled 'Shelby' Tube samplers (ASTM D1587-00) to obtain relatively undisturbed samples in the cohesive soils. Field vane shear tests were conducted using an MTO 'N'-vane in cohesive soils for assessing undrained shear strengths (ASTM D 2573-01). The results of the in situ tests are shown on the Record of Borehole sheets in Appendix A.

The test pits were excavated into the embankment side slopes to depths of between about 1.2 m and 1.7 m below the existing ground surface. Bulk soil samples were obtained at selected depths using a hand shovel and 76 mm O.D thin-walled 'Shelby' Tube samplers were pushed into the bottom of some of the test pits to obtain relatively undisturbed samples of the cohesive soils. Field vane shear tests were conducted in the side walls of the test pits using a Roctest M-3 hand vane to assess undrained shear strengths of the shallow cohesive soils below the embankment side slopes. In-situ density testing with a Troxler nuclear density gauge was also performed at selected depths in the test pits to assess the in situ density and water content of the fill materials. The results of the in situ tests are shown on the Field Test Pit Logs in Appendix B.

Groundwater conditions in the open boreholes and test pits were observed during the field investigation and piezometers were installed in boreholes 08-1, 08-2 and 08-3 to allow monitoring of the groundwater levels at these locations. The piezometers consisted of 46 mm diameter PVC pipe, with a slotted screen surrounded by a sand filter and sealed at a select depth within the boreholes. Groundwater level observations, piezometer installation details and water level readings are shown on the Record of Borehole sheets and Field Test Pit Logs. It should be noted that groundwater levels as encountered in the open boreholes and test pits during the investigation may not be representative of static conditions since the groundwater levels may not have stabilized on completion of drilling/excavating. Furthermore, groundwater elevations will vary depending on seasonal fluctuations, precipitation and local soil permeability. Upon completion of the drilling operations all of the boreholes and test pits were abandoned in accordance with Ontario Regulation 903 (as amended by O. Reg. 372).

The field work was monitored on a full time basis by a member of our engineering staff who arranged for the clearance of underground utility services, directed and/or carried out the sampling and in situ testing operations, logged the boreholes and test pits and examined and cared for the soil samples. The soil samples were identified in the field, placed in appropriate containers, labelled and transported to our Mississauga geotechnical laboratory where the samples underwent further detailed visual examination and laboratory testing. Index and classification testing consisting of water content determinations, Atterberg limits, grain size distributions and specific gravity tests were carried out on samples of the embankment fill and overburden soils. In addition, standard Proctor maximum dry density tests and specialized strength and deformation testing including direct shear, one-dimensional consolidation (oedometer) and triaxial testing, were carried out on selected soil samples. All of the laboratory tests were carried out to MTO and/or ASTM Standards as appropriate.

The boreholes and test pits were laid-out in the field by Golder and the completed locations were surveyed by Chambers and Associates Surveying Ltd., a registered Ontario Land Surveyor. The borehole and test pit locations (including MTM NAD83 northing and easting coordinates) and ground surface elevations (referenced to geodetic datum) are shown on Drawing 1 and summarized below.

Borehole/Test Pit Number	***MTM NAD83 Northing (m)***	***MTM NAD83 Easting (m)***	***Ground Surface Elevation (m)***
Borehole 08-1	4756576.3	645972.2	186.7
Borehole 08-2	4756645.0	645994.6	185.5
Borehole 08-3	4756778.5	646032.6	181.8
Borehole 08-4	4756765.0	646036.6	182.2
Test Pit TP-1	4756781.3	646024.8	179.3
Test Pit TP-2	4756648.8	645985.8	182.3
Test Pit TP-3	4756562.9	645958.0	183.6
Test Pit TP-4	4756624.4	645977.4	182.8
Test Pit TP-5	4756564.2	645990.6	183.3
Test Pit TP-6	4756588.1	645997.6	183.0
Test Pit TP-7	4756626.0	646005.9	182.9
Test Pit TP-8	4756764.9	646048.2	178.6

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The project site is located within the physiographic region known as the Haldimand clay plain. Subsoils in this physiographic region generally consist of glacial lacustrine deposits of silts and clays over a thin layer of glacial till underlain by dolomite limestone bedrock (Chapman, L.J. and Putnam, D.F., 1984).

4.2 Subsurface Conditions

The detailed subsurface soil and groundwater conditions encountered in the boreholes and test pits advanced during this investigation, together with the results of the in situ and laboratory index tests are provided on the

Record of Borehole sheets and Field Test Pit Logs presented in Appendix A and B, respectively. The results of all of the laboratory testing are presented on Figures C1 to C12 in Appendix C. 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. The subsoil conditions will vary between and beyond the borehole locations. The inferred soil stratigraphy at the site, based on the results of the previous and current investigations, is shown on Drawing 1.

In summary, the new boreholes were advanced from the top of the embankment, through the roadway and encountered approximately 0.2 m of asphalt, 0.6 m of sand and gravel fill overlying the clay to silty clay embankment fill. The embankment fill ranged in thickness from about 5.9 m to 9.0 m at the investigated locations. Beneath the embankment fill, the boreholes encountered an upper silty clay to clay crust, underlain by a lower clayey silt to silty clay, overlying a clayey silt to sandy silt till. The test pits were excavated into the side slopes of the embankment above the top of the toe berm (where present), and encountered the silty clay to clay embankment fill overlain by up to as much as 1 m of sand and gravel fill at some locations.

A detailed description of the subsurface conditions encountered in the boreholes and test pits is provided in the following sections.

4.2.1 Topsoil

A surficial layer of topsoil, ranging in thickness from about 50 mm to 100 mm, was encountered at the ground surface on the side slopes of the embankment in test pits TP-2 to TP-7.

4.2.2 Asphalt

An approximately 200 m thick layer of asphalt was encountered at the ground surface in all of the boreholes advanced from the roadway surface at the site. The ground surface at the borehole locations varied from about Elevation 181.8 m to 186.7 m.

4.2.3 Sand and Gravel to Gravelly Sand Fill

Granular fill materials were encountered at the ground surface in TP-1, underlying the topsoil in TP-3, TP-5 and TP-6 and underlying the asphalt in all of the boreholes advanced from the roadway surface during the current investigation.

The granular fill underlying the asphalt forms the base course for the pavement structure of the highway and is about 0.6 m thick and comprised of a brown and grey, sand and gravel, trace to some silt. The Standard Penetration Test (SPT) ‘N’ values measured within the sand and gravel fill ranged from 24 blows per 0.3 m of penetration to greater than 50 blows per 0.07 m of penetration, indicating that the fill has a compact to very dense relative density. The natural water content measured on select samples of the granular fill underlying the asphalt varied from about 1.4 to 4.5 percent.

The granular fill encountered at the ground surface on the embankment side slopes at some of the test pit locations is a dry to moist, grey, sand and gravel, trace to some silt, trace clay to sandy gravel, trace silt, trace clay. The sand and gravel fill on the side slopes, where present, ranged in thickness from about 0.15 m in TP-3 (where it was found on the up-slope side of the test pit only), to as much as 1.1 m thick in TP-1.

The natural water content measured in the laboratory on select samples of the granular fill material from the test pits varies from about 0.2 to 3.7 percent. In situ density testing carried out on the sand and gravel fill in the test pits (with a nuclear density gauge) measured dry densities ranging from about 1600 kg/m^3 to 1950 kg/m^3 with an average value of about 1800 kg/m^3. The in situ nuclear density tests also measured water contents on the granular fill ranging from about of 2.1 to 8.9 percent with an average value of about 4.9 percent.

The results of grain size distribution testing performed on two samples of the granular fill encountered in the test pits are shown on Figure C1 in Appendix C.

Laboratory consolidated drained direct shear (DS) tests were carried out on one selected sample of the sand and gravel fill from test pit TP-5. The details of the test results are shown on Figure C2 in Appendix C. The results of the direct shear test are summarized below.

Test Pit/Sample Number	Depth (m)	Effective Cohesion Intercept, c' (kPa)	Effective Angle of Internal Friction, ϕ' (degrees)
TP-5 / Sa #1	0.3	0	50

Note: assessed shear strength parameters are only valid over range of stress conditions in test.

4.2.4 Clay Fill

Clay fill was encountered in all of the boreholes and test pits advanced at the site, either at the ground surface, or underlying the topsoil and/or granular fill, where present. All of the test pits were terminated within the clay fill, and all of the boreholes were advanced at least 3 m below the bottom of the clayey embankment fill/top of original ground surface which was encountered at elevations between about 175.7 m and 177.7 m.

The top of the clay fill underlying the pavement structure was encountered at elevations ranging from about 181.0 m to 185.9 m and the thickness of the fill ranges from about 5.1 m to 8.2 m at the borehole locations. The clay fill material used to construct the embankment was sourced from a local borrow pit immediately adjacent to the site which now forms a storm pond on the east side of the existing Highway 140. The fill as encountered at the borehole and test pit locations is generally comprised of a brown, clay , some silt, trace sand, trace gravel. The clay fill was found to contain organics near the base of the fill/just above the original ground surface.

The Standard Penetration Test (SPT) 'N' values measured within the clay fill at the borehole locations (ie. within the core of the embankment) ranged from 7 to 21 blows per 0.3 m of penetration suggesting a firm to very stiff consistency. In situ field vane testing in the boreholes carried out within the clay fill measured undrained shear strengths of greater than 120 kPa indicating a very stiff consistency.

In situ field vane testing carried out with an M-3 hand vane in clay fill encountered in the test pits located on the side slopes of the embankment measured undrained shear strengths ranging from about 16 kPa to 70 kPa indicating a soft to stiff consistency, but with an average value of about 35 kPa, indicating a generally firm consistency.

The natural water content measured in the laboratory on select samples of the clay fill material from the boreholes and test pits varies from about 17 to 31 percent with an average of about 25 percent. In situ density testing carried out on the clay fill in the test pits (with a nuclear density gauge) measured dry densities ranging from about 1450 kg/m^3 to 1600 kg/m^3 with an average value of about 1500 kg/m^3. The in situ nuclear density tests also measured water contents on the clay fill ranging from about of 21 to 28 percent with an average value of about 24 percent.

Grain size distributions for three (3) samples of the clay fill are shown on Figure C3 of Appendix C. Atterberg limits testing was carried out on twelve (12) samples of the clay fill. The liquid limit generally ranges from about 50 to 63 percent and the plastic limit ranges from about 21 to 24 percent, yielding a plasticity index ranging from about 30 to 40 percent. The results of the Atterberg limits testing are shown on the plasticity chart on Figure C4 in Appendix C, indicating that the fill is predominantly a clay of high plasticity.

Laboratory consolidated, drained direct shear (DS) tests were carried out on two (2) samples of the clay fill from test pit TP-8. In total, 2 sets of 3 specimens were tested. One sample tested (TP-8, Sa#2) was a relatively 'undisturbed' Shelby tube sample, while the other sample tested (TP-8, Sa#1) was a bulk sample that was recompacted in the laboratory to the average measured in situ dry density (1500 kg/m^3) and water content (24%) prior to testing. During the tests, both the peak and residual strengths were measured. The details of the test results are shown on Figure C5 and C6 in Appendix C. The results of the direct shear tests are summarized below.

Test Pit/Sample Number	Depth (m)	Peak		Residual	
		Effective Cohesion Intercept, c' (kPa)	Effective Angle of Internal Friction, ϕ' (degrees)	Effective Cohesion Intercept, c' (kPa)	Effective Angle of Internal Friction, ϕ' (degrees)
TP-8 / Sa #2 Undisturbed	0.8	7	28	2	28
TP-8 / Sa #1 Recompacted	0.5	4	34	0	34

Note: assessed shear strength parameters are only valid over range of stress conditions in test.

Laboratory consolidated, undrained triaxial compression tests (CIU) with pore pressure measurement were carried out on three (3) samples of the clay fill from boreholes 08-2, 08-3 and from test pit TP-3. In total, 1 set of 3 specimens and 2 sets of 2 specimens were tested. All samples tested were relatively 'undisturbed' Shelby tube samples. The details of the test results are shown on Figure C7, C8 and C9 in Appendix C. The results of the triaxial tests are summarized below.

Borehole or Test Pit / Sample Number	Depth (m)	Effective Cohesion Intercept, c' (kPa)	Effective Angle of Internal Friction, ϕ' (degrees)
08-2 / Sa #7	6.4	0	30
TP-3 / Sa #3	1.3	4	29
08-3 / Sa #3	1.8	3	29

Note: assessed shear strength parameters are only valid over range of stress conditions in test.

The samples tested in the direct shear and triaxial tests were consolidated to pressures representative of the estimated in situ effective stresses (under dry and saturated conditions) at the respective sample depths. Note that the interpreted effective strength parameters provided above are applicable only to design situations for which the stress conditions during testing are representative (i.e. in this case, for relatively low confining stresses at shallow depth on the side slopes of the existing embankment). Reference should be made to individual test reports for details of the testing conditions. Additional discussion regarding the measured strength parameters and how they have been combined and employed in the numerical analyses carried out for this project is provided in a subsequent section of this report.Two standard Proctor maximum dry density tests were performed on samples of the clay fill from TP-2 and TP-4. The results indicate that the maximum dry density for this soil is about 1550 kg/m^3 with an optimum moisture content of about 25 percent. The results of the standard Proctor test are shown on Figures C10 and C11 in Appendix C.

Laboratory consolidation (oedometer) testing was carried out on two (2) specimens of the clay fill material obtained from Shelby tube samples in Boreholes 08-1 and 08-2 to assess the compressibility characteristics of the fill materials. One sample was located at a shallow depth in the embankment fill (at about 1.8 m below top of roadway) and the other sample was located deeper in the embankment fill (at about 6.1 m below top of roadway). Bulk unit weights of about 19.9 kN/m^3 and 19.3 kN/m^3 and a specific gravity of about 2.8 were measured on each of the consolidation test specimens. The details of the test results are shown on Figures C12 and C13, and are summarized below.

Borehole / Sample Number	Sample Depth / Elevation	σ_{vo}' (kPa)	σ_p' (kPa)	C_c*	C_r+	e_o	c_v* (cm^2/s)
08-1 Sa #3	1.8 m / 184.9 m	35	(see below)	.066	.012	.721	1.6×10^{-2}
08-2 Sa #7	6.4 m / 179.1 m	125	(see below)	.086	.011	.812	2.2×10^{-1}

Note: * For stress range of 20 kPa ≤ σ_V' ≤ 185 kPa
+ For stress range of 5 kPa ≤ σ_V' ≤ 20 kPa

where: σ_{vo}' is the vertical effective overburden stress (in kPa)
σ_p' is the preconsolidation stress (in kPa) – not applicable
e_o is initial void ratio
C_c is the compression index
C_r is the recompression index
cv is the coefficient of consolidation (in cm^2/s)

It should be noted that based on the geologic history of the clay embankment fill (i.e. 'young', recompacted sediments), the term preconsolidation pressure has little meaning and cannot be logically defined for these materials based on the laboratory test results.

4.2.5 Upper Silty Clay to Clay (Crust)

An upper deposit/crust of brown, silty clay to clay, trace sand, trace gravel was encountered below the embankment fill in all of the boreholes advanced as part of this investigation. Boreholes 08-1, 08-2 and 08-3 penetrated through the upper silty clay to clay deposit and Borehole 08-4 was terminated within this deposit. The top of the upper silty clay to clay crust/original ground surface was encountered at elevations ranging from about 175.7 m to 177.7 m and the thickness of the crust varies from about 5.3 m to 9.9 m.

The Standard Penetration Test (SPT) 'N' values measured within the upper silty clay to clay crust typically range from 8 to 26 blows per 0.3 m of penetration, indicating a stiff to very stiff consistency. In situ field vane testing carried out within the silty clay to clay crust measured undrained shear strengths values of greater than 120 kPa indicating a very stiff consistency.

The natural water content measured on select samples of the upper silty clay to clay crust typically varies from about 19 percent to 36 percent with an average value of 26 percent. Atterberg limits testing was carried out on a total of fifteen (15) samples of the upper silty clay to clay during the previous (1960's and 1970's) investigations by MTO and the current investigation. The liquid limit generally ranges from about 36 to60 percent and the plastic limit ranges from about 18 to 26 percent, yielding a plasticity index ranging from about 15 to 35 percent. The results of the Atterberg limits testing are shown on the plasticity chart on Figure C14 in Appendix C, indicating that this upper crust is predominantly a silty clay to clay of moderate to high plasticity.

4.2.6 Lower Clayey Silt to Silty Clay

A lower deposit of brown, clayey silt to silty clay, trace sand, trace gravel was encountered below the upper silty clay to clay crust in Boreholes 08-1, 08-2 and 08-3. The top of the lower clayey silt to silty clay was encountered between about Elevations 167.8 m and 170.4 m. The lower clayey silt to silty clay was not fully penetrated in

Borehole 08-2; however, it was fully penetrated in Boreholes 08-1 and 08-3 where its thickness varied from about 3.2 m to 14.6 m.

The Standard Penetration Test (SPT) 'N' values measured within the lower clayey silt to silty clay deposit typically range from 2 to 8 blows per 0.3 m of penetration, indicating a soft to firm consistency. In situ field vane testing carried out within the lower clayey silt to silty clay measured undrained shear strength values ranging from about 57 kPa to greater than 120 kPa with an average value of about 76 kPa indicating a generally stiff consistency.

The natural water content measured on select samples of the lower clayey silt to silty claytypically varies from about 10 percent to 47 percent with an average value of about 29 percent. Atterberg limit testing was carried out on a total of fourteen (14) samples of the lower clayey silt to silty clay during the previous (1960's and 1970's) investigations by MTO and the current investigation. The liquid limit generally ranges from about 18 to 64 percent and the plastic limit ranges from about 11 to 31 percent, yielding a plasticity index ranging from about 6 to 38 percent. The results of the Atterberg limits testing are shown on the plasticity chart on Figure C15 in Appendix C, indicating that this lower deposit is predominantly a clayey silt to silty clay of low to intermediate plasticity.

Laboratory consolidation (oedometer) testing was carried out on two (2) specimens of the lower clayey silt to silty clay deposit obtained from Shelby tube samples in Borehole 08-1. Preconsolidation pressures of about 275 kPa and 385 kPa were estimated from the void ratio versus logarithmic pressure plots and from the total work versus pressure plots. A bulk unit weight of about 19.5 kN/m^3 was measured on both specimens and a specific gravity between about 2.76 and 2.79 were measured on the two samples. The details of the test results are shown on Figures C16 and C17, and are summarized below.

Borehole / Sample No.	Sample Depth / Elevation	σ_{vo}' (kPa) (Pre)$^+$	σ_{vf}' (kPa) (Post)$^+$	σ_p' (kPa)	OCR (Post)	C_c	C_r	e_o	C_v^* (cm^2/s)	$C\alpha_{(\varepsilon)}$ %
08-1 Sa #17	20.1 m / 166.6 m	160	315	300	≈1.0	.24	.054	0.70	2.5×10^{-2}	.22
08-1 Sa #21	27.7 m / 159.0 m	260	390	400	≈1.0	.38	.075	0.86	3.4×10^{-3}	.20

Note: * For stress range of 150 kPa ≤ σ_v' ≤ 300 kPa
$^+$ 'Pre' – implies before 1960 (prior to existing embankment construction)
'Post' – implies after end of primary consolidation due to existing embankment construction

where: σ_{vo}' is the vertical effective overburden stress (in kPa)
σ_p' is the preconsolidation stress (in kPa)
OCR is overconsolidation ratio (note: estimated to be about 1.0 for the final stress conditions)
e_o is initial void ratio
C_c is the compression index
C_r is the recompression index
c_v is the coefficient of consolidation (in cm^2/s)

It is noted that the interpretation of preconsolidation stress is difficult for these samples due to the rounded nature of the void ratio-log effective stress curves. Given this, the values of preconsolidation stress were assessed primarily based on the 'Work'-method proposed by Becker et al. (1987).

4.2.7 Clayey Silt to Silt (Till)

A layer of brown, clayey silt, some sand, some gravel to silt, some sand, some gravel, some clay (till), was encountered below the lower clayey silt to silty clay in Boreholes 08-1 and 08-3. The top of the clayey silt to silt till was encountered between about Elevation 153.2 m and 167.2 m and its thickness varies from about 2.2 m to 3.1 m.

Two Standard Penetration Test (SPT) 'N' values measured on the clayey silt to silt till deposit were 31 and 89 blows per 0.3 m of penetration, indicating a dense to very dense relative density.

Two natural water contents measured on select samples of thisdeposit were about 10 percent.

4.2.8 Sandy Silt to Sand (Till)

A deposit of brown to grey, sandy silt, some gravel, trace to some clay to sand, some silt, trace gravel (till) was encountered below the clayey silt to silt till layer in Boreholes 08-1 and 08-3. The top of the sandy silt to sand till was encountered between about Elevation 150.1 m and 165.0 m. Borehole 08-1 and 08-3 were terminated within this deposit without penetrating it at depths of 37.2 m and 17.4 m (elevation 149.5 m and 164.4 m) respectively.

Two Standard Penetration Tests (SPT) 'N' values measured on the sandy silt to sand till deposit were 36 blows per 0.3 m of penetration and 50 blows per 0.05 m of penetration, indicating a dense to very dense relative density.

4.2.9 Groundwater Conditions

The groundwater levels were observed in the open boreholes and tests pits during and upon completion of the drilling/excavation operations, and are provided on the Record of Borehole Sheets and the Field Test Pit Logs included in Appendix A and B, respectively. Piezometers were installed at the site in Boreholes 08-1, 08-2 and 08-3 to monitor changes in the groundwater level following completion of drilling. Details of the piezometer installations are shown on the Record of Borehole Sheets. Water levels as measured in the piezometers are shown on the Record of Borehole Sheets and are summarized below.

Borehole No.	Ground Surface Elevation (m)	Strata containing Piezometer Tip	Water Level Depth (m)	Water Level Elevation (m)	Date
08-1	186.7	Clayey Silt to Silt (Till)	19.2	167.5	October 21, 2008
08-2	185.5	Clay Fill	Dry to 7.6	<177.9	October 21, 2008
08-3	181.8	Clayey Silt to Silt (Till)	14.4	167.4	October 21, 2008

Based on the piezometer installations and the measured readings indicated above, it appears that there is likely little excess pore pressure within the clay embankment fill and that the total head within the clayey silt to silt till (underlying the native clayey silt to clay stratum) is at above Elevation 167.5 m (just below the base of the upper silty clay to clay crust). However, it should be noted that water levels observed in piezometers installed at the site by MTO in 1968 and 1972 (Geocres Reports No. 30L 14-036 and 30L 14-045) in the upper silty clay to clay stratum indicate an upper groundwater table is present at the site at about Elevation 176.0 m (i.e. about 1.5 m below the original ground surface) which appears to be consistent with the observed water level in the storm water retention pond located immediately east of the north embankment and approach. This information combined with the water levels observed in the recently installed piezometers indicates that a downward pore pressure gradient likely exists at the site, varying from about Elevation 176.0 m to 167.5 m across the thickness of the native clayey silt to clay stratum.

The test pits excavated within the side slopes of the upper clay embankment fill were mostly dry upon completion of excavation to the depths noted on the Test Pit logs. Water seepage was only observed in test pit TP-3 at a depth of about 1 m on the date of excavation.

It should be noted that the groundwater levels are subject to seasonal fluctuations, where typically higher groundwater levels may be present during the spring months and at times of sustained heavy rainfall.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Mr. Matthew Kelly, EIT., with technical input from Mr. Murty Devata, P.Eng., and reviewed by Mr. J. Paul Dittrich, Ph.D., P.Eng., a senior geotechnical engineer and Principal with Golder. Mr. Fin Heffernan, P.Eng., Golder's Designated MTO Contact for this project, conducted an independent quality review of the report.

Report Signature Page

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PART B

FOUNDATION DESIGN REPORT

HIGHWAY 140/CNR OVERPASS

NORTH EMBANKMENT AND APPROACH

MINISTRY OF TRANSPORTATION, ONTARIO

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides engineering design recommendations for the remediation of the Highway 140/CNR overpass structure north embankment and approach. The recommendations are based on:

- a review of the available background information regarding the original construction and from the previous foundation investigations at the site;
- the observations from the site visit/field reconnaissance and discussions with the MTO Area Maintenance Coordinator, as well as an assessment of the performance of previous remediation efforts at the site; and
- an interpretation of the factual data obtained from the previous and current field and laboratory investigation programs, along with back analysis of the performance of the existing embankment and analysis of the proposed remedial alternatives.

The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible remediation alternatives and to carry out the design of the remedial measures for the north embankment and approach. Where comments are made on construction, they are provided in order to highlight those aspects which could affect the design of the project. Those requiring information on the aspects of construction should make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 Site History

The following summarizes the history of the embankment performance at the site along with a chronology of the key events based on our review of the relevant available information in the MTO Geocres system and discussions with MTO.

- **October to November 1968** – MTO performs a foundation investigation for design of the bridge structure foundations, approaches and associated embankments.
- **May 1971 -** the north and south embankments and approaches to the Highway 140/CNR overpass structure are constructed (up to about 9.5 m in height with 2H:1V side slopes) using locally available native clay soil excavated from borrow sources immediately adjacent to the highway alignment and located to the north and south of the CN rail tracks. Visual observations of the borrow sources indicated that the clay material from the south pit (used to construct the south embankment) appeared to be of a higher moisture content than optimum. Construction of the north embankment commences on May 4, 1971, followed by construction of the south embankment which commences on May 21, 1971. Based on visual observations from MTO personnel during the construction, it was noted that the fill was placed directly on the existing terrain (topsoil was not removed); the surface drainage in the vicinity of the embankments was generally poor at the time of placement (with numerous areas of ponded surface water); the fill material placed in the lower portion of the embankment along the south approach appeared to have a higher natural water content making compaction difficult. Subsequent notes from this time suggest that the embankment fills were placed during unfavourable/wet weather conditions.
- **July 5, 1971** – major instability occurs on a portion of the side slopes of the south embankment and approach when the embankment is within about 1.2 m of its final grade. The section of slope failure is approximately 150 m in length and the failure is described as consisting of as much as 0.6 m of subsidence at the crest, longitudinal tension cracks up to 1 m wide opening with the main ‘body’ of the embankment (from 1.5 m to 9 m on either side of the embankment centreline) and bulging at the toes of the embankment fill by as much as 1 m beyond the original geometry.
- **July 9, 1971** – inspection of the instability is carried out by MTO (Mr. M. Devata) including examination of several test pits through the failed area. A soft, thin layer (about 0.3 m thick) of cohesive organic material is identified in the test pits at the contact of the fill material and natural subsoil. Further, the

tension cracks observed at the surface of the failed roadway are reported to extend down to the soft organic layer at the original ground surface. Water seepage into test pit excavations on the side slopes of the embankment is noted at one location.

- **August 30, 1971** – instability occurs on the side slopes of the full height north embankment. The degree of distress is reportedly less than the failure on the south slope.
- **September 14, 1971** – south embankment is remediated by constructing approximately 6 m to 10 m wide mid-height berms over a length of about 175 m. A portion of the original surficial organic material (to a depth of about 0.6 m) was also to be removed over the footprint of the remediated area and in particular below the toe area of the new berm.
- **September 22, 1971** – north embankment is remediated in a similar manner to that carried out on the south embankment by constructing approximately 6 m to 10 m wide mid-height berms on both side slopes. The length of the berms at this time is unknown but it is inferred that the berm on the east side is longer than that constructed on the west side.
- **October 1, 1971** – major instability occurs on the west side slope of the south embankment and approach. The location of the failure is unknown, but presumed to be beyond the previously remediated area (i.e. beyond the limits of the mid-height berm). The extent of the subsidence, tension cracks and toe bulging are reportedly very similar to those that occurred in July 1971. It is presumed that this new area of instability was subsequently remediated in a similar manner to that described above.
- **January 1972** – instability occurs on the side slopes of the north embankment. The extent of the failure(s) is less severe than the previous instability and it is reported that the west side of the embankment showed more distress that the east. It is presumed that this new area of instability was subsequently remediated in a similar manner to that described above.
- **January 1973** – site visits carried out by MTO to assess performance of remediated embankments. Tension cracks running parallel to the roadway are observed on the upper slopes of the embankments (above the mid-height berms). Localized surficial sloughing is evident on the 2H:1V upper portions of the slope above the berm, in particular on the east and west sides of the north embankment. Seeding and mulching applied in Fall of 1972 reportedly did not hold in place due to lack of root development. During this time, site visits to several nearby embankments associated with the St. Lawrence Seaway Authority were also carried out; observations indicate that where embankments are constructed with the local silty clay, those with 3H:1V side slopes appeared stable, while those with 2H:1V side slopes show similar signs of surficial instability.
- **April 1973** – site visit carried out by MTO confirm similar observations to those in January, in particular, localized surficial sloughing evident in upper portion of embankment side slopes. In addition, subdrains in granular backfill behind abutment are not functioning.
- **December 1990** – site visit carried out by MTO to assess performance of embankments. Localized surficial sloughing evident on the portion of the upper 2H:1V slopes above the berms; in particular on the east and west sides of the north approach fill. The slope below the berm appears reasonably stable.
- **June 1991** – site visit carried out by MTO to assess performance of embankments. Tension cracks noted running parallel to the roadway along the shoulders on both sides of the upper slopes of the north embankment, extending about 200 m north of the bridge. Surficial sloughing noted on the upper slopes above the berm of the north embankment, particularly on the east side. Tension cracks also noted on the east side of the lower slopes along the edge of the berm, about 60 m in length and up to 20 cm wide. Localized circular failures (about 16 m wide) observed on the east and west side of the south abutment forward slope.
- **October 1991** – site visit carried out by MTO to assess performance of embankments. Tension cracks noted running parallel to the roadway along the shoulders of the upper slopes of the north embankment (extending about 260 m north of the bridge) and south embankment (extending about 250 m south of the bridge). Surficial sloughing noted on the upper slopes above the berm of the north embankment (over a section about 100 m long) and the south embankment (over a section about 25 m long). Settlements of

up to 80 cm have occurred associated with the surficial failure along the upper slope of the south approach. Tension cracks also noted on all of the lower slopes along the edges of the berms, Localized circular failures observed on the east and west sides of the south abutment forward slope (about 16 m wide) and on the east and west sides of the north abutment forward slopes (about 10 m wide). Localized slope failure has occurred on the lower slope of the east side of the north embankment fill, causing bulging at the toe of the slope.

- **1991 to 1997** – although not explicitly documented in the Geocres literature, it is believed that additional remedial measures in the form of placement of granular blankets may have been carried out at select locations of the slopes (most likely within the approach areas and/or on the embankment front slopes) during this time.
- **August 1997** – site visit carried out by MTO to assess performance of embankments. Slope distress and surficial movements noted to be confined to the areas above the berms and close to the top of slope. No instabilities were identified below the berms.
- **Mid-1998** – additional remedial measures carried out on south embankment (MTO Contract No. 98-116). Remediation is to include (based on contract drawings) flattening upper slopes to at least 3H:1V with Granular ‘A’, a 200 mm earth cap, 50 mm topsoil and seed and cover. Extent of remediation is up to about 200 m long on east slope and about 100 m long on west slope.
- **1998 to 2006** – based on discussion with MTO Area Maintenance Coordinator, following slope flattening at the south embankment in 1998, no additional remediation or maintenance has been required on this embankment and there have been no reports of any tension cracks, surficial instability or other signs of embankment distress. However, on the north embankment, on-going distress and poor performance has continued in the form of tension cracks, surficial instability and associated sloughing/ground loss near top of embankment side slopes above berm. These problems have required regular annual maintenance (once or twice per year) mostly in the form of granular fill placement (end-dumping and blading/spreading) to widen the shoulder area near the crest of the embankment slopes on both sides of the north embankment.
- **Mid-2006** – additional remedial measures carried out on the north embankment (MTO Contract No. 2006-2034). Remediation included excavation/re-grading of sloughed material on embankment side slopes (above berm), placement of topsoil, seed and cover, removal and replacement of guide rail, construct curb and gutter along edge of roadway and construct concrete outlets and rip-rap lined channels with geotextile at select intervals along slope face.
- **September 2006** – within about six (6) weeks of having completed Contract No. 2006-2034, localized surficial sloughing and settlement of the new guide rail was reported along several sections of the east and west sides of the north embankment and approach.
- **August 2008** – Golder selected to carry out Foundation Investigation and Design of Highway 140/CNR Structure North Approach Embankment to assess causes of the embankment distress and recommend remedial measures.

6.2 Assessment of Factors Affecting Embankment Performance

In order to develop remediation options to address the on-going poor performance of the north embankment it is first necessary to assess the potential cause(s) of the distress. This assessment has involved the following three main components:

- Review of available background information describing the history of the site;
- Field reconnaissance and discussions with MTO Area Maintenance Coordinator; and
- Identification of potential mechanisms causing distress.

The details of each of these components are described in the following sections.

6.2.1 Review of Background Information

As summarized in Section 6.1, there is a significant amount of information from the MTO documenting the history of construction, embankment performance problems and subsequent remediation carried out at the site. Based on a review of this information, it is clear that since almost the completion of original construction, the high clay fill embankments have been affected by a series of significant slope instabilities ranging from large crest-to-toe failures (with associated wide tension cracks and slope subsidence that extended well behind the embankment crests) to surficial sloughing confined to the near surface embankment slopes. There has also been various levels of remediation carried out at the site in response to the stability problems ranging from the construction of wide mid-height stability berms (at both embankments), granular blanket overlays (near the highest portions of the embankments within the approach areas and on the front slopes), application of seed and mulch, slope flattening with granular fill (at the south embankment only) and slope re-grading and drainage improvement measures including installation of curb-and-gutter and construction of rip-rap lined drainage channels down the slope face(s).

It would seem that the remediation efforts carried out to date have been most effective in stabilizing the larger portions of the embankment with little evidence of subsequent problems that continued to affect the main body or 'core' of the embankment, in particular after construction of the mid-height berms. However, based on the documented site observations by MTO, evidence of tension cracks and surficial sloughing on the upper embankment slopes of the embankments has persisted at the site since the early 1970's. Following the 1998 additional remediation of the south embankment by granular slope flattening, there have been no further documented observations of surficial slope instability. On the north embankment, however, surficial instability in the form of sloughing on the upper slope faces (above the berm), tension cracks near the embankment crest and deformations to the guide rail have occurred for the last 36 years and continue at present.

It is interesting to note that the observational evidence of the performance of other fill embankments in the Welland area constructed with the local clay sourced from the Haldimand clay plain (as documented by MTO) seems to suggest similar problems with surficial slope stability for embankments constructed with slope profiles steeper than 3H:1V.

6.2.2 Field Reconnaissance

On September 23, 2008, Golder visited the site to examine the current condition of the embankment and slopes and plan the details/locations of the field investigation program. During this time, a meeting and site walkover was held with Mr. Brian Minor, the MTO Maintenance Coordinator for Central Region Operations, Niagara who had been involved with the site since circa 1970. Mr. Minor provided valuable insight on the performance, frequency and type of maintenance on the embankments at the site and also gave details of the construction method(s) used by the Contractor during the 2006 remediation at the north embankment. Mr. Minor's insights are summarized as follows:

- Distress on the embankments is usually in the form of settlement of the fill materials near the embankment crests and associated movement (settlement and tilting) of the guide rails. However, slumping on the slope faces has also occurred.
- Maintenance has been required on a regular and annual or semi-annual basis at both embankments (north and south) during most of their design lives.
- Maintenance typically involves the placement (essentially end-dumping) of granular fill on the shoulders of the embankment, followed by 'blading' the fill to level it off with little to no compactive effort.
- Since the remediation (i.e. slope flattening) at the south embankment in or about 1998, no annual maintenance has been required and performance has been satisfactory.
- Prior to remediation (i.e. slope grading and installation of curb-and-gutter and guide rail replacement) at the north embankment in 2006, settlement and/or sloughing at the upper embankment crests had

become so severe that in some places the posts for the guide rail were no longer embedded in the embankment and the cables were suspending the posts.

- During the remediation in 2006, the Contractor's efforts on the re-grading of the slopes mostly involved moving/pushing the granular material (previously placed during remediation) that had settled and sloughed down the slope faces back up to the top of the slopes. Little to no compaction was carried out.

The observations at the north embankment side slopes as made by Golder during the field reconnaissance on September 23, 2008 are summarized as follows:

- Settlement of the granular material on the embankment shoulders, likely due to sloughing of the fill on the side slopes was noted in several areas (up to 70 m in length) on both the east and west sides of the embankment.
- Distress of the new guide rail (in the form of settlement and/or tilting) was also evident in most of the places where the sloughing was observed.
- A tension crack, approximately 10 m to 15 m in length was observed on the west side of the embankment at the crest of the slope behind the guide rail.
- Bulging at the toe of the upper slope was observed on the east side of the embankment.
- A depression (i.e. settlement) in the roadway surface was present just south of the bridge structure mostly in the SBL. A padding of asphalt has also been placed in this area. Vehicles approaching the bridge in the SBL showed clear signs of traversing a 'bump' or 'dip' before moving onto the bridge.
- Vegetation on the embankment sides slopes varied from heavy vegetation (including what appeared to be tall grasses, reeds and crown-vetch in the highest parts of the embankment) to typical grass (over most of the embankment slopes).
- Granular fill material (up to as much as about 1 m thick at the test pit locations) was present on the slope faces at some locations (mainly near the highest parts of the embankment – within the approach close to the bridge).

The locations of most of the observed areas of distress described above are shown on Drawing 1. Select photographs from the September 2008 site visit showing some of the typical types of distress observed at the north embankment are shown in Appendix E.

6.2.3 Identification of Potential Mechanisms

Based on review of the background information, the field reconnaissance and investigation, and discussions with MTO regarding the embankment performance, four main mechanisms were identified as potential contributors to the distress experienced at the north approach and embankment. They are as follows:

- Compression/settlement of the embankment fill
- Settlement of the foundation soils
- Global embankment stability
- Surficial embankment stability

An assessment of each of these mechanisms has been carried out and the details of the analysis, the results and the potential contribution or significance of each to the on-going distress are described in the following section.

6.3 Analysis of Potential Mechanisms

Using the results of the in situ testing and laboratory testing on samples from the field investigation, each of the identified mechanisms was analysed to assess the potential contribution to the historical and recently observed embankment distress.

6.3.1 Compression/Settlement of Embankment Fill

To estimate the magnitude of compression or settlement that may have occurred within the embankment fill itself under its own weight, an analysis was carried out on the critical (i.e. highest) section of the north embankment using spreadsheet calculations. The calculations were based on the 'e-logσ' Method' to estimating settlement as a function of initial void ratio, initial and final effective stress, preconsolidation stress, and recompression and compression index as described in CFEM (2006).

In the analysis, it was assumed that the 9.5 m high embankment section was constructed in 32 – 0.3 m thick lifts, and that a nominal, compaction equipment induced quasi-preconsolidation pressure (of 20 kPa) would have been present within each lift as a result of placement and compaction to at least 95% of the standard Proctor maximum dry density.

As described in Section 4, two (2) laboratory, one-dimension (oedometer) consolidation tests were performed on specimens of the clay embankment fill material obtained from Shelby tube samples taken within the main body of the embankment. One sample was taken from a shallow depth in the embankment fill (at about 1.8 m below top of roadway) and the other sample was taken from a deeper depth in the embankment fill (at about 6.1 m below top of roadway). Based on an average of the laboratory consolidation test results, and the discussion above, the following deformation parameters were employed in the settlement analysis.

Total Embankment Height (m)	Clay Fill Layer Thickness (m)	Unit Weight (kN/m^3)	σ_{vo}' (mid-layer) (kPa)	σ_p' (compaction induced) (kPa)	C_c	C_r	e_o	c_v* (cm^2/s)
9.5	0.3	19.5	2.9	20	.076	.012	.767	(see below)

Based on the above, a total compression settlement of 0.29 m is estimated to have occurred within the highest section of the embankment under the self weight of the fill.

It is difficult to estimate the length of time required to complete the settlement associated with the fill placement as the appropriate length of the drainage path (single lift versus the total embankment height) is open to debate. In addition, the values of coefficient of consolidation (c_v) interpreted from the laboratory consolidation test results range over an order of magnitude, from about $2\text{x}10^{-1}$ cm^2/s to $2\text{x}10^{-2}$ cm^2/s. Further, if empirical correlations are utilized to estimate c_v for a recompacted soil based on the average liquid limit in the clay embankment fill ($w_{L(avg)}$ = 56%), a much lower value is obtained ($3.6\text{x}10^{-4}$ cm^2/s). The following table summarizes the range of possible values of coefficient of consolidation for the clay fill and the associated calculated time to reach 90% consolidation conservatively assuming a drainage path length equal to one-half the height (4.75 m) of the highest embankment section (i.e. for drainage to the upper fractured clay crust and/or embankment side slopes).

Data Source	Estimated c_v (cm^2/s)	Estimated t_{90}	
		(days)	(years)
Lab Consolidation Test (average)*	1.2×10^{-1}	20	0.05
Lab Consolidation Test (low bound)*	1.6×10^{-2}	140	0.38
Empirical Correlation (NavFac, 1971)	3.6×10^{-4}	6150	16.8

Note: * For stress range of 20 kPa ≤ σ_v' ≤ 185 kPa

Although the range of estimated time to reach 90% consolidation of the embankment fill is large, given that the embankment fill was initially placed in 1971 (almost 40 years ago) , regardless of the actual time, it can be assumed that the primary compression/settlement is complete.

The magnitude of secondary (creep) settlement for the clay fill is expected to be about 25 mm per log-cycle of time based on the results of the laboratory consolidation tests. Given this, assuming t_{90} for the primary consolidation was completed in 140 days (0.38 year), the embankment fills would currently be just starting a third log-cycle of creep (i.e. from about 38 to 380 years) and the magnitude of the secondary settlement remaining over the life of the highway would be negligible.

6.3.2 Settlement of Foundation Soils

To estimate the magnitude of settlement that may have occurred within the foundation stratum due to construction of the embankment, an analysis was carried out of the full north embankment geometry using the commercially available program Settle3D Version 2.0 (by Rocscience Inc.). For the analysis, the bulk unit weight of 19.5 kN/m^3 for the embankment fill was employed and the critical subsurface section (in terms of the deepest clayey strata) was modelled as was encountered below the highest portion of the embankment.

As described in Section 4, two (2) laboratory, one-dimension (oedometer) consolidation tests were performed on specimens of the native lower clayey silt to silt clay stratum obtained from Shelby tube samples. This data was combined with the results of the in situ field vane tests and laboratory index tests conducted as a part of this study as well as with similar data (including laboratory consolidation tests) obtained during the MTO's previous investigations at the site in 1968 and 1972 (Geocres Reports 30L 14-036 and 30L 14-045 – see Appendix D) to assess the deformation parameters for the clayey foundation soils as shown in Appendix F.

Values of void ratio (e_o) from the consolidation tests and from estimates based on the measured water contents employing a Specific Gravity (G_s) of 2.75 (from laboratory testing) were utilized to develop a profile and design line of e_o versus elevation as shown on Figure F1.

Values of recompression index (C_r) and compression index (C_c) were estimated from the consolidation test e-logσ' plots as well as from the laboratory index test data using empirical correlations proposed in literature by Koppula (1986), Terzaghi and Peck (1967), Kulhawy and Mayne (1990), Azzouz et al. (1976) and Britto and Gunn (1987). Profiles and the design lines of C_r and C_c versus elevation are shown on Figures F2 and F3.

Values of preconsolidation stress (σ'_p) were estimated from the consolidation test e-logσ' plots (using the Casagrande construction and the Strain-Energy method proposed by Becker (1987)). Estimated values of preconsolidation stress from consolidation tests carried out as a part of the previous studies at the site by MTO (Geocres 30L14-036) as well as from investigations by MTO at nearby sites for the Welland Canal (Geocres 30L14-005) were also utilized. The following correlation relating the measured in situ undrained shear strengths to preconsolidation stress (Mesri, 1975) was also employed:

$$\sigma_p' = \frac{s_{u(mob)}}{0.22} \qquad \text{(after Mesri, 1975)}$$

where :

$s_{u(mob)} = \mu s_{u(FV)}$ (after Bjerrum, 1973)
σ_p' = pre-consolidation stress (kPa)
$s_{u(mob)}$ = average mobilized undrained shear strength (kPa)
$s_{u(FV)}$ = undrained shear strength from field vane test (kPa)
μ = Bjerrum's correction factor based on Plasticity Index

The profile and design line for σ'_p along with an estimate of the vertical effective stress (σ'_v) prior to the embankment construction are shown on Figure F4.

Using the above, the Settle^{3D} analysis indicates a total consolidation settlement of about 0.4 m within the foundation strata below the highest section of the embankment. The results of the Settle^{3D} analysis including the modelled embankment geometry and contour fields of final vertical effective stress (σ'_f) and total consolidation settlement are shown on Figures F5 and F6, respectively.

The length of time required to complete the consolidation settlement of the foundation strata is a function of the value of coefficient of consolidation of the native clayey strata and the assumed length of drainage path. Given the very stiff consistency, heavily over-consolidated and likely fractured nature of the crust, it is reasonable to assume that consolidation/recompression will occur quickly in the crust and that the rate of consolidation will be primarily controlled by the coefficient of consolidation and thickness of the underlying stiff clayey silt to silty clay stratum. The values of coefficient of consolidation (c_v) interpreted from the laboratory consolidation test results range over just less than an order of magnitude, from about $2x10^{-2}$ cm^2/s to $3x10^{-3}$ cm^2/s for samples obtained within this stratum. Further, if empirical correlations are utilized to estimate c_v for a recompacted soil based on the average liquid limit in the clay embankment fill ($w_{L(avg)}$ = 48%), a value of $1.9x10^{-3}$ cm^2/s is obtained. The following table summarizes the range of possible values of coefficient of consolidation for the clay fill and the associated calculated time to reach 90% consolidation assuming a drainage path length equal to one-half the thickness (8 m) of the deepest portion of the stiff clayey silt to silty clay stratum located below the crust.

Data Source	Estimated c_v (cm^2/s)	Estimated t_{90} (years)
Lab Consolidation Test (average)*	1.4×10^{-2}	1.2
Lab Consolidation Test (low bound)*	3.4×10^{-3}	5.1
Empirical Correlation (NavFac, 1971)	1.9×10^{-3}	9.1

Note: * For stress range of 150 kPa ≤ σ_v' ≤ 300 kPa

Although the range of estimated time to reach 90% consolidation of the foundation strata is large, given that the embankment fill was initially placed in 1971 (almost 40 years ago) , regardless of the actual time, it can be assumed that the primary consolidation is complete.

The magnitude of secondary (creep) settlement for the portion of the clayey silt to silty clay foundation stratum that likely became normally consolidated due to the embankment construction (between about Elev. 165 m and 153 m - about 12 m thick, see Figure F4) is expected to be about 25 mm per log-cycle of time based on the results of the laboratory consolidation tests. Given this, assuming t_{90} for the primary consolidation was completed in 5.1 years, the foundation soils would currently be nearing the end of the first log-cycle of creep (i.e. from about 5 to 50 years) and the magnitude of the secondary settlement remaining over the life of the highway would be negligible.

6.3.3 Global Embankment Stability

To assess the global stability of the original and current embankment geometries, analyses were performed on the critical (i.e. highest) section of the approach embankment. In this context, global stability refers to slip surfaces that pass from crest to toe over the full embankment height and/or engage the underlying foundation strata.

Limit equilibrium slope stability analysis were performed using the commercially available program Slide Version 5.035 (by Rocscience Inc.), employing the Morgenstern-Price method of analysis. For all analyses, the factors of safety of numerous potential failure surfaces were computed in order to establish the minimum factor of safety. The factor of safety is defined as the ratio of forces tending to resist failure to the driving forces tending to cause failure. A target minimum factor of safety of 1.3 is normally used in the design of embankment slopes under static conditions. Factors of safety that are less than about 1.0 indicate that failure is expected and less than about 1.1 suggest that large deformations are likely to occur which may then lead to failure in strain softening materials. In general, circular slip surface were utilized in the assessment and both total stress and effective stress analyses were carried out.

6.3.3.1 Total Stress Analysis

As described in Section 4, in situ field vane testing to measure the undrained shear strength (s_u) of the clay embankment fill and the clayey foundation strata was carried out as part of the investigation. An M-3 hand vane was used to measure undrained shear strengths at shallow depths below the embankment side slopes in the test pits. An MTO 'N'-vane was used to measure undrained shear strengths within the main body (or 'core') of the embankment as well as in the foundation strata in the boreholes. This data was combined with the results of the in situ field vane tests conducted during the MTO's previous investigations at the site in 1968 and 1972 (Geocres Reports 30L 14-036 and 30L 14-045 – see Appendix D) to assess undrained strength profiles and design lines of s_u versus elevation for the clayey embankment fill (below side slopes and in main body) and clayey foundation soils as shown on Figure F7 in Appendix F. For the analysis, a bulk unit weight of 19.5 kN/m^3 was employed for the embankment fill and clayey foundation strata.

Using the above, for a 9.5 m high embankment with 2H:1V side slopes (i.e. critical section of the original embankment geometry), the total stress Slide analysis indicates a Factor of Safety (FoS) > 2.5 for a slip surface within the clay embankment fill extending from crest to toe, as shown on Figure F8. For slip surfaces that extend from the embankment crest into the foundation soil (i.e. into the very stiff crust), the FoS is greater than 2.7, as shown on Figure F9. Based on these results, it is unlikely that the undrained shear strengths of the embankment fill and foundation strata were controlling the original embankment failures that occurred in July and August 1971, shortly after the original embankment construction.

Similarly, for a 9.5 m high embankment with 4.2 m high by 8.8 m wide toe berms (i.e. critical section of the initially remediated embankment), the total stress Slide analysis indicates a FoS greater than 3.1 for slip surfaces within the embankment fill and a FoS greater than 3.4 for slip surfaces that extend from the embankment crest into the foundation soil as shown on Figures F10 and F11, respectively.

Based on the above results (i.e. FoS>2.5 for all analyses) and considering that failures of the embankment are documented to have occurred, the analysis indicates that for the clay materials at this site, the total stress parameters (i.e. undrained shear strengths) are not critical to the assessment of the global embankment stability.

6.3.3.2 Effective Stress Analysis

As described in Section 4, laboratory drained direct shear (DS) tests were carried out on 2 sets of 3 specimens of the clay fill. One set of tests was carried out on a relatively 'undisturbed' Shelby tube sample of the fill, while the other set of tests was carried out on a laboratory recompacted sample of the clay fill. The results of the DS testing, in terms of peak and residual shear strengths, are shown on Figure F12 in Appendix F and it can be seen on the plot that there does not appear to be any significant difference between the shear strengths measured on the 'undisturbed' and recompacted samples of the fill. On Figure 12, straight lines representing the peak and residual Mohr-Coulomb failure envelopes have been 'fit' to the data. The results of these interpretations are summarized as follows:

Drained Direct Shear Tests		Effective Cohesion Intercept, c' (kPa)	Effective Angle of Internal Friction, ϕ' (degrees)
Mohr-Coulomb Failure Envelope	Peak	4	34
	Residual	0	34

Note: for stress range 0 kPa ≤ σ_v' ≤ 40 kPa

In addition, laboratory consolidated undrained triaxial compression tests (CIU) with pore pressure measurement were carried out on 1 set of 3 specimens and 2 sets of 2 specimens from relatively 'undisturbed' Shelby tube samples of the clay fill. The results of the current CIU triaxial testing have been combined with the results of triaxial testing carried out by MTO on specimens of the embankment fill during their previous investigation of the site in 1972 (Geocres Report 30 L-45 – see Appendix D) and the data are shown on Figure F13. On Figure 13, straight lines representing the Mohr-Coulomb failure envelopes at different stress levels have been 'fit' to the data. The results of these interpretations are summarized as follows:

Consolidated Undrained Triaxial Tests		Effective Cohesion Intercept, c' (kPa)	Effective Angle of Internal Friction, ϕ' (degrees)
Mohr-Coulomb Failure Envelope	0 kPa ≤ σ_v' ≤ 40 kPa	4	30
	60 kPa ≤ σ_v' ≤ 250 kPa	0	23

Although best-fitting straight lines through the shear strength data, as described above, is convenient for analyses that employ the Mohr-Coulomb failure envelope to model the shear strength of a soil, in reality, this approach is an over-simplification of the actual non-linear trend that best represents the overall soil behaviour. In addition, with this approach, special care is required in the analysis to check that the range of stresses that are operative in a solution (i.e. on a particular slip surface in a slope stability analysis) are within the range of stresses over which the Mohr-Coulomb envelope has been defined. Further, at very low stresses, it is well accepted in literature (Lo and Morin, 1972), that the shear strength of soils is highly non-linear and that the actual failure envelope should pass through the origin (i.e. effective cohesion intercept, c'=0 kPa). This fact is of particular importance when analysing shallow surficial slope failures as will be described in the next section. The best approach to defining the effective shear strength of a soil based on direct shear and triaxial test data is to fit a non-linear, fully defined shear strength envelope through the data starting at the origin. This approach has been carried out and the fully defined shear strength envelopes for the results of the direct shear testing (peak and residual) and triaxial testing on Figures F14 and F15, respectively.

Laboratory shear strength testing on samples of the native clayey silt to silty clay strata has not been carried out as part of the current assignment. However, consolidated undrained (CIU) triaxial tests with pore pressure measurement have been carried out on samples of native silty clay strata from the site and from areas close to the site (in the Welland area) during previous investigations by the MTO in 1967 and 1972 (Geocres No. 30L 14-005 and 30L-45). The results of the interpreted best fit, Mohr-Coulomb strength envelopes to the test data are summarized as follows:

	Location of Sample in Foundation Soil Strata	Effective Cohesion Intercept, c' (kPa)	Effective Angle of Internal Friction, ϕ' (degrees)
Hwy 140/CNR Overpass North Embankment MTO 1972 (Geocres No. 30L-45)	Unknown	14	25
Forkes Road Crossing of Proposed Welland Canal MTO 1967 (Geocres No. 30L 14-05)	Ground Surface to Elevation 169 m (Crust)	7	24
	Elevation 169 m to 164 m (below Crust)	0	25

The results of the triaxial testing on the native clayey foundation soils appear to be consistent with each other; are higher than the effective shear strength parameters measured on the clay fill; and the higher values of c' in two of the tests on the native soils are likely attributable to the over-consolidated nature of the silty clay crust. Based on the above, the following average effective strength parameters have been used for the foundation soils in the analysis.

Strata	Effective Cohesion Intercept, c' (kPa)	Effective Angle of Internal Friction, ϕ' (degrees)
Upper Silty Clay to Clay (Crust)	10	25
Lower Clayey Silt to Silty Clay	0	25

In addition to defining the shear strength parameters of the clay fill and native clayey foundation strata, it is also necessary to define the location of the groundwater table at the site and assess the potential for excess pore pressures to be present in the clay embankment fill near the end of construction and at present, if any.

Based on the piezometers installed within the clayey silt to silt (till) during the current investigation, the lower groundwater table is observed to be at about Elevation167.5 m, just below the base of the upper silty clay to clay crust. This groundwater level is similar and only slightly lower than that reported for piezometers installed in the till during the MTO's previous investigations at the site in 1968 and 1972 (Geocres Reports 30L 14-036 and 30L 14-045). During these investigations, MTO also installed piezometers within the upper portion the silty clay to clay stratum and noted that a shallower, perhaps perched groundwater table was present in this upper stratum. Finally, a single piezometer installed within the clay embankment fill during the current investigation was dry indicating no pore pressures at present within the embankment fill. However, piezometers installed by the MTO in 1972 within the embankment fill and immediately below the fill in the shallow native clay soils shortly after construction and after the initial failures indicated erratic water levels/pore pressures ranging from dry (within the upper fills) to excess pore water heads of about 5 m or higher above original ground surface (in the lower fills) and about 4 m in the shallow native clay soils. The results of the water level measurements in the piezometers installed at the site (past and present) are summarized below.

Year	Strata	Average Water Level Elevation (m)	Excess Pore Water Head (above original ground surface) (m)
1968	Silty Clay to Clay	176.0	-
	Clayey Silt to Silt (Till)	169.5	-
1972	Upper Clay Fill	-	0
	Lower Clay Fill	≥ 182.3	≥ 4.9
	Silty Clay to Clay	176.0 to 181.5	0 to 4.1
	Clayey Silt to Silt (Till)	169.5	-
2008	Clay Fill	-	0
	Clayey Silt to Silt (Till)	167.5	-

Note: original ground surface/base of embankment fill at about Elevation 177.4 m

The above measurements suggest the following:

- a downward gradient likely exists in the native silty clay to clay stratum (from 176.0 m to 169.5 m);
- excess pore water pressures of about 50 kPa or greater were likely present near the end of construction in the lower embankment fills and upper native silty clay to clay strata; and,
- currently, there is likely no excess pore water pressure present in the lower clay embankment fills and/or within the main body or 'core' of the embankment.

Combining the above and using a fully defined strength envelope for the clay fill, for the 9.5 m high embankment with 2H:1V side slopes (i.e. critical section of the original embankment geometry), the effective stress Slide analysis indicates a Factor of Safety (FoS) ≈ 1.06 at the end-of-construction (with excess pore pressures of about 50 kPa in lower half of the embankment fill and upper foundation stratum) for a slip surface within the clay embankment extending from crest to toe as shown on Figure F16 in Appendix F. Although slightly above unity, assigning higher excess pore pressures in the embankment fill and/or the addition of a water filled, tension crack (consistent the field observations at the time of failure) would reduce to the FoS closer to 1.0. As such, the results of this analysis are consistent with the observations in the field at or shortly after the end of construction in July and August 1971 when the embankment failed.

Similarly, for a 9.5 m high embankment with approximately 4 m high by 9 m wide toe berms (i.e. critical section of the initially remediated embankment), the effective stress Slide analysis indicates a FoS ≈ 1.8 shortly after the berm construction (with excess pore pressure of about 50 kPa in the lower half of the embankment and upper foundation stratum) for slip surfaces within the embankment fill as shown on Figure F17.

For the existing embankment geometry, the effective stress Slide analysis indicates a FoS ≈ 1.8 at the long-term after the berm construction and with no excess pore pressure remaining in either the embankment fill or upper foundation stratum, for a slip surface extending from crest to toe of the embankment fill as shown on Figure F18.

Based on the above results, and the consistency of the results with the observed embankment performance/time of known failures, the analysis indicates that for the clay materials at this site, the effective stress parameters are critical to the assessment of embankment stability.

6.3.4 Surficial Embankment Stability

To assess the surficial embankment stability of the post-toe berm construction and current embankment geometry, analyses were performed on the critical (i.e. highest) section of the approach embankment, but specifically considering the stability of the upper embankment slopes above the toe berm. In the analyses, shallow, wedge-type failure surfaces were utilized with depths ranging from about 0.25 m to 1.0 m below the embankment side slopes.

The lower embankment slopes (i.e. on the toe berm) are generally flatter than the upper slopes, with lower slope profiles ranging from about 2.5H:1V to 5H:1V as shown on Drawing 1. The performance of the lower slopes has been notably better than that of the upper slopes with no reported instances of failures, tension cracks or sloughing, likely as a result of the flatter profile(s). Given this, the stability of the lower slopes was not specifically analysed.

Limit equilibrium slope stability analysis were performed using the commercially available program Slide Version 5.035 (by Rocscience Inc.), employing the Morgenstern-Price method of analysis. Factors of safety that are less than about 1.0 indicate that failure is expected and less than about 1.1 suggest that deformations are likely to occur which may then lead to failure, in particular in strain softening materials.

6.3.4.1 Total Stress Analysis

The details of the analysis and profiles of undrained shear strength as described in Section 6.3.3.1 were also employed in the total stress surficial embankment stability analysis.

For the approximately 5 m high upper embankment section (above the toe berm) with 2H:1V side slopes (i.e. critical section of the embankment geometry), the total stress Slide analysis indicates a Factor of Safety (FoS) > 3.4 for a wedge-type sliding surface within the clay embankment fill at a depth of about 1.0 m and extending from crest to toe of upper slope, as shown on Figure F19. Based on these results, it is unlikely that the undrained shear strength of the near surface embankment fill is controlling the surficial instability and sloughing that has been noted on the upper embankment slopes from 1973 to present.

6.3.4.2 Effective Stress Analysis

The details of the stability analysis and effective shear strength parameters as described in Section 6.3.3.2 were also employed in the effective stress surficial embankment stability analysis. However, given the range in measured effective strength parameters (c' and ϕ'), in particular in the low normal stress range that is most applicable to shallow slope stability problems, an assessment had to be made as to the most appropriate values to be employed in the analysis. In addition, an assessment of the most likely pore pressure condition in the shallow, upper slopes was also required. Although most of the test pits were dry upon completion of excavation in early-October 2008 (except for TP-3 where water seepage at a depth of 1 m was noted), the pore pressure condition in the near surface soils of the upper slopes is likely transitional in nature and varies with the seasonal temperature and precipitation. As part of this assessment, a literature review was carried out to achieve a better understanding of the dominant factors and likely conditions affecting this type of analysis, as discussed in the following section.

6.3.4.2.1 Strength and Pore Pressures in Near Surface Soils

As explained in literature, experience has shown that in properly constructed, plastic clay embankments, the outer, shallow layers can experience dramatic strength loss over time while the global stability of the overall embankment remains unchanged. This strength loss, and the subsequent shallow, sloughing-type failures associated with it represent a costly maintenance problem in highway embankments (Aubeny and Lytton, 2004). Research carried out by Zhang, Tao and Morvant (2005) indicates that the loss of strength happens in three phases:

1. Shrinkage cracks form at the surface of the slope due to shrinking and swelling resulting from seasonal changes in moisture and temperature;
2. Water then infiltrates the slope through the shrinkage cracks during subsequent wet seasons causing the near surface soil to become saturated; and,
3. The shear strength of the near surface clay then deteriorates due to the elevated moisture content.

Laboratory studies by Kayyal and Wright (1991) and Rogers and Wright (1986) indicate that wetting of compacted soils can cause a dramatic loss in the effective cohesion/shear strength (c'), while the friction angle (ϕ') of the soil is virtually unchanged. The water infiltration and subsequent loss of shear strength causing instabilities can occur anywhere from months to years after construction. The timeline can be influenced by the plasticity index of the soil, local weather conditions and degree of compaction of the soil near the edges of the slope during construction (Zhang, Tao and Morvant, 2005, and Greenwood, Holt and Herrick, 1985). Weather conditions can also induce a saturated condition in the near surface soils. In the spring, the surficial soils thaw from the ground surface down, and become saturated due to the layer beneath being frozen and preventing drainage (Andersland and Ladanyi, 2004).

Based on the above, it appears that the shear strength and pore pressure conditions in shallow clay slopes can be a dynamic and transitional process. Given this, a series of analyses have been carried out to assess the sensitivity of the Factor of Safety for shallow slip planes to these variables, as described below.

6.3.4.2.2 Stability Based on Direct Shear Test Data

As discussed previously (and shown on Figure F12), based on fitting a linear, Mohr-Coulomb failure envelope to the results of the laboratory direct shear tests carried out on samples of the shallow clay embankment fill (at low normal stresses), the effective friction angle (ϕ') of the material is about 34^{o} while the effective cohesion (c') can vary from about 4 kPa to 0 kPa.

Further, as discussed in Section 6.3.3.2, a better approach to defining the shear strength is to fit a non-linear, fully defined shear strength envelope through the laboratory data starting at the origin. This approach has been carried out and the fully defined shear strength envelopes for the results of the direct shear testing (peak and residual) are shown on Figure F14.

Using these two approaches to defining the shear strength of the clay embankment fill, and varying the pore pressure conditions in the shallow slopes from dry (i.e. zero pore pressure) to saturated (i.e. hydrostatic pore pressure from slope surface to base of the sliding surface), the results of the effective stress Slide analysis for shallow sliding surfaces on the upper embankment slopes are shown on Figures F20 and F21. It can be seen that the Factor of Safety is very sensitive to the range of conditions on the slope faces ranging from a high of greater than 2.5 (for dry conditions and peak shear strengths) to a low of less than unity, implying failure would occur (for saturated conditions and residual/post-peak shear strengths). A typical result of the effective stress Slide analysis, using a fully defined failure envelope based on the direct shear testing, is shown on Figure F22.

In our opinion, as noted previously, the non-linear, fully defined strength envelope represents the best approach to representing the shear strength of shallow clay soils. As shown on Figure F21, under saturated slope conditions, the Factor of Safety will be about one or less for sliding surfaces varying from 0.25 m to 1 m deep. The results of these analyses are consistent with the observations in the field since 1973 where shallow, sloughing-type slope failures on the upper embankment slopes (above the berm) have been an annual, but seasonal problem at the site, most likely having occurred following periods of freeze-thaw (early spring) or heavy rainfall (late fall).

6.3.4.2.3 Stability Based on Triaxial Test Data

As discussed previously (and shown on Figure F13), based on fitting a linear, Mohr-Coulomb failure envelope to the results of the laboratory triaxial tests carried out on samples of the shallow clay embankment fill (at low normal stresses), the effective friction angle (ϕ') of the material is about 30^o while the effective cohesion (c') is estimated to be as high as about 4 kPa.

As discussed in Section 6.3.3.2, a better approach to defining the shear strength is to fit a non-linear, fully defined shear strength envelope through the laboratory data starting at the origin. This approach has been carried out and the fully defined shear strength envelope for the results of the triaxial testing is shown on Figure F15.

Using these two approaches to defining the shear strength of the clay embankment fill, and varying the pore pressure conditions in the shallow slopes from dry to saturated, the results of the effective stress Slide analysis for shallow sliding surfaces on the upper embankment slopes are shown on Figures F23 and F24. Once again, it can be seen that the Factor of Safety is very sensitive to the range of conditions on the slope faces ranging from a high of greater than about 2.5 (for dry conditions and high shear strengths) to a low of less than unity, implying failure would occur (for saturated conditions and low shear strengths). A typical result of the effective stress Slide analysis is shown on Figure F25.

As noted previously, in our opinion, the non-linear, fully defined strength envelope represents the best approach to representing the shear strength of shallow clay soils. As shown on Figure F24, under saturated slope conditions, the Factor of Safety will be 1 or less for sliding surfaces varying from 0.25 m to 1 m deep. The results of these analyses are again consistent with the observations in the field since 1973 where shallow, sloughing-type slope failures on the upper embankment slopes (above the berm) have been an annual, but seasonal problem at the site, most likely having occurred following periods of freeze-thaw (early spring) or heavy rainfall (late fall).

6.4 Remediation Options

Based on the assessment of the potential factors affecting the performance of the north embankment and approach, including the analysis of the various mechanisms, the recent and past site observation including those relating to the effect of the previous remediation activities (including slope flattening of the south embankment slopes), in our opinion, it is most likely that the cause of the on-going distress on the north embankment is of a surficial nature and related to a combination of the following factors:

- Geometry (i.e. relative steepness) of the existing upper embankment slopes (above the toe berm);
- Mineralogy of the local soils and its inherent effect on the effective shear strength of the clay fill; and
- Effects of local climate including precipitation and wetting-and-drying cycles as well as snow melt and freezing-and-thawing cycles.

Given this, the remediation of the north embankment and approach should focus on methods that increase the local stability of the upper embankment slopes (i.e. above the berm), control the run-off at/over the slope crest and down the slope faces, improve drainage from within the side slopes and promote deep-rooted vegetation on the flattened slope faces.

The following sections provide an overview of eleven (11) remediation schemes that could be considered for this site. A summary of the advantages, disadvantages, relative costs and risks/consequences for each of the remediation alternatives is provided in Table 1. From a foundations perspective, the deep benching and slope flattening with granular material is the preferred remediation option for this site.

6.4.1 Deep Benching and Granular Slope Flattening

Adding a granular buttress to the existing 2H:1V upper embankment side slopes, from the crest of the embankment to the top of the mid-height berms, at a profile of 2.5H:1V (or flatter), is a viable remediation option to mitigate the on-going surficial instability on the north embankment. To improve the long-term performance of this method, it is recommended that the new granular fill material be keyed into the existing clay embankment by a series of deep benches to remove as much of the previously distresses/weakened material within about the frost depth on the current side slopes. The deep benching of the new granular fill into the existing earth slopes should be carried out in accordance with OPSD 208.010 with the benches constructed to the maximum specified dimensions. In this regard, it is recommended the dimension of the benches be 1.0 m high by 2.0 m wide. It should be noted that the extent of excavation/removal of existing earth slope material required with this option will require removal (and replacement) of the existing guide rail, however, the excavation should not encroach into the existing travelled lane(s). Further discussion on the requirements for temporary protection systems is provided in Section 6.7.

A subdrain should be provided within the granular fill near the interface with the existing clay fill. Granular A or Granular B Type I fill (both with not more than 5 percent passing the number 200 sieve) could be used for the slope flattening and the fill should be placed and compacted in accordance with the requirements of OPSS 501. A schematic of this remediation option is shown on Drawing G1 in Appendix G.

6.4.2 Standard Benching and Granular Slope Flattening

Slope flattening with granular fill in the manner describe above, but without deep benching into the existing embankment fill can also be considered. Keying the new granular fill into the existing earth fill with the standard benching dimensions (i.e. 0.3 m < Bench Height < 1.0 m) will reduce the volume of excavated material, however, without the deep benching (described previously), there is a risk that a zone of weakened material will remain below the granular slope flattening that may affect the performance of the flattened slopes. The fill for the slope flattening should be benched/keyed into the existing side slopes in accordance with OPSD 208.010. Granular A or Granular B Type 1 fill (both with not more than 5 percent passing the number 200 sieve) could be used for construction of the granular slope flattening and the fill should be placed and compacted in accordance with the requirements of OPSS 501. A schematic of this remediation option is shown on Drawing G2.

6.4.3 Slope Flattening with Silty Clay

Flattening the side slopes of the north embankment with cohesive (i.e. silty clay) fill is a feasible option and could be considered for slope remediation at this site.

However, due to the lower strength of the locally available silty clay, it would be necessary to construct the new side slopes at a profile of not steeper than 3.5H:1V to mitigate the potential for any further instabilities. At this profile, the new side slopes could be constructed with material similar to that of the existing embankment and could be sourced from borrow pits located in close proximity to the embankment site. Prior to placing the new fill on the existing slopes (and extending out from beyond the toe of the existing toe berm), all vegetation and organic materials should be removed. The fill for the new side slopes should be keyed into the existing side slopes using the standard benching dimensions in accordance with OPSD 208.010 and compacted in accordance with the requirements of OPSS 501. In addition, the filling should take place in the summer period when the clayey material can be placed near optimum moisture content. A schematic of this remediation method is shown on Drawing G3.

It should be noted that the use of non-free draining, slope flattening material would still be subject to wetting-drying and freeze-thaw cycles resulting in a risk of some localized surficial sloughing occurring on the final slope surface.

6.4.4 Granular Blanket at 2H: 1V (without Slope Flattening)

Adding a minimum 1.2 m thick granular blanket to the existing embankment side slopes, from the crest of the slope to the top of the existing berms, at a profile of 2H:1V, as shown in Drawing G4, could be considered as a remediation option for the surficial embankment instabilities.

Prior to adding the granular blanket all surficial vegetation and topsoil/organic matter should be removed as well as any of the existing loose granular material that has been pushed over the crest of the slope during the maintenance and previous remediation at the site. The granular (fill) blanket should be keyed into the existing side slopes using the standard benching dimensions in accordance with OPSD 208.010. Granular A or Granular B Type 1 fill (both with not more than 5 percent passing the number 200 sieve) could be used for construction of the granular blanket and the fill should be placed and compacted in accordance with the requirements of OPSS 501.

6.4.5 Partial Removal and Replacement with Granular Fill at 2H:1V (without Slope Flattening)

Removing a minimum of about 1.2 m of the existing embankment fill on the outer edges of the embankment slopes (from slope crest to top of the existing berms) and replacing it with granular fill at a profile of 2H:1V, as shown in Drawing G5 could be considered as a remediation option for the surficial embankment instabilities.

In order to minimize the effect of the excavation on the performance of the existing roadway, the partial removal of the existing clay embankment fill would have to be carried out in a series of strips of limited width. The granular fill should be keyed into the existing embankment fill using the standard benching dimensions in accordance with OPSD 208.010. Granular A or Granular B Type 1 (both containing not more than 5 percent passing the number 200 sieve) could be used for construction and the granular fill should be placed and compacted in accordance with the requirements of OPSS 501.

6.4.6 Partial Sub-Excavation and Reconstruction with Geogrid Reinforcment

Partial sub-excavation of the existing embankment, followed by reconstruction of the side slopes with geogrid reinforcement, as shown in Drawing G6, could be considered as a remediation option for the surficial embankment instabilities.

In order to minimize the effect of the excavation on the performance of the existing roadway, the partial sub-excavation into the existing clay embankment fill would have to be carried out in a series of strips of limited width and/or temporary shoring may be required. The geogrid reinforcement would have to be installed extending from the face of the side slopes to at least 6.5 m into the embankment and should be placed at least at 500 mm vertical spacing. These preliminary dimensions are based on the results of limit equilibrium stability analysis considering a wedge-type failure around the geogrid reinforcement zone. Each layer of geogrid should be wrapped over the face of the slope and tied into the next level to further stabilize and enhance the surficial stability the slope face. It should be noted that each of these recommendations is of a preliminary nature and a detailed design in conjunction with the geogrid supplier would have to be carried out if this method was adopted.

Considering the depth of excavation into the side slopes of the embankment necessary to install the minimum required length of geogrid, the construction operations for this option could not be performed while leaving both lanes of the highway open to traffic. Traffic would likely have to be reduced to a single lane. A schematic of this remediation option is shown on Drawing G6.

6.4.7 Counterfort Drains

Installing a series of counterfort drains along the upper slope face of the embankment could be considered as a remediation option at this site. The counterfort drains and trench drains should be at least 1.5 m deep and spaced at 10 m to 20 m along the slope above the berm. A schematic of this remediation option is shown on Drawing G7.

The purpose of the drainage system is to alleviate the accumulation of moisture/infiltration of precipitation and therefore the cycles of wetting and drying and subsequent weakening of the near surface clay material on the slopes. This option would require annual inspections and some maintenance to ensure the proper performance of the drains.

6.4.8 Slope Re-grading and Vegetation

If a smaller scale of remediation is desired, consideration could be given to re-grading the over-steepened sections of the slope (i.e. where the loose granular fills are present), followed by the application of new topsoil along with dense re-seeding of the slopes with thick, deep rooted vegetation designed to improve surficial stability.

Providing the slopes with a thick vegetative cover such as the Crown Vetch mix specified in OPSS 572, or similar as designed by a landscape architect, may reduce the on-going surfical stability problems at the site. However this approach should be viewed as a temporary solution that would likely require annual inspections and maintenance to re-grade and/or re-seed areas of the slope where the initial applications did not sufficiently germinate. problems.

6.4.9 Do Nothing

Leaving the embankment in its present configuration and doing no further remediation could also be considered. The stability problems being encountered at this site are surficial in nature and do not presently affect the performance of the travelled lanes of the highway; however dipping and tilting of the guide rail has occurred in the past and could continue or worsen in the future which may affect the safety of the travelling public. This option would also require annual inspection and maintenance of the extent currently being carried out. This approach is not recommended as a remediation option at this site.

6.4.10 Cement-Soil Mixing

Enhancing the shear strength of clayey soils by the addition of cement and or lime (so called cement-stabilization or lime-stabilization) has been utilized in the past, mostly to improve the subgrade characteristics of roadways prior to pavement structure construction. However, such an approach could be considered to improve the strength of the shallow clay fills on the upper embankment slopes at the site. The details of the actual method of construction/in situ mixing would require additional design and perhaps even field trials to assess the effectiveness. However, the mixing could potentially be carried out at either discrete locations laid out on a grid across the face of the slope, as shown schematically on Drawing G8, or in strips (of limited width) across the slope.

Based on a literature review along with previous experience on cement-stabilization projects, it is understood that mixing between about 5 percent and 15 percent of cement, by mass, into a clayey soil is a typical approach used to improve the geotechnical characteristics. In the report by Prusinski and Bhattacharja (1999) it is described that the addition of cement (or lime) to clayey soils should have the effect of decreasing the soils plasticity index (PI) by as much as 45% to 65% for soils with plasticity indices similar to those at the Hwy 140 site. A reduction in plasticity index of this magnitude should by accompanied by a corresponding increase in effective friction angle (ϕ’) as the inverse relation between PI and ϕ’ is well documented in literature.

Given this, as a first step to assess the potential affect of the addition of cement to the clay fill soils at the site, a series of samples were prepared in the laboratory each with a different percentage additive of cement. The bulk clay samples from the site were first air-dried at room temperature, crushed and then each of the specimens was prepared by drying mixing normal Portland cement (Type GU) into the clay, followed by re-wetting (to optimum moisture content, 24%) and a minimum curing period of 24 hours in a humid room. This process was considered to represent the ‘ideal’ mixing conditions and it is understood that such conditions would likely not be repeatable in the field. Following the curing period, laboratory Atterberg limits testing was carried out on each of the cement treated samples. The results of the testing are including as Table C1 in Appendix C and summarized as follows:

% of Cement Additive (by mass)	Plasticity Index, PI (%)
0 (natural soil)	34
5	32
10	31
15	24

Note: all of the above Atterberg limits tests were dried at 50^{o}C.

Based on the index test results, under ideal laboratory mixing conditions, a reduction of PI of only up to about 25% is achievable for the clay soils at this site. Given the relatively low degree of improvement achieved (for a 15% addition of cement) and considering that constructability issues (including imperfect, non-uniform mixing and loss of cement due to weather/wind,etc.) it is estimated that a large amount of cement (far in excess of 15%) would have to be utilized to have a sufficient effect/improvement to the shear strength of the clays at this site. This would make the relative costs of such a solution much higher than some of the other alternatives being considered. In addition, the long-term, post-treatment performance of the slopes would be highly dependent on the level of QA/QC during construction. As such, this option is not recommended for this site.

6.4.11 Slope Cover with Rock Protection and Mass Concrete

Covering the side slopes of the embankment with a minimum thickness of rock protection and mass concrete might be considered as a remediation option for the surficial embankment instabilities. This approach is sometimes adopted by MTO to treat surficial sloughing on localized areas of slopes and relies on the free-draining properties of the rock fill along with the impermeability of the concrete to reduce the accumulation of moisture/infiltration of precipitation and therefore enhance the surficial stability.

For this option the vegetation and existing topsoil should be stripped from the embankment side slopes prior to placement of the rock protection. The rock protection would be placed on the side slopes in accordance with OPSS 511 and then covered by a blanket of mass concrete as shown schematically on Drawing G9.

This option is not recommended for this site due to the potential for the concrete to crack as a result of settlement, frost heave, temperature changes, etc. which could then allow water infiltration. The water infiltration would result in similar near surface conditions as those previously experienced which could cause the surficial slope instabilities to persist if left unmaintained.

6.5 Preferred Remediation Option

Following consultaton with MTO Foundations and MTO Regional Geotechnical Section, it is understood that remediating the embankment by flattening the upper side slopes to 2.5H:1V (or flatter) with a granular buttress keyed into the existing embankment side slopes with deep benching, is the preferred remediation option for this site (see Section 6.4.1 and Drawing G1).

The new granular fill should be benched into the existing embankment side slopes in accordance with OPSD 208.010. This OPSD specifies that the bench height should range between 0.3 m and 1.0 m (resulting in a corresponding bench width ranging from about 0.6 m to 2.0 m); however for this site it is recommended that the maximum bench height of 1.0 m and a bench width of 2.0 m be specified in the Contract Documents and Drawings to ensure a sufficient removal of the existing and previously weakened near surface fill slope materials. In addition, the surface of the benches should be sloped with a minimum of 3% fall to promote drainage away from the existing embankment fill. Either Granular B Type I or Granular A (both with not more than 5 percent passing the No. 200 sieve) should be specified for use as the granular fill material and the fill should be placed in lifts not exceeding 300 mm loose thickness and uniformly compacted in accordance with OPSS 501.

The deep benches should be cut into the existing embankment slopes starting from the toe of the upper embankment slope (i.e. top of the mid-height berm (where present)) or from the original ground surface (where the berm is not present), continuing up to within about 0.3 m of the outside edge of the concrete curb (where present), or to the outside edge of the asphalt (where the concrete curb is not present). For construction access, the existing guide rail should be temporarily removed, a temporary concrete barrier installed, and the guide rail replaced following completion of construction of the slope remediation.

The existing slope surface drains (installed as part of the 2006 contract at the site) should be reinstated at the same locations following the slope flattening. Longitudinal drainage should also be provided at the heel of the granular portion of the slope flattening and be comprised of a 150 mm diameter perforated sub-drain (or similar MTO approved drainage pipe) encased within a 0.3 m by 0.3 m section of concrete sand surrounded by a non-woven, geotextile wrap. Alternatively, a 0.3 m by 0.3 m section of 19 mm clear stone could be used as the granular surround to the sub-drain (if approved by MTO). If a clear stone is utilized, the non-woven geotextile should be field stitched with a minimum 150 mm overlap or as per the manufacturers instructions. Lateral drainage pipes from the longitudinal heel drain to the new face of the slope flattening should be provided at minimum 25 m centers. The details of the drainage outet(s) will need to be specified by the detail design consultant.

Details of the slope remediation requirements, as described above, for a series of cross-sections along the slope are shown on Drawings H1 to H7 in Appendix H. The extent of the slope remediation should be carried out within the approximate limits shown in plan on Drawing 1 and summarized as follows:

Highway 140 North Embankment and Approach	Approx. Length of Remediation Zone (m)	Approx. Station Limits (0+000 at North Expansion Joint)
East Slope	230 m	0+060 to 0+290
West Slope	280 m	0+010 to 0+290

Effective stress stability analysis of the proposed remediation scheme (as described above) has been carried as shown on Figure H8 and H9. The results indicate a Factor of Safety greater than 1.3 for surficial and global failure surfaces.

6.6 Surface treatment

The final slope surface treatment details are to be as per the requirements of MTO and will include suitable soil and vegetative cover to prevent erosion and shallow sloughing. The re-use of the existing clayey embankment slope material removed during the deep benching should not be permitted for use as part of the final soil cover.

We understand that MTO is considering applying topsoil and seed and cover following the placement of a 200 mm earth cap on the flattened slopes. Depending on the type of material used for the earth cap, this layer could be subjected to sloughing upon saturation. Consideration should be given to adopting a flatter slope profile (up to 3H:1V) to minimize the chance of having future maintenance problems with the final surface treatment/slope cover if an earth cap is to be included. If, however, the 2.5H:1V side slope profile is adopted, it is recommended that annual inspections be carried out to check if any surficial sloughing of the surface treatment has occurred and to repair any such areas as soon as possible. If left unattended, these types of localize surficial sloughs could become larger problems in the future.

6.7 Temporary Protection Systems

As discussed in Section 6.4.1, the excavation/removal of the existing earth slope material, required as part of the recommended deep benching associated with the preferred remediation option for this site, will require removal (and replacement) of the existing guide rail, however, the excavation should not encroach into the existing travelled lane(s). As such, it is anticipated that no temporary protection systems will be required to support the travelled portion of the highway on the crest of the existing embankment.

A temporary concrete barrier system will however likely be required along the edge of the existing highway (i.e. along the white line and/or curb-and-gutter) to protect the travelling public from the adjacent work zone and vice-versa.

6.8 Lessons Learned and Considerations for Future Projects

When sourcing earth fill for embankment projects in southwestern, Ontario, the local and near surface, native clayey soils are often considered for use in construction. This material is often selected for the following reasons:

- Readily available at or near project site (may even be surplus material remaining from site grading operations);
- In situ water content is generally at or close to the plastic limit of the soil resulting in ease of compaction; and
- Perception that because the upper/near surface clayey soils within the crust have a high, undrained shear strength (i.e. consistency is generally defined as stiff to very stiff), the material will have a superior strength (as compared with the less stiff material below the crust) for construction.

While the first two points are generally valid, the third point is a misconception. Although the undrained shear strength of upper clayey soils in the crust may be high, the drained or effective shear strength of these materials may not be. Further, even though 'undisturbed' or intact samples of the clayey crust may have relatively high interpreted effective strength parameters (i.e. $c'>0$), the apparent cohesion is generally lost on remoulding during compaction and it is the effective friction angle (ϕ') of these soils that will control their long-term performance. This is particularly true when assessing the stability in the surficial slope zone (at shallow depth) that is affected by wetting-and-drying cycles and freezing-and-thawing cycles which can drastically reduce any remaining apparent cohesion in the compacted clayey fills and allow saturation of the near surface clay layers.

As described by Quigley (1975), weathering processes cause clay mineral alternations within the weathering profile leading to major changes in shear strength in the weathered tills of eastern Canada. The three important types of weathering are:

1. Oxidation weathering of chlorite to smectite;
2. Removal of K^+ from illite to produce soil vermiculite; and,
3. Adsorption of Al and Fe hydroxide to complexes at low pH onto degraded illite to produce a "pseudo smectite".

All of these processes are noted by Quigley (1975) to be effective in reducing the residual friction angles of these soils from as high as about 29^o (unweathered) to as low as 16^o to 19^o for weathered till soils.

It is well documented in literature (Mitchell, 1993) that there is an inverse relation between PI and ϕ' for clayey soils and that an increase in plasticity index usually corresponds to a decrease in effective friction angle (ϕ'). It is also interesting to note that a review of the profiles of Atterberg limits with depth for many soil strata in southern Ontario show a similar trend of high PIs in the near surface and weathered part of the stratum followed by lower

PIs in the lower and unweathered part of the stratum. As such, it would appear that Quigley's suggestion of lower effective friction angles within the weathered zone of a clay stratum is valid.

Based on the above, care should be taken when approving borrow sources for embankment construction that comprise the use of near surface clay soils from within the weathered portion of a stratum. Additional literature search, review of case studies and analysis would be required to produce a guideline that could allow modification to OPSS 212 to avoid the use of high plasticity clay soils and/or modification to OPSD 202.010 to specify flatter earth embankment slopes (than the typical 2H:1V) when constructing with high plasticity clay fills, However, based on a limited review of case studies (including the current site), it would appear that when the plasticity index (PI) of the clay soil to be used as borrow for embankment construction is greater than about 30%, a side slope flatter than 2H:1V should be specified and/or laboratory shear strength testing and analysis should be carried out to confirm the recommendations for construction. Alternatively, if a flatter fill slope profile cannot be accommodated at the site, the design of the embankment fill should incorporate a free-draining granular blanket over the clayey slopes having a thickness equal to the frost depth plus 0.5 m.

7.0 CLOSURE

This Geotechnical Investigation and Design Report was prepared by Mr. Matthew Kelly, E.I.T. and Mr. J. Paul Dittrich, Ph.D., P.Eng., a Principal and geotechnical engineer with Golder, with technical input from Mr. Murty Devata, P. Eng., Mr. Fin Heffernan, P. Eng., Golder's Designated MTO Contact for this project, conducted an independent quality review of the report.

Report Signature Page

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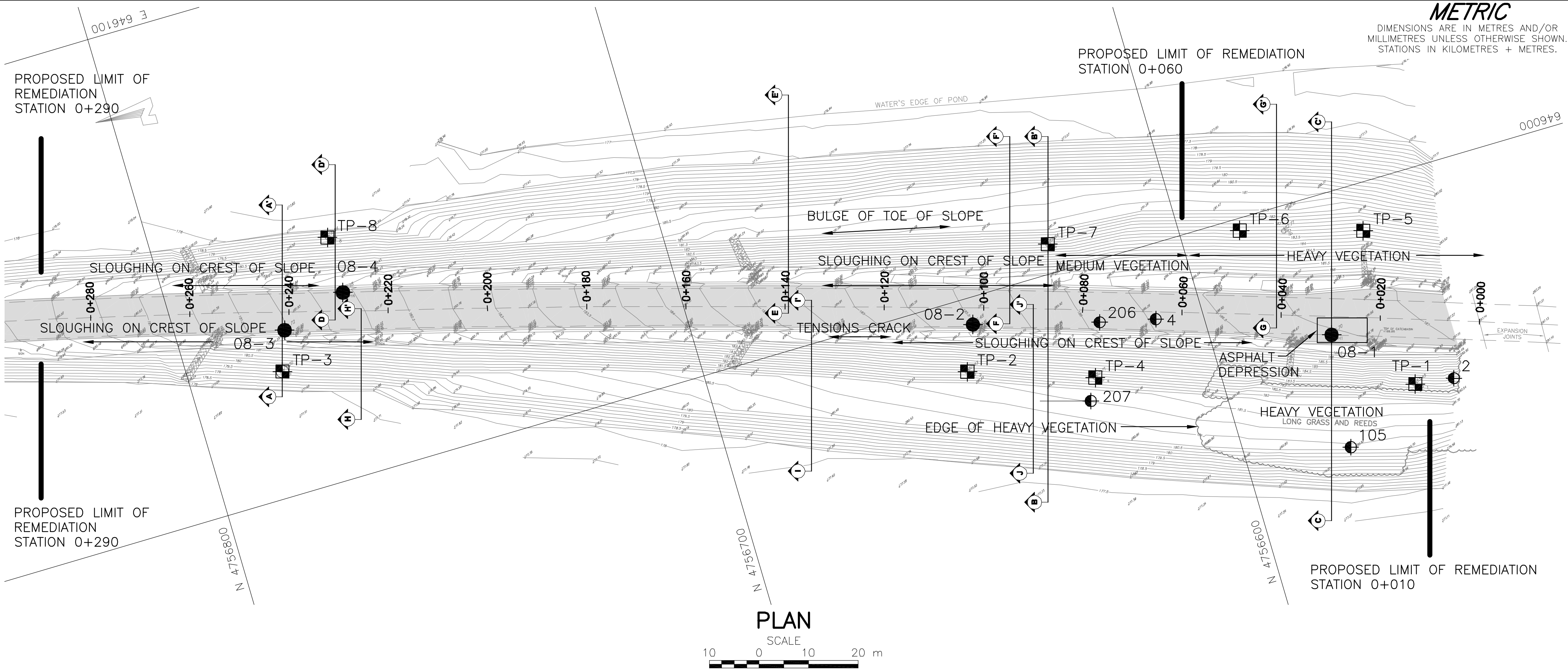
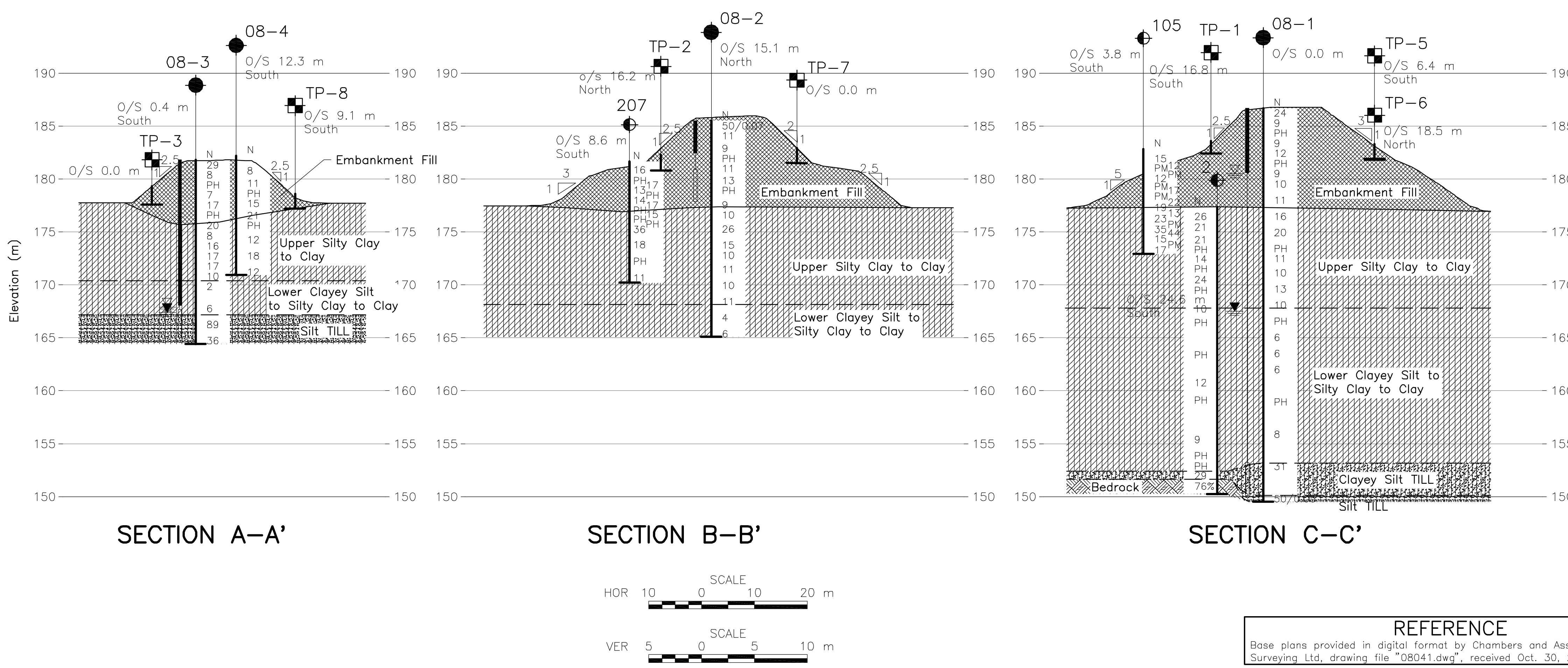




CONT No. 2008–E–0013

WP No.

SHEET

HWY 140 – CNR OVERPASS

North Embankment and Approach

BOREHOLE LAYOUT AND SOIL STRATA

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA

KEY PLAN
NOT TO SCALE

LEGEND

- Borehole – Current Investigation (Golder 2008)
- Test Pit – Current Investigation (Golder 2008)
- Borehole – Previous Investigation (MTO 1968, 1972)
- Seal
- Piezometer
- N — Standard Penetration Test Value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in piezometer, measured on October 21, 2008
- WL upon completion of drilling

No.	ELEVATION	CO–ORDINATES	
		NORTHING	EASTING
08–1	186.7	4756576.3	645972.2
08–2	185.5	4756645.0	645994.6
08–3	181.8	4756778.5	646032.6
08–4	182.2	4756765.0	646036.6
TP–1	179.3	4756781.3	646024.8
TP–2	182.3	4756648.8	645985.8
TP–3	183.6	4756562.9	645958.0
TP–4	182.8	4756624.4	645977.4
TP–5	183.3	4756564.2	645990.6
TP–6	183.0	4756588.1	645997.6
TP–7	182.9	4756626.0	646005.9
TP–8	178.6	4756764.9	646048.2
2	177.4	4756555.1	645956.9
4	177.4	4756609.4	645985.3
105	182.9	4756579.0	645949.4
206	185.9	4756620.4	645987.8
207	181.7	4756626.6	645973.1

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

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

The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Base plans provided in digital format by Chambers and Associates Surveying Ltd, drawing file "08041.dwg", received Oct. 30, 2008.

NO.	DATE	BY	REVISION

Geocres No. 30L14–50

HWY. 140		PROJECT NO. 08–1111–0031	DIST.
SUBM'D. MWK	CHKD. MWK	DATE: 14–Aug–2009	SITE:
DRAWN: DD	CHKD. JPD	APPD.	DWG. 1

TABLE 1
EVALUATION OF REMEDIATION ALTERNATIVES – HIGHWAY 140/CNR OVERPASS NORTH EMBANKMENT AND APPROACH

Remediation Option	*Rank*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks/Consequences*
Slope Flattening with Granular B, Type I or Granular A (both with not more than 5 percent passing the number 200 sieve) (2.5H:1V or flatter) (with deep benches into existing embankment, 2 m wide x 1 m high)	1	- Straight forward construction. - Removes weakened material on surface of existing slopes. - New slope flattening material keyed in very well to existing embankment. - Minimal disruption to traffic.	- Additional clayey spoil material generated from deep benching. - Somewhat more difficult to construct than granular buttress or blanket on side slopes. - Slight risk to traffic due to benching in existing slope.	- Low to medium construction and materials cost. - Higher cost than granular blanket or buttress options due to additional material required in deep benches.	- Very low risk of future stability issues. - Would induce some minor additional settlement due to increased loading of foundation soils. - Some monitoring during construction (i.e. visual) may be required to manage slightly higher risk to traffic.
Slope Flattening with Granular B, Type I or Granular A (both with not more than 5 percent passing the number 200 sieve) (2.5H:1V) (buttress above toe berm)	2	- Straight forward construction. - Minimal disruption and low risk to traffic. - Minimal clayey spoil material generated as a result of construction.	- A portion of the existing weakened surficial slope material will remain in place.	- Low to medium construction and material costs.	- Low risk of further stability issues, however some risk of performance issues (i.e. localized sloughing requiring maintenance) exists as a result of the weakened zone near surface of the slope remaining in place. -Would induce some minor additional settlement due to increased loading of foundation soils.
Slope Flattening with Silty Clay (3.5H:1V)	3	- Straight forward construction. - Minimal disruption and low risk to traffic. - Minimal clayey spoil material generated as a result of construction. - May be possible to source slope flattening material locally/close to site.	- A portion of the existing weakened surficial slope material will remain in place. - Non-free draining slope flattening material could still be subject to some localized surficial sloughing. - Small amount of land acquistion may be necessary to accommodate the wider embankment footprint.	- Low construction and material costs. - Potential for additional costs associated with land acquistion.	- Low risk of further stability, however some risk of performance issues (i.e. localized sloughing requiring maintenance) as a result of the weakened zone near surface of the slope and due to non-free draining material. - Would induce some additional settlement due to increased loading of foundation soils.

Granular B, Type I or Granular A Blanket on Side Slopes (both with not more than 5 percent passing the number 200 sieve) (2H:1V, 1.2 m thick) (above toe berm)	4	- Straight forward construction. - Smaller volume of slope flattening material required. - Minimal disruption and low risk to traffic. - Minimal clayey spoil material generated as a result of construction.	- A portion of the existing weakened surficial slope material will remain in place.	- Low to medium construction and material costs.	- Low to medium risk of further stability issues, however some risk of performance issues (i.e. localized sloughing requiring maintenance) exists as a result of the weakened zone near surface of the slope and due to the relatively steeper final slope profile. -Would induce some minor additional settlement due to increased loading of foundation soils.
Partial Removal and Replacement with Granular B, Type I or Granular A (both with not more than 5 percent passing the number 200 sieve) (1.2 m deep into embankment) (constructed in strips of limited width) (No Slope Flattening)	5	- Smaller volume of new / replacement fill material required. - Geometry of existing embankment remains relatively unchanged.	- Significant volume of additional clayey spoil material generated from excavation into existing embankment. - More difficult construction operation; requires excavation and backfilling in short sections to maintain stability of existing embankment. - Slight risk to traffic due to excavation into existing slope. - Disruption to traffic (i.e. short lane closures) will be required.	- Medium construction and materials cost. - Higher cost than granular blanket or buttress options due to construction in stages and material disposal costs.	- Low to medium risk of future stability issues, however some risk of localized performance issues due to the relatively steeper final slope profile. - Monitoring of stability during construction (i.e. visual) will be required to manage higher risk to traffic.
Partial Sub-Excavation and Reconstruction with Geogrid (wrapped face) (6.5 m embedment, silty clay backfill) (constructed in strips of limited width)	6	- Very little to no new fill material required for construction (since existing clayey fill re-used). - Minimal clayey spoil material generated as a result of construction. - Geometry of existing embankment remains relatively unchanged. - Innovative solution.	- More difficult construction operation; requires deeper excavation, geogrid placement and backfilling in short sections to maintain stability of existing embankment. - Higher risk to traffic due to excavation into existing slope. - Disruption to traffic (i.e. lane closures) will be required.	- Medium to high initial construction and material costs. - Additional costs likely required in future for maintenance/repair of wrapped geogrid facing.	- Medium risk of further surficial stability problems due to use of non-free draining material and possible poor performance and/or durability of geogrid at wrapped face. - Greater QA/QC requirements during installation of geogrid. - Monitoring of stability during construction (i.e. visual) will be required to manage higher risk to traffic.

Slope Drainage System (counterfort drains 1.5 m deep, 10 m to 20 m spacing, laid out in herringbone pattern)	7	- Very little new fill material required for construction. - Geometry of existing embankment remains relatively unchanged. - Straight forward construction. - Minimal disruption and low risk to traffic.	- Some volume of clayey spoil material generated from trench excavation into existing embankment. - Some annual maintenance may be required to ensure functionality of drains.	- Low to medium initial cost. - Potential for on-going maintenance costs.	- Medium risk of further surficial stability problems due to stabilization relying only on drainage for increased stability. - Annual inspections and maintenance to repair localized sloughing and/or drains may be required.
Slope Regrading and Vegetation	8	- Little to no new fill material required for construction. - Geometry of existing embankment remains relatively unchanged. - Straight forward construction. - Minimal disruption and low risk to traffic.	- Likely only a temporary solution. - Annual maintenance likely required to inspect and monitor growth/continuity of vegetation and for reinstatement in localized areas.	- Very low initial cost. - Potential for on-going maintenance/re-seeding costs.	- Medium to High risk of further surficial stability problems due to stabilization relying only on increased vegetation (deep root mass) for increased stability. - Annual maintenance to repair localized sloughing may be required.
Do Nothing (continue annual maintenance)	N/R	- No initial cost.	- Does not eliminate stability problems. - Leaning guard rail represents a risk to the safety of the travelling public	- No initial cost. - High long term cost due to continued annual maintenance.	- High risk of on-going surficial stability problems. - Risk of severity of surficial stability increasing. - Continued annual maintenance to repair localized sloughing will be required. -Potential source of liability to MTO due to safety risk associated with leaning guard rail.

Cement Soil Mixing (discrete columns on grid, or, full sub-excavate, mix and replace in strips of limited width)	N/R	- Very little to no new fill material required for construction. - Minimal clayey spoil material generated as a result of construction. - Geometry of existing embankment remains relatively unchanged. - Innovative solution.	- High material and labour costs may be prohibitive. -Difficult to ensure proper field mixing and therefore no guarantee of strength increase. - Preliminary laboratory tests indicate technique may not be feasible in this soil type.	- Very high construction costs. - Extra costs for additional engineering and laboratory testing will be required.	- Medium to High risk of further surficial stability problems due to difficulty to achieve uniform field mixing. - High level of QA/QC required during construction; performance is highly dependent on quality of construction. - Maintenance to repair localized sloughing may be required.
Slope Cover with Rock Fill and Concrete	N/R	- Minimal clayey spoil material generated as a result of construction. - Geometry of existing embankment remains relatively unchanged. - Minimizes future infiltration from rain and snowmelt.	- Not aesthetically pleasing. - Cover could be susceptible to damage (i.e. cracking) from frost heave and settlement which would decrease effectiveness of solution. -Annual inspection and maintenance likely required to check performance and/or repair localized areas.	- High material costs. - Potential for on-going maintenance costs.	- Medium to High risk of further surficial stability problems.. - Maintenance to repair covering and/or localized sloughing may be required.

APPENDIX A

Record of Boreholes

LIST OF SYMBOLS

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

I.	**GENERAL**
π	3.1416
in 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
υ	poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - \mu$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
μ	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 ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

(a)	**Index Properties (continued)**
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 (over-consolidated 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	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d)	**Shear Strength**
τ_p, τ_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 ($\phi = 0$ 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

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

Notes: 1 $\tau = c' + \sigma' \tan \phi'$
2 shear strength = (compressive strength)/2

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
SS	Split-spoon
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 Cone 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, 60º 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

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project 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_u, s_u kPa	c_u, s_u 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 limit
w_l	liquid limit
C	consolidation (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 (LV-laboratory vane test)
γ	unit weight

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

Golder Associates

RECORD OF BOREHOLE No 08-1

 METRIC

PROJECT 08-1111-0031
W.O. 2008-E-0013
LOCATION N 4756576.3 ;E 645972.2
ORIGINATED BY MWK
DIST HWY 140
BOREHOLE TYPE Power Auger , 108 mm I.D. Hollow-Stem Augers
COMPILED BY MWK
DATUM Geodetic
DATE October 7, 2008
CHECKED BY JPD

ELEV DEPTH	SOIL PROFILE DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	DYNAMIC CONE PENETRATION / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
186.7	GROUND SURFACE								
0.0	ASPHALT								
0.2	Sand and gravel, trace to some silt (FILL) Compact Brown/grey Moist	1	SS	24					
185.9 / 0.8	Clay, some silt, trace sand, trace gravel (FILL) Stiff to very stiff Brown Moist	2	SS	9	186				
		3	TO	PH	185		52		
		4	SS	9	184	>120, >120			
		5	SS	12	183				
		6	TO	PH	182				
		7	SS	9	181	>120, >120			
		8	SS	10	180	>120, >120			
	Contains organics between about 8.4 m and 9.0 m depth	9	SS	11	178				
177.7 / 9.0	SILTY CLAY to CLAY, trace sand, trace gravel Very stiff to stiff Brown Moist to wet Contains organics between 9.9 m and 10.5 m depth				177	>120			
		10	SS	16	176	>120, >120			
		11	SS	20	175				
		12	TO	PH	174				
		13	SS	11	173				
					172				

Continued Next Page

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

MIS-MTO 001 08-1111-0031.GPJ GAL-MISS.GDT 8/13/09 DD

Golder Associates

RECORD OF BOREHOLE No 08-1

 METRIC

PROJECT 08-1111-0031

W.O. 2008-E-0013 LOCATION N 4756576.3 ;E 645972.2 ORIGINATED BY MWK

DIST HWY 140 BOREHOLE TYPE Power Auger , 108 mm I.D. Hollow-Stem Augers COMPILED BY MWK

DATUM Geodetic DATE October 7, 2008 CHECKED BY JPD

-- CONTINUED FROM PREVIOUS PAGE --

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES
	SILTY CLAY to CLAY, trace sand, trace gravel Very stiff to stiff Brown Moist to wet	14	SS	10
		15	SS	13
		16	SS	10
167.8 18.9	SILTY CLAY to CLAYEY SILT, trace sand, trace gravel Firm to stiff Brown Wet			
		17	TO	PH
		18	SS	6
		19	SS	6
		20	SS	6
		21	TO	PH

SOIL PROFILE | STRAT PLOT | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

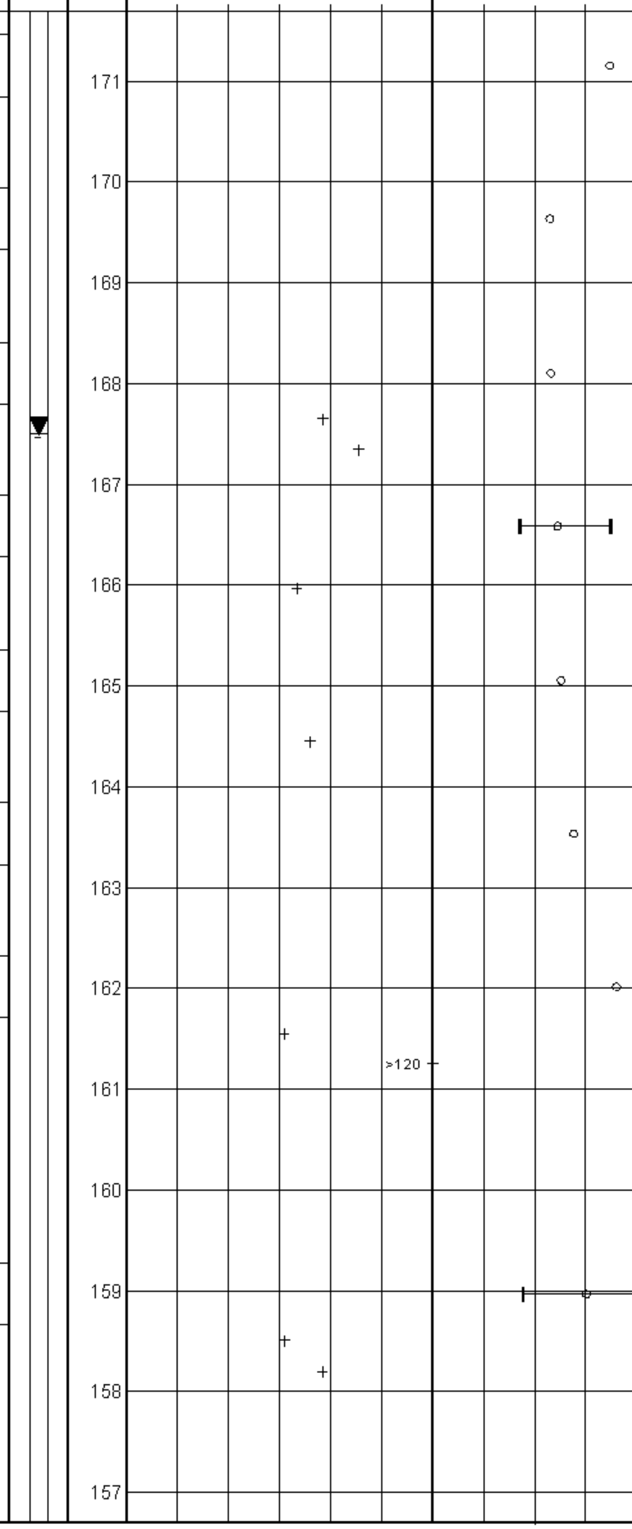
SHEAR STRENGTH kPa
O UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × REMOULDED
20 40 60 80 100

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

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



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

RECORD OF BOREHOLE No 08-1

 METRIC

PROJECT 08-1111-0031
W.O. 2008-E-0013
LOCATION N 4756576.3 ;E 645972.2
ORIGINATED BY MWK
DIST HWY 140
BOREHOLE TYPE Power Auger , 108 mm I.D. Hollow-Stem Augers
COMPILED BY MWK
DATUM Geodetic
DATE October 7, 2008
CHECKED BY JPD

-- CONTINUED FROM PREVIOUS PAGE --

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES
	SILTY CLAY to CLAYEY SILT, trace sand, trace gravel Firm to stiff Brown Wet	22	SS	8
153.2 33.5	CLAYEY SILT, some sand, some gravel (TILL) Hard Brown Wet	23	SS	31
150.1 36.6	Sandy SILT, some gravel, trace to some clay (TILL) Very dense Grey Wet	24	SS	50/0.05
149.5 37.2	END OF BOREHOLE			

NOTES:

1. Water level at 6.2 m depth in monitoring well on completion of installation .

2. Water level in monitoring well measured at 19.2 m (Elev. 167.5 m) depth on October 21, 2008.

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (156, 155, 154, 153, 152, 151, 150); 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 (%) (10 20 30); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

+3, ×3: Numbers refer to Sensitivity   O3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 08-2

 METRIC

PROJECT 08-1111-0031
W.O. 2008-E-0013 LOCATION N 4756645.0 ;E 645994.6 ORIGINATED BY MWK
DIST HWY 140 BOREHOLE TYPE Power Auger , 108 mm I.D. Hollow-Stem Augers COMPILED BY MWK
DATUM Geodetic DATE October 6, 2008 CHECKED BY JPD

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)
185.5	GROUND SURFACE				
0.0	ASPHALT	1	SS	50/0.07	
0.2	Sand and gravel, trace to some silt (FILL) Very dense Grey/Brown Moist				
184.7 0.8	Clay, some silt, trace sand, trace gravel (FILL) Stiff to very stiff Brown Moist	2	SS	11	
	Occasional silt seam between about 2.3 m and 2.9 m depth	3	SS	9	
		4	TO	PH	
		5	SS	11	
		6	SS	13	
		7	TO	PH	63
		8	SS	9	
177.1 8.4	SILTY CLAY to CLAY, trace sand, trace gravel Stiff to very stiff Brown Moist to wet Contains organics between about 8.4 m and 9.0 m depth	9	SS	10	
		10	SS	26	
		11	SS	15	
		12	SS	10	
		13	SS	11	

Shear strength (field vane): >120 at approx. elev. 183.8, 183.5, 180.8, 180.4, 178.6, 178.3, 176.3, 176.0, 174.8

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (185 – 171); 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 (%) 10 20 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

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

RECORD OF BOREHOLE No 08-2

 METRIC

PROJECT 08-1111-0031
W.O. 2008-E-0013 LOCATION N 4756645.0 ;E 645994.6 ORIGINATED BY MWK
DIST HWY 140 BOREHOLE TYPE Power Auger , 108 mm I.D. Hollow-Stem Augers COMPILED BY MWK
DATUM Geodetic DATE October 6, 2008 CHECKED BY JPD

-- CONTINUED FROM PREVIOUS PAGE --

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY CLAY to CLAY, trace sand, trace gravel; Stiff to very stiff; Brown; Moist to wet	14	SS	10	170	o		
					169			
168.1		15	SS	11		o		
17.4	SILTY CLAY to CLAYEY SILT, trace sand, trace gravel; Firm to stiff; Brown; Moist to wet				168			
		16	SS	4	167	o		
					166			
165.1		17	SS	6		o		
20.4	END OF BOREHOLE							

NOTES:

1. Borehole dry upon completion of drilling.

2. Monitoring well dry on completion of installation.

3. Monitoring well dry to 7.6 m depth (Elev. 177.9 m) on October 21, 2008.

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

Golder Associates

RECORD OF BOREHOLE No 08-3

 METRIC

PROJECT 08-1111-0031			
W.O. 2008-E-0013	LOCATION N 4756778.5 ;E 646032.6	ORIGINATED BY MWK	
DIST	HWY 140	BOREHOLE TYPE Power Auger, 108 mm I.D. Hollow-Stem Augers	COMPILED BY MWK
DATUM Geodetic	DATE September 29, 2008	CHECKED BY JPD	

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES
181.8	GROUND SURFACE			
0.0	ASPHALT			
0.2	Sand and gravel, trace to some silt (FILL) Compact Brown/grey Moist	1	SS	29
181.0 / 0.8	Clay, some silt, trace sand, trace gravel (FILL) Firm to very stiff Brown Moist	2	SS	8
		3	TO	PH
		4	SS	7
	Contains organics between about 3.8 m and 5.2 m depth	5	SS	17
		6	TO	PH
		7	SS	20
175.7 / 6.1	SILTY CLAY to CLAY, trace sand, trace gravel Stiff to very stiff Brown Moist	8	SS	8
		9	SS	16
		10	SS	17
		11	SS	17
		12	SS	10
170.4 / 11.4	SILTY CLAY to CLAYEY SILT Firm to stiff Brown Wet	13	SS	2
		14	SS	6
167.2 / 14.6				

Continued Next Page

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (181–167); 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 (%) 10, 20, 30; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Plotted values: 54, 54, >120, >120, >120, >120, 47.3, >120, >120

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

RECORD OF BOREHOLE No 08-3

 METRIC

PROJECT 08-1111-0031

W.O. 2008-E-0013 | LOCATION N 4756778.5 ;E 646032.6 | ORIGINATED BY MWK

DIST | HWY 140 | BOREHOLE TYPE Power Auger , 108 mm I.D. Hollow-Stem Augers | COMPILED BY MWK

DATUM Geodetic | DATE September 29, 2008 | CHECKED BY JPD

ELEV DEPTH	SOIL PROFILE: DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---						
	SILT, some sand, some gravel, some clay (TILL) Very dense Brown Moist	15	SS	89	166		
165.0 16.8	SAND, some silt, trace gravel (TILL) Dense Brown Wet	16	SS	36	165		
164.4 17.4	END OF BOREHOLE						

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; 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 (%) 10 20 30; UNIT WEIGHT γ kN/m³.

NOTES:

1. Water level at a 15.7 m depth inside augers on completion of drilling.

2. Water level at 13.9 m depth in monitoring well on completion of installation.

3. Water level in monitoring well measured at 14.4 m depth (Elev. 167.4 m) on October 21, 2008.

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

RECORD OF BOREHOLE No 08-4

 METRIC

PROJECT 08-1111-0031
W.O. 2008-E-0013 LOCATION N 4756765.0 ;E 646036.6 ORIGINATED BY MWK
DIST HWY 140 BOREHOLE TYPE Power Auger , 108 mm I.D. Hollow-Stem Augers COMPILED BY MWK
DATUM Geodetic DATE October 7, 2008 CHECKED BY JPD

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	REMARKS
182.2	GROUND SURFACE							
0.0	ASPHALT							
0.2	Sand and gravel, trace to some silt (FILL) Very dense Grey Moist	1	SS	50/0.15	182			
181.4 0.8	Clay, some silt, trace sand, trace gravel (FILL) Firm to very stiff Brown Moist	2	SS	8	181	>120, >120		
		3	SS	11	180			
		4	TO	PH	179			
		5	SS	15	178	>120, >120	53	
	Contains organics between about 5.3 m and 2.9 m depth	6	SS	21	177			
176.3 5.9	SILTY CLAY to CLAY, trace sand, trace gravel Stiff to very stiff Brown Moist	7	TO	PH	176	>120, >120		
		8	SS	12	175, 174	>120	42	
	Occasional grey, silty sand seams between about 9.1 m and 9.6 m depth	9	SS	18	173, 172	>120		
170.9		10	SS	12	171			
11.3	END OF BOREHOLE							

NOTE:

1. Borehole dry upon completion of drilling.

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

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

APPENDIX B

Field Test Pit Logs and Figures

FIELD TEST PIT LOG

JOB NUMBER:	08-1111-0031	**JOB NAME:**	MTO / Highway 140 Embankment / Welland	**DATE:**	October 3, 2008
TEST PIT NUMBER:	TP-1	**LOCATION:**	N 4756781.3 E 646024.8	**ELEVATION:**	179.3 m
MACHINE TYPE:	CAT Mini Excavator	**TEST PIT SIZE:**	Approx. 1 m x 1.5 m	**DATUM:**	Geodetic
TEMP/WEATHER:	Sunny, 10^{o}C	**CONTRACTOR:**	Roadside Rentals Inc.		

Depth			**In Situ Density Tests**			**Samples**		**M-3 Vane**		**Remarks / Lab Test Results**
From (m)	**To (m)**	**Soil Description**	**Depth (m)**	**Dry Density (kg/m^3)**	**Water Content (%)**	**No.**	**Depth (m)**	**Depth (m)**	**(kPa)**	**GR/SA/SI/CL (%) Atterberg Limits (%) w/c %**
0.0	1.1	Sand and Gravel, trace to some silt, trace clay (Fill), moist, grey.	0	1680	4.5	1	0			3.3%
			0	1602	5.9					
			0	1656	4.8					
			0.3	1835	5.2					
						2	0.5			3.7%
			0.6	1800	6.9					
1.1	1.7	Clay, some silt, trace sand, trace gravel (Fill), firm, moist, brown	1.1	1535	21.4					
								1.2	33	17.4%
						3	1.3	1.3	40	PL=13.9 LL=18.6 PI=4.7
						4	1.5 - 1.7			29.3%
1.7		End of Test Pit								

Comments:

For additional details and test pit photos, see page 2 of 2.

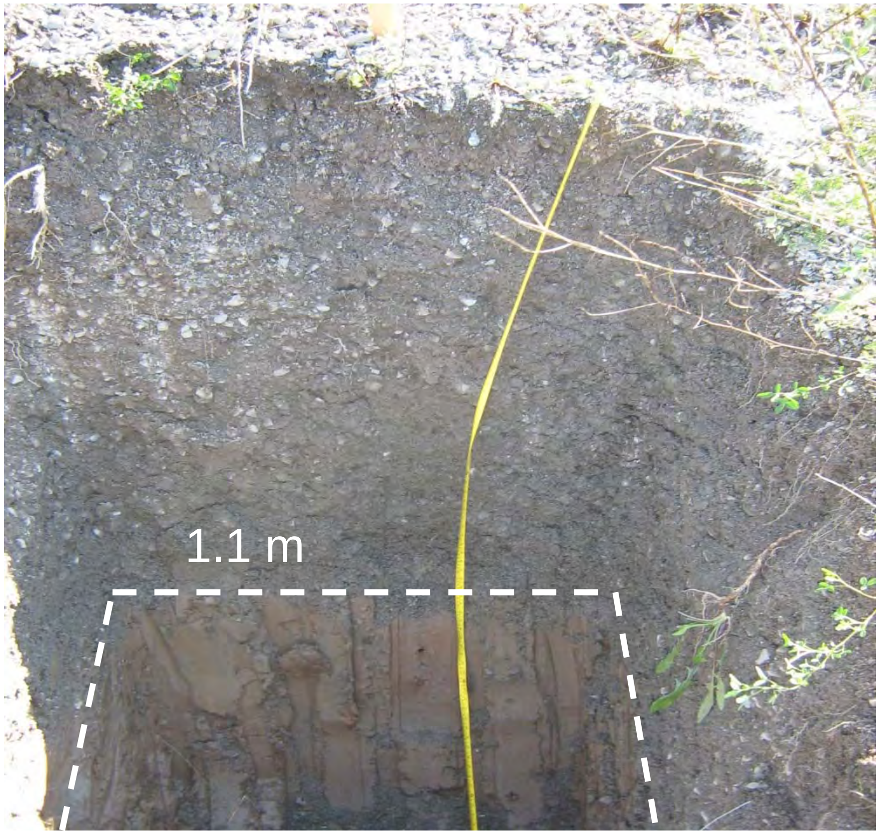
Water Conditions in Test Pit:

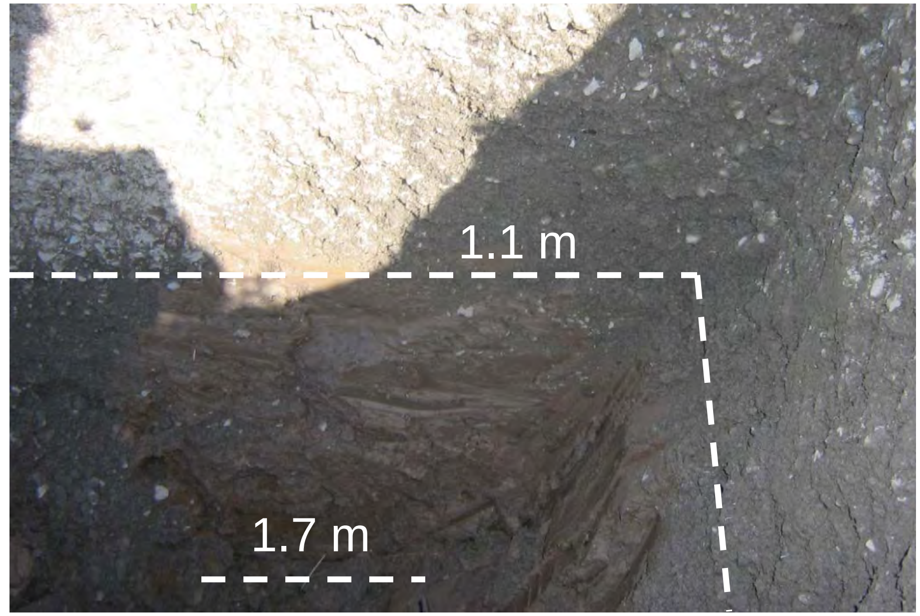
Moist soil at bottom of test pit. No seepage.

☒ Test Pit Dry

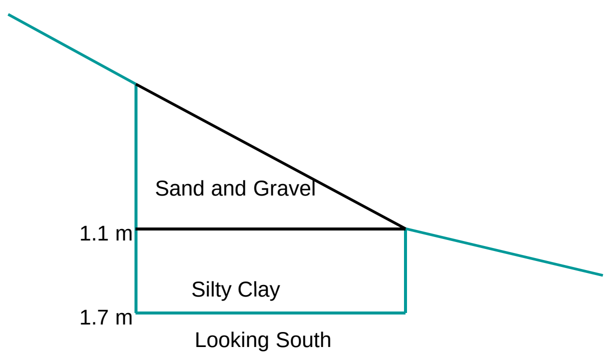
JOB No:	08-1111-0031
TEST PIT No.:	TP-1
ENGINEER:	MWK

Test Pit – TP-1





- No Seepage (dry pit)
- Final depth 1.7 m



Description/Notes:

0.0 m to 1.1 m	Sand and Gravel, trace to some silt, trace clay (Fill), moist, grey.
1.1 m to 1.7 m	Clay, some silt, trace sand, trace gravel (Fill), firm, moist, brown

M-3 Vane at 1.2 m depth
33 KPa

M-3 Vane at 1.3 m depth
40 KPa

Sample #4 - Shelby Tube sample taken from 1.5 m to 1.7 m

Job number: 08-1111-0031
Date: October 2008
Engineer: MWK

JOB NUMBER:	08-1111-0031	**JOB NAME:**	MTO / Highway 140 Embankment / Welland	**DATE:**	October 3,2008
TEST PIT NUMBER:	TP-2	**LOCATION:**	N 4756648.8 E 645985.8	**ELEVATION:**	182.3 m
MACHINE TYPE:	CAT Mini Excavator	**TEST PIT SIZE:**	Approx. 1 m x 2.0 m	**DATUM:**	Geodetic
TEMP/WEATHER:	Sunny, 9^{o}C	**CONTRACTOR:**	Roadside Rentals Inc.		

Depth			**In Situ Density Tests**			**Samples**		**M-3 Vane**		**Remarks / Lab Test Results**
From (m)	**To (m)**	**Soil Description**	**Depth (m)**	**Dry Density (kg/m^3)**	**Water Content (%)**	**No.**	**Depth (m)**	**Depth (m)**	**(kPa)**	**GR/SA/SI/CL (%) Atterberg Limits (%) w/c %**
0.0	0.1	Topsoil								
0.1	1.5	Clay, some silt, trace sand, trace gravel (Fill), soft to firm, moist to wet, brown							21	
								0.15		
			0.3	1518	23.6			0.3	26	PL=23.6 LL=59.7 PI=36.1
								0.35	32	
								0.4	34	
								0.5	37	
								0.6	29	
								0.7	24	
								0.8	31	
								0.9	37	
						1	1.0	1	32	PL=23.6 LL=59.7 PI=36.1 23.8%
								1.1	37	
								1.2	39	
								1.3	38	17.8%
						2	1.4	1.4	40	16/37/17/46
			1.5	1459	27.0	3	1.5 – 1.7	1.5	40	25.4%
								1.6	41	
1.5		End of Test Pit								

Comments:

For additional details and test pit photos, see page 2 of 2.

Water Conditions in Test Pit:

Moist to wet soil near bottom of test pit. No seepage.

☒ Test Pit Dry

JOB No:	08-1111-0031
TEST PIT No.:	TP-2
ENGINEER:	MWK

Test Pit – TP-2

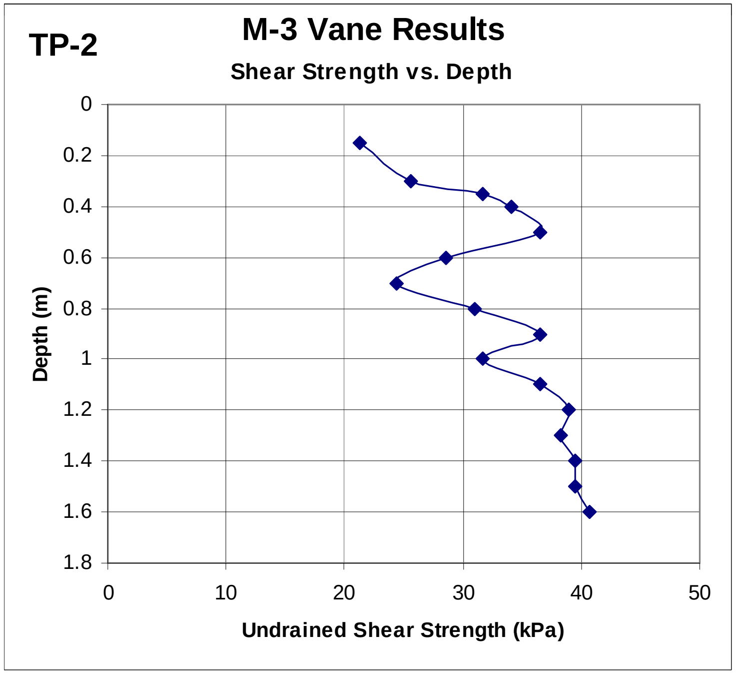
•No Seepage (dry pit)
•Final depth 1.5 m

Description/Notes:

0.0 m to 0.1 m TOPSOIL

0.1 m to 1.5 m Clay, some silt, trace sand, trace gravel (Fill), soft to firm, moist to wet, brown

Sample #3 - Shelby Tube sample taken from 1.5 m to 1.7 m



Job number: 08-1111-0031
Date: October 2008
Engineer: MWK

FIELD TEST PIT LOG

JOB NUMBER:	08-1111-0031	**JOB NAME:**	MTO / Highway 140 Embankment / Welland	**DATE:**	October 3, 2008
TEST PIT NUMBER:	TP-3	**LOCATION:**	N 4756562.9 E 645958.0	**ELEVATION:**	183.6 m
MACHINE TYPE:	CAT Mini Excavator	**TEST PIT SIZE:**	Approx. 1 m x 2.0 m	**DATUM:**	Geodetic
TEMP/WEATHER:	Sunny, 8^{o}C	**CONTRACTOR:**	Roadside Rentals Inc.		

Depth			**In Situ Density Tests**			**Samples**		**M-3 Vane**		**Remarks / Lab Test Results**
From (m)	**To (m)**	**Soil Description**	**Depth (m)**	**Dry Density (kg/m^3)**	**Water Content (%)**	**No.**	**Depth (m)**	**Depth (m)**	**(kPa)**	**GR/SA/SI/CL (%) Atterberg Limits (%) w/c %**
0.0	0.05	Topsoil								
0.05	0.2	Sand and Gravel, trace to s ome silt, trace clay (Fill), moist, grey	0.15	1953	8.9	1	0.15			Up-slope side of test pit only (see attached figure) 1.4%
0.2	1.2	Clay, some silt, trace sand, trace gravel (Fill), soft to firm, moist to wet, brown						0.3	16	
			0.4	1490	26.9			0.45	25	
								0.6	23	
								0.75	30	
						2	0.9	1	27	31.3%
			1.2	1443	28.0	3	1.2 – 1.4	1.15	37	30.2%
								1.3	39	PL=22.1 LL=56.5 PI=34.4
1.2		End of Test Pit								

Comments:

For additional details and test pit photos, see page 2 of 2.

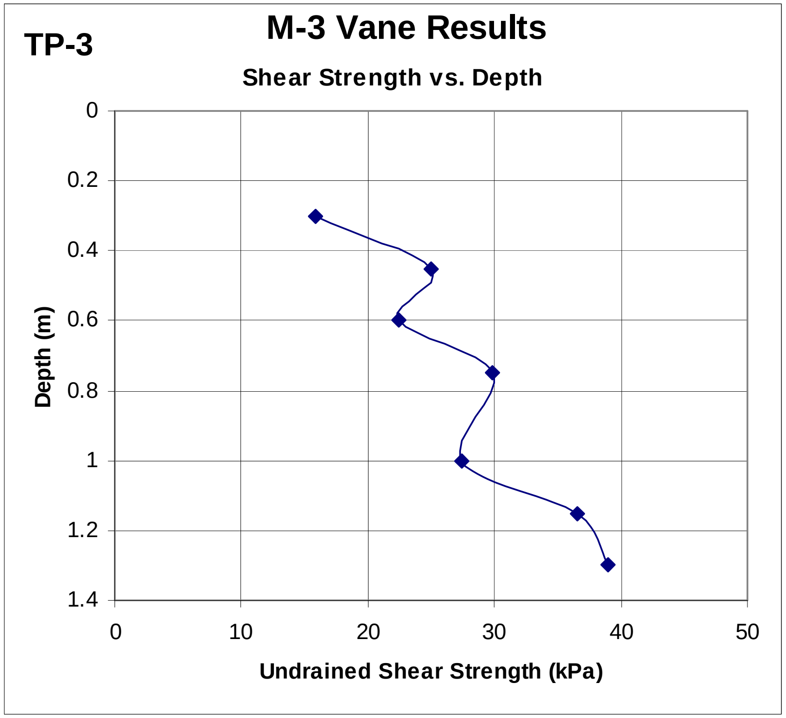
Water Conditions in Test Pit:

Water seeping into test pit at approx. 1.0 m depth.

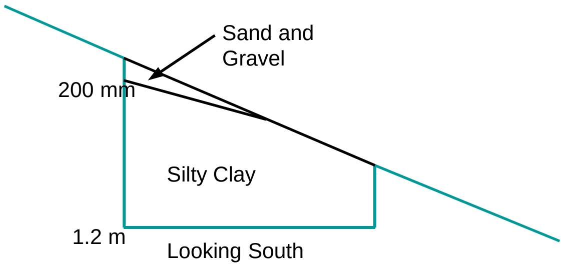
☐ Test Pit Dry

JOB No: 08-1111-0031
TEST PIT No.: TP-3
ENGINEER: MWK

Test Pit – TP-3



•Water seeping into test pit at approx. 1.0 m
•Final depth 1.2 m



Description/Notes:

0.0 m to 0.05 m TOPSOIL

0.05 m to 0.2 m Sand and Gravel, trace to s ome silt, trace clay (Fill), moist, grey

0.2 m to 1.2 m Clay, some silt, trace sand, trace gravel (Fill), soft to firm, moist to wet, brown.

Sample #3 - Shelby Tube sample taken from 1.2 m to 1.4 m

Job number: 08-1111-0031
Date: October 2008
Engineer: MWK

FIELD TEST PIT LOG

JOB NUMBER:	08-1111-0031	**JOB NAME:**	MTO / Highway 140 Embankment / Welland	**DATE:**	October 3, 2008
TEST PIT NUMBER:	TP-4	**LOCATION:**	N 4756624.4 E 645977.4	**ELEVATION:**	182.8 m
MACHINE TYPE:	CAT Mini Excavator	**TEST PIT SIZE:**	Approx. 1 m x 2.0 m	**DATUM:**	Geodetic
TEMP/WEATHER:	Sunny, 9°C	**CONTRACTOR:**	Roadside Rentals Inc.		

Depth			**In Situ Density Tests**			**Samples**		**M-3 Vane**		**Remarks / Lab Test Results**
From (m)	**To (m)**	**Soil Description**	**Depth (m)**	**Dry Density (kg/m^3)**	**Water Content (%)**	**No.**	**Depth (m)**	**Depth (m)**	**(kPa)**	**GR/SA/SI/CL (%) Atterberg Limits (%) w/c %**
0.0	0.05	Topsoil								
0.05	1.5	Clay, some silt, trace sand, trace gravel (Fill), firm, moist to wet, brown	0.3	1500	24.3			0.3	31	
								0.4	28	23.1%
						1	0.5	0.5	27	0/2/29/69
			0.6	1621	22.7			0.6	30	PL=22.6 LL=56.0 PI=33.4
								0.7	33	
								0.8	32	
			0.9	1539	26.1			0.9	32	
								1.0	34	
						2	1.1	1.1	37	24.0%
			1.2	1568	24.4			1.2	30	PL=23.2 LL=57.5 PI=34.3
								1.3	29	
								1.4	35	
						3	1.5 – 1.7	1.5	35	22.7%
								1.6	27	
								1.7	39	
								0.3	31	
1.5		End of Test Pit								

Comments:

For additional details and test pit photos, see page 2 of 2.

Water Conditions in Test Pit:

Moist soil at bottom of test pit. No seepage.

☒ Test Pit Dry

JOB No:	08-1111-0031
TEST PIT No.:	TP-4
ENGINEER:	MWK

Test Pit – TP-4

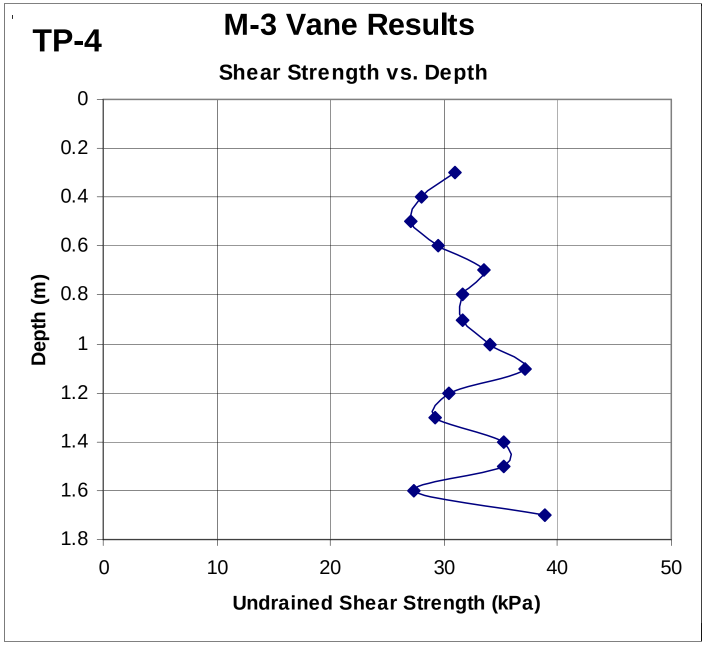
•No Seepage (dry pit)
•Final depth 1.5 m

Description/Notes:

0.0 m to 0.05 m TOPSOIL

0.05 m to 1.5 m Clay, some silt, trace sand, trace gravel (Fill), firm, moist to wet, brown

Sample #3 - Shelby Tube sample taken from 1.5 m to 1.7 m



Job number: 08-1111-0031
Date: October 2008
Engineer: MWK

FIELD TEST PIT LOG

JOB NUMBER:	08-1111-0031	**JOB NAME:**	MTO / Highway 140 Embankment / Welland	**DATE:**	October 3, 2008
TEST PIT NUMBER:	TP-5	**LOCATION:**	N 4756564.2 E 645990.6	**ELEVATION:**	183.3 m
MACHINE TYPE:	CAT Mini Excavator	**TEST PIT SIZE:**	Approx. 1 m x 2.0 m	**DATUM:**	Geodetic
TEMP/WEATHER:	Sunny, 13°C	**CONTRACTOR:**	Roadside Rentals Inc.		

Depth			**In Situ Density Tests**			**Samples**		**M-3 Vane**		**Remarks / Lab Test Results**
From (m)	**To (m)**	**Soil Description**	**Depth (m)**	**Dry Density (kg/m^3)**	**Water Content (%)**	**No.**	**Depth (m)**	**Depth (m)**	**(kPa)**	**GR/SA/SI/CL (%) Atterberg Limits (%) w/c %**
0.0	0.1	Topsoil								
0.1	1.0	Sand and Gravel, trace to s ome silt, trace clay (Fill), moist, grey	0.3 0.6	1842 1874	3.3 2.1	1	0.3			34/54/10/2 2.6%
1.0	1.4	Clay, some silt, trace sand, trace gravel (Fill), firm to stiff, moist, brown						1.1	57	Unable to push Shelby Tube at 1.4 m
								1.2	59	
						2	1.3	1.3	43	PL=23.1 LL=54.1 PI=31
								1.4	70	24.8%
1.4		End of Test Pit								

Comments:

For additional details and test pit photos, see page 2 of 2.

Water Conditions in Test Pit:

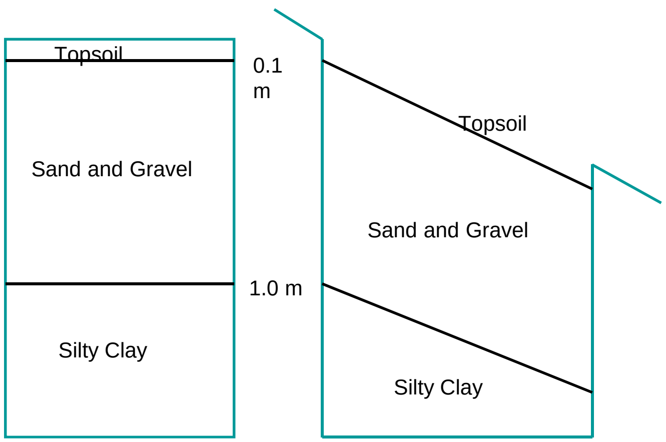
Moist soil at bottom of test pit. No seepage.

☒ Test Pit Dry

JOB No: 08-1111-0031
TEST PIT No.: TP-5

Test Pit – TP-5

•No Seepage (dry pit)
•Final depth 1.4 m



Looking West   Looking North

<u>Description/Notes:</u>

0.0 m to 0.1 m   TOPSOIL

0.1 m to 1.0 m   Sand and Gravel, trace to s ome silt, trace clay (Fill), moist, grey.

1.0 m to 1.4 m   Clay, some silt, trace sand, trace gravel (Fill), firm to stiff, moist, brown

M-3 Vane at 1.1 m depth 57 KPa
M-3 Vane at 1.2 m depth 59 KPa
M-3 Vane at 1.3 m depth 43 KPa
M-3 Vane at 1.4 m depth 70 KPa

Unable to push Shelby tube at 1.4 m

Job number: 08-1111-0031
Date: October 2008
Engineer: MWK

ENGINEER: MWK

FIELD TEST PIT LOG

JOB NUMBER:	08-1111-0031	**JOB NAME:**	MTO / Highway 140 Embankment / Welland	**DATE:**	October 3, 2008
TEST PIT NUMBER:	TP-6	**LOCATION:**	N 4756588.1 E 645997.6	**ELEVATION:**	183.0 m
MACHINE TYPE:	CAT Mini Excavator	**TEST PIT SIZE:**	Approx. 1 m x 2.0 m	**DATUM:**	Geodetic
TEMP/WEATHER:	Sunny, 13°C	**CONTRACTOR:**	Roadside Rentals Inc.		

Depth			**In Situ Density Tests**			**Samples**		**M-3 Vane**		**Remarks / Lab Test Results**
From (m)	**To (m)**	**Soil Description**	**Depth (m)**	**Dry Density (kg/m^3)**	**Water Content (%)**	**No.**	**Depth (m)**	**Depth (m)**	**(kPa)**	**GR/SA/SI/CL (%) Atterberg Limits (%) w/c %**
0.0	0.1	Topsoil								
0.1	0.5	Sand and Gravel, trace to s ome silt, trace clay (Fill), dry, grey	0.5	1937	3.4	1	0.4			1.5%
0.5	0.9	Sandy gravel, trace silt, trace clay, dry, grey	0.75	1748	4.1	2	0.7			75/22/2/1 0.2%
0.9	1.2	Clay, some silt, trace sand, trace gravel (Fill), firm to stiff, moist to wet, brown				3	1.1	1.0 1.1 1.2 1.3	46 48 59 61	2/3/30/65 19.1% Unable to push Shelby Tube at 1.2 m
1.2		End of Test Pit								

Comments:

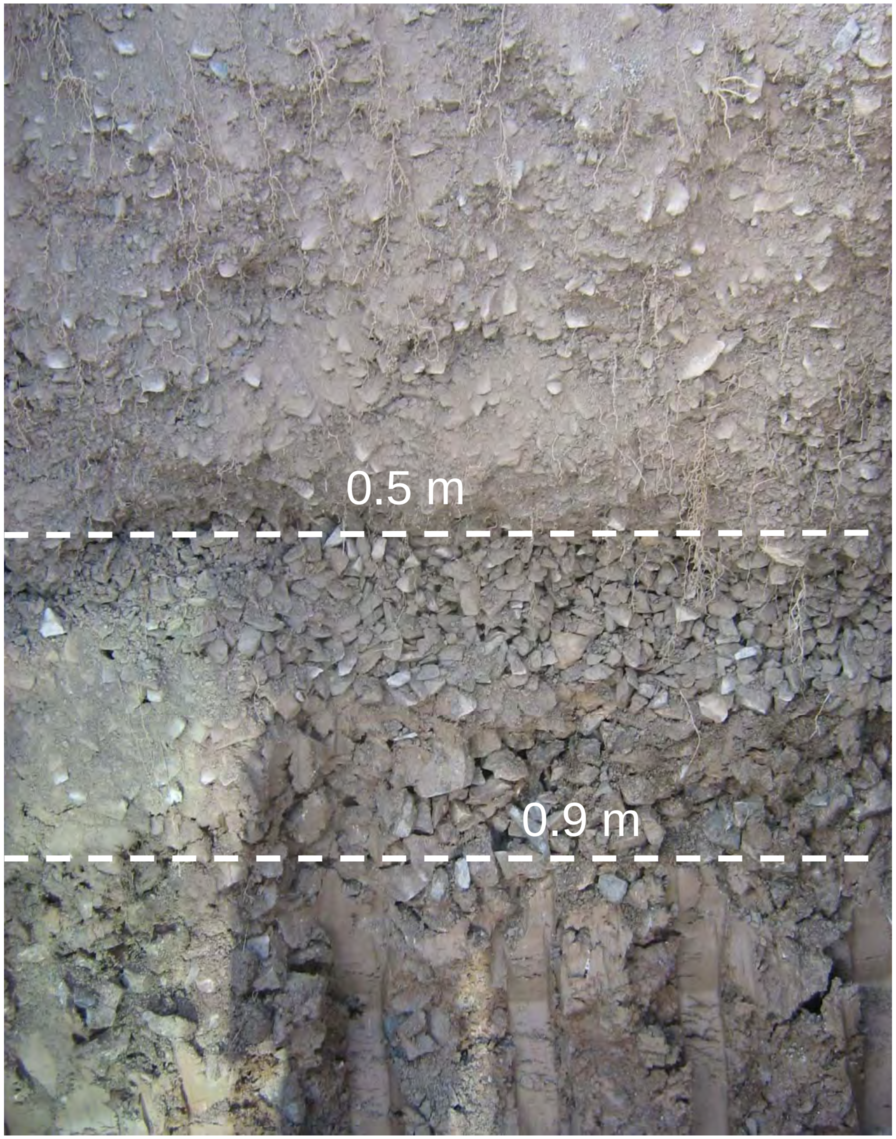
For additional details and test pit photos, see page 2 of 2.

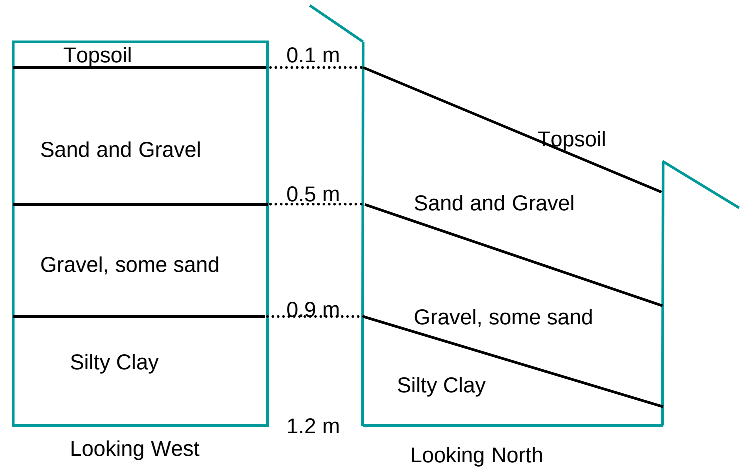
Water Conditions in Test Pit:

Moist to wet soil below 0.9 m. No seepage.

☒ Test Pit Dry

Test Pit – TP-6





•No Seepage (dry pit)
•Final depth 1.2 m

Description/Notes:

0.0 m to 0.1 m	TOPSOIL
0.1 m to 0.5 m	Sand and Gravel, trace to s ome silt, trace clay (Fill), dry, grey
0.5 m to 0.9 m	Sandy gravel, trace silt, trace clay, dry, grey
0.9 m to 1.2 m	Clay, some silt, trace sand, trace gravel (Fill), firm to stiff, moist to wet, brown

M-3 Vane at 1.0 m depth 46 KPa
M-3 Vane at 1.1 m depth 48 KPa
M-3 Vane at 1.2 m depth 59 KPa
M-3 Vane at 1.3 m depth 61 KPa

Unable to push Shelby tube at 1.2 m

Job number: 08-1111-0031
Date: October 2008
Engineer: MWK

FIELD TEST PIT LOG

JOB NUMBER:	08-1111-0031	**JOB NAME:**	MTO / Highway 140 Embankment / Welland	**DATE:**	October 3, 2008
TEST PIT NUMBER:	TP-7	**LOCATION:**	N 4756626.0 E 646005.9	**ELEVATION:**	182.9 m
MACHINE TYPE:	CAT Mini Excavator	**TEST PIT SIZE:**	Approx. 1 m x 2.0 m	**DATUM:**	Geodetic
TEMP/WEATHER:	Sunny, 12^{o}C	**CONTRACTOR:**	Roadside Rentals Inc.		

Depth			**In Situ Density Tests**			**Samples**		**M-3 Vane**		**Remarks / Lab Test Results**
From (m)	**To (m)**	**Soil Description**	**Depth (m)**	**Dry Density (kg/m^3)**	**Water Content (%)**	**No.**	**Depth (m)**	**Depth (m)**	**(kPa)**	**GR/SA/SI/CL (%) Atterberg Limits (%) w/c %**
0.0	0.05	Topsoil, trace gravel.								
0.05	1.4	Clay, some silt, trace sand, trace gravel (Fill), firm, moist, brown						0.1	26	
								0.2	29	
			0.3	1523	21.0			0.4	26	
								0.5	25	
								0.6	32	
								0.7	31	
								0.8	35	
								0.9	35	
								1.0	38	
								1.2	32	
								1.3	39	
						1	1.4 – 1.6	1.4	35	24.0%
								1.5	37	
1.4		End of Test Pit								

Comments:

For additional details and test pit photos, see page 2 of 2.

Water Conditions in Test Pit:

Moist soil at bottom of test pit. No seepage.

☒ Test Pit Dry

JOB No: 08-1111-0031
TEST PIT No.: TP-7
ENGINEER: MWK

Test Pit – TP-7

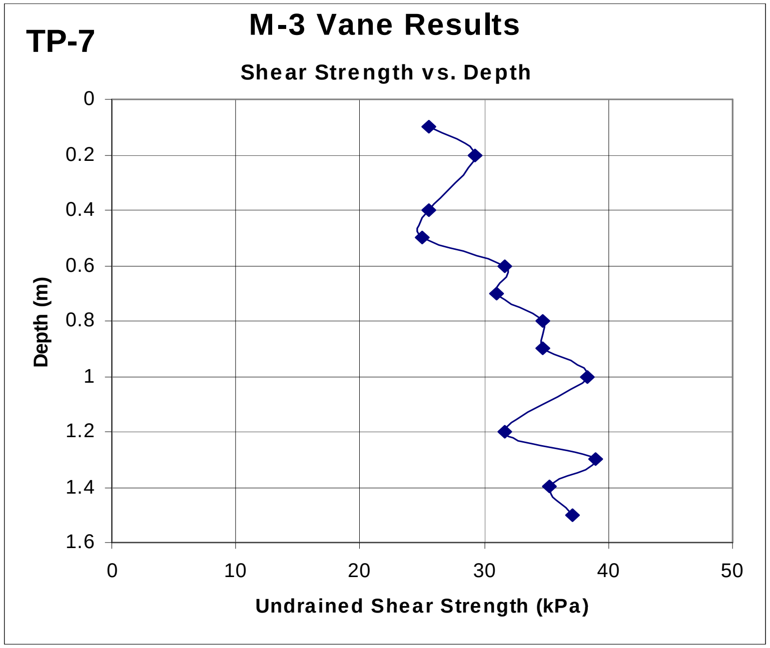
•No Seepage (dry pit)
•Final depth 1.4 m

Description/Notes:

0.0 m to 0.05 m TOPSOIL

0.05 m to 1.4 m Clay, some silt, trace sand, trace gravel (Fill), firm, moist, brown

Sample #1 - Shelby Tube sample taken from 1.4 m to 1.6 m



Job number: 08-1111-0031
Date: October 2008
Engineer: MWK

FIELD TEST PIT LOG

JOB NUMBER:	08-1111-0031	**JOB NAME:**	MTO / Highway 140 Embankment / Welland	**DATE:**	October 3, 2008
TEST PIT NUMBER:	TP-8	**LOCATION:**	N 4756764.9 E 646048.2	**ELEVATION:**	178.6 m
MACHINE TYPE:	CAT Mini Excavator	**TEST PIT SIZE:**	Approx. 1 m x 2.0 m	**DATUM:**	Geodetic
TEMP/WEATHER:	Sunny, 10^{o}C	**CONTRACTOR:**	Roadside Rentals Inc.		

Depth			**In Situ Density Tests**			**Samples**		**M-3 Vane**		**Remarks / Lab Test Results**
From (m)	**To (m)**	**Soil Description**	**Depth (m)**	**Dry Density (kg/m^3)**	**Water Content (%)**	**No.**	**Depth (m)**	**Depth (m)**	**(kPa)**	**GR/SA/SI/CL (%) Atterberg Limits (%) w/c %**
0.0	1.4	Clay, some silt, trace sand, trace gravel (Fill), soft to firm, moist, brown						0.2	26	
								0.3	29	
			0.5	1419	23.5	1	0.5	0.5	30	22.7%
								0.6	38	
			0.75	1462	21.1			0.7	37	
						2	0.7 – 0.9	0.8	28	PL=24.5 LL=56.0 PI=31.5
								0.9	26	30.3%
								1.0	24	
								1.1	33	
								1.2	37	
								1.3	40	
								1.4	39	
1.4		End of Test Pit								

Comments:

For additional details and test pit photos, see page 2 of 2.

Water Conditions in Test Pit:

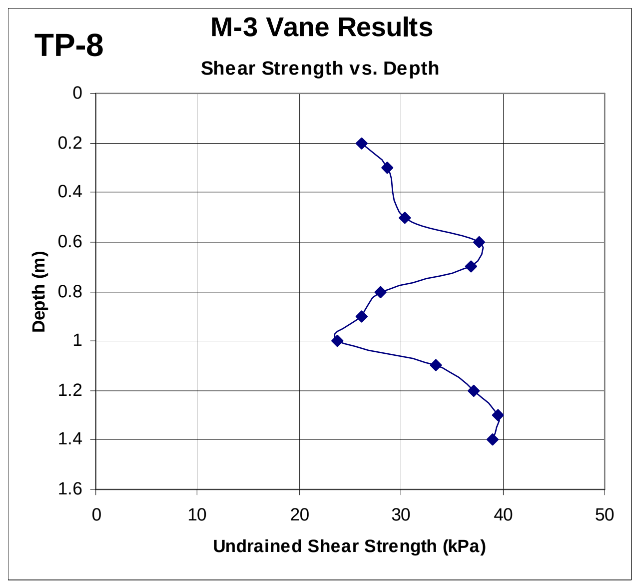
Moist soil at bottom of test pit. No seepage.

☒ Test Pit Dry

JOB No:	08-1111-0031
TEST PIT No.:	TP-8
ENGINEER:	MWK

Test Pit – TP-8

- No Seepage (dry pit)
- Final depth 1.4 m



Description/Notes:

0.0 m to 1.4 m Clay, some silt, trace sand, trace gravel (Fill), soft to firm, moist, brown

Sample #2 - Shelby Tube sample taken from 0.7 m to 0.9 m

Job number: 08-1111-0031
Date: October 2008
Engineer: MWK

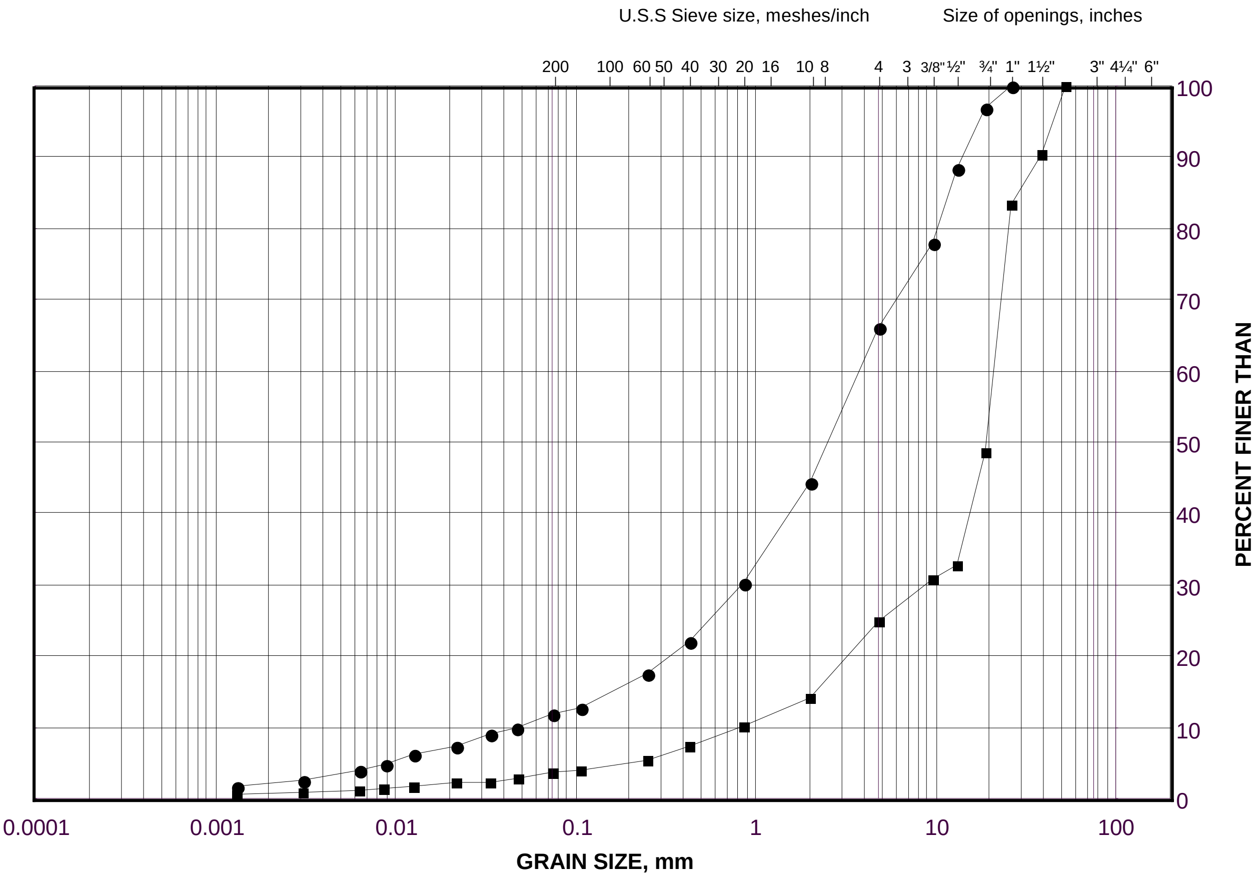
APPENDIX C

Laboratory Test Results

GRAIN SIZE DISTRIBUTION

Sand and Gravel to Sandy Gravel Fill

FIGURE C1



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	**COBBLE**
FINE GRAINED	**SAND SIZE**			**GRAVEL SIZE**		**SIZE**

LEGEND

SYMBOL	TEST PIT	SAMPLE	DEPTH(m)
●	5	1	0.3
■	6	2	0.7

Project Number: 08-1111-0031

Checked By: MWK / JPD

Golder Associates

Date: 25-May-09

CONSOLIDATED DRAINED DIRECT SHEAR TEST Sand and Gravel Fill		FIGURE C2 (Sheet 1 of 3)	
TEST STAGE	A	B	C
TEST PIT NUMBER	5	5	5
SAMPLE NUMBER	1	1	1
SAMPLE DEPTH, (m)	0.3	0.3	0.3
SAMPLE HEIGHT, (mm)	28.89	29.28	29.14
SAMPLE LENGTH, (mm)	60	60	60
WATER CONTENT, BEFORE TEST, (%)	4.6	4.6	4.6
NORMAL (CONSOLIDATION) STRESS, (kPa)	10	20	30
WATER CONTENT, AFTER TEST, (%)	17.4	16.5	16.7
DISPLACEMENT RATE, mm/min	0.007	0.007	0.007
TIME TO FAILURE, min	363	467	1134
PEAK SHEAR STRESS, (kPa)	14.65	21.10	46.61
HORIZONTAL DISPLACEMENT AT PEAK, (mm)	2.54	3.27	7.94
RESIDUAL SHEAR STRESS, (kPa)	16.09	21.80	42.11
HORIZONTAL DISPLACEMENT AT RESIDUAL, (mm)	13.73	15.66	15.80
DRY DENSITY, initial, Mg/m^3	1.80	1.80	1.80
WET DENSITY, initial, Mg/m^3	1.878	1.878	1.878

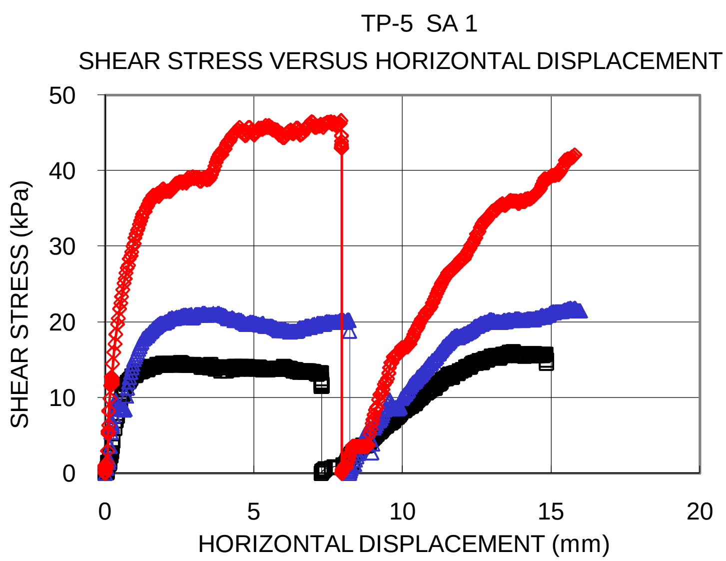
TEST NOTES:

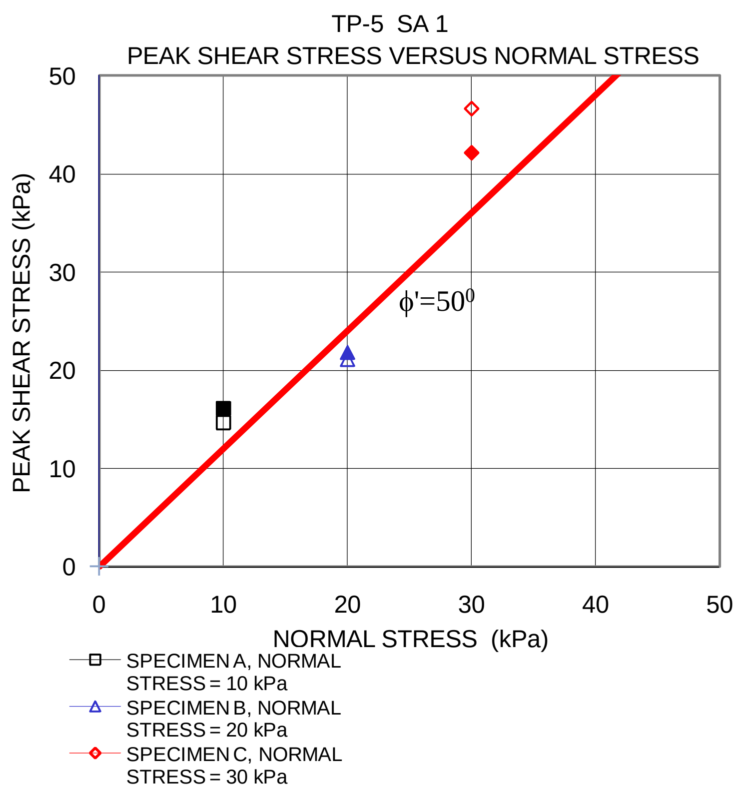
Direct shear test performed only on the portion of the sample passing the #4 sieve

Date: 11/20/2008 Prepared By: LFG

Project No. 08-1111-0031 **Golder Associates** Checked By: MM

CONSOLIDATED DRAINED DIRECT SHEAR TEST Sand and Gravel Fill	FIGURE C2 (Sheet 2 of 3)



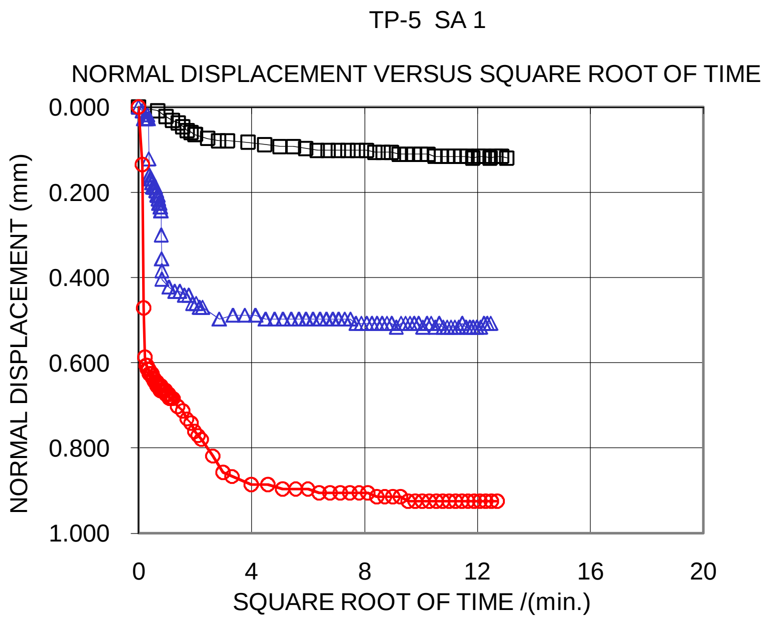


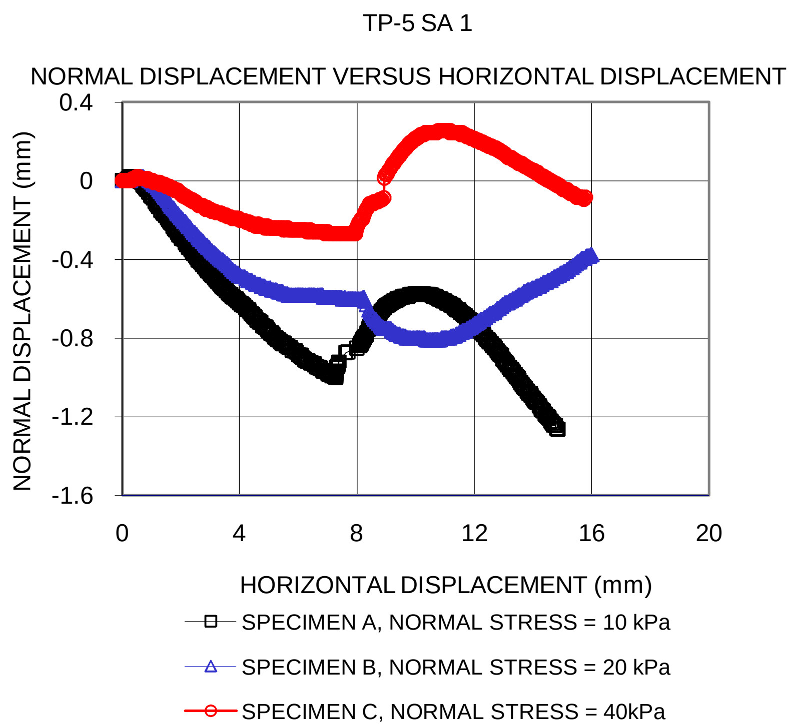
Date:	11/20/2008		Prepared By:	LFG
Project No.	08-1111-0031	**Golder Associates**	Checked By:	MM

CONSOLIDATED DRAINED DIRECT SHEAR TEST

Sand and Gravel Fill

FIGURE C2
(Sheet 3 of 3)





Date: 11/20/2008
Project No. 08-1111-0031

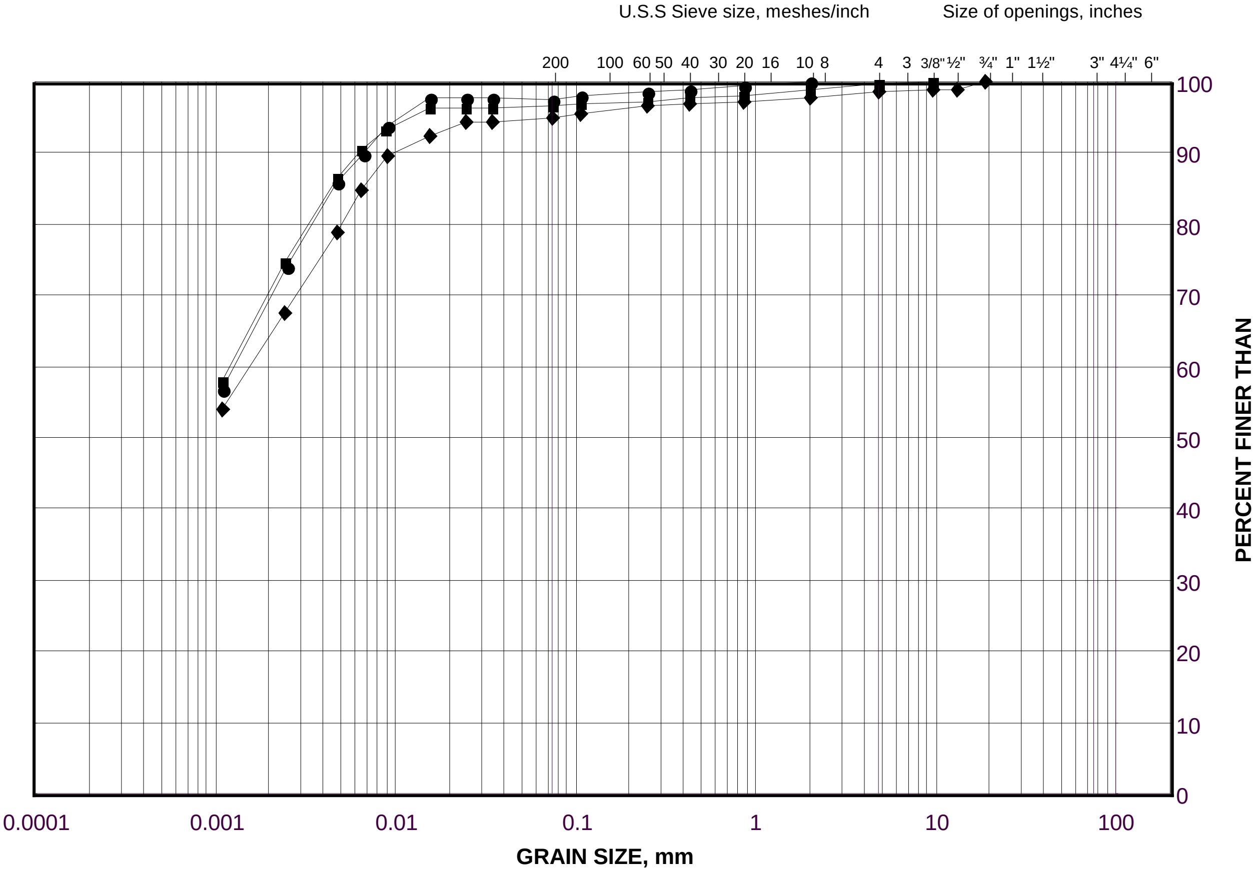
Golder Associates

Prepared By: LFG
Checked By: MM

GRAIN SIZE DISTRIBUTION

Clay Fill

FIGURE C3



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	**SAND SIZE**			**GRAVEL SIZE**		**SIZE**

LEGEND

SYMBOL	TEST PIT	SAMPLE	DEPTH(m)
●	4	1	0.0 - 5.0
■	2	2	1.4
◆	6	3	1.1

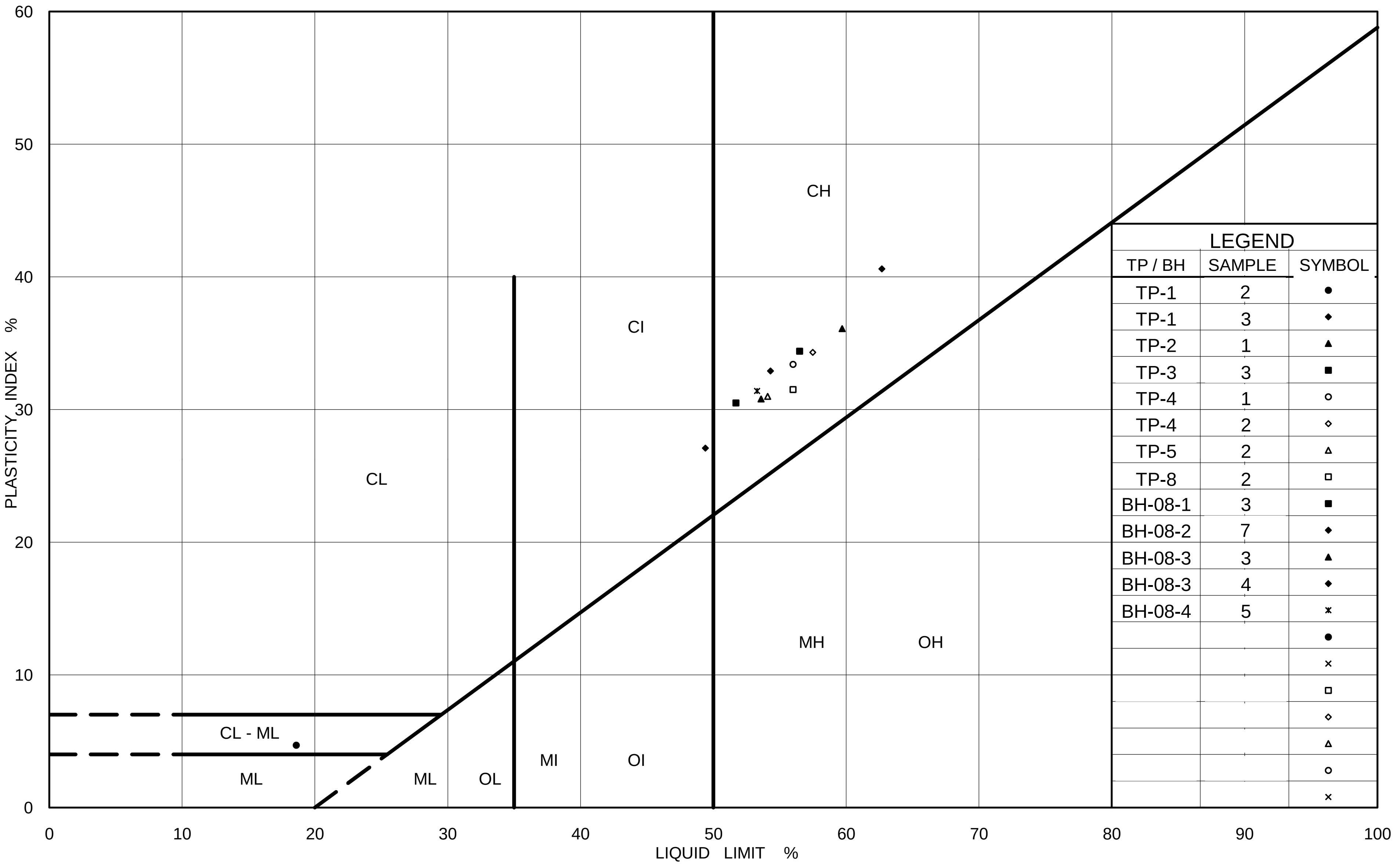
Project Number: 08-1111-0031

Checked By: MWK / JPD

Golder Associates

Date: 25-May-09

Oct 75, FF-S-21



LEGEND

TP / BH	SAMPLE	SYMBOL
TP-1	2	●
TP-1	3	◆
TP-2	1	▲
TP-3	3	■
TP-4	1	○
TP-4	2	◇
TP-5	2	△
TP-8	2	□
BH-08-1	3	■
BH-08-2	7	◆
BH-08-3	3	▲
BH-08-3	4	◆
BH-08-4	5	✕
		●
		×
		□
		◇
		△
		○
		×

Ministry of Transportation

Ontario

PLASTICITY CHART
Clay Fill

Figure No. C4

Project No. 08-1111-0031

Checked By: MWK / JPD

CONSOLIDATED DRAINED DIRECT SHEAR TEST

Undisturbed Clay Fill

FIGURE C5
(Sheet 1 of 3)

TEST STAGE	A	B	C
TEST PIT NUMBER	8	8	8
SAMPLE NUMBER	2	2	2
SAMPLE DEPTH, (m)	0.7-0.9	0.7-0.9	0.7-0.9
SAMPLE HEIGHT, (mm)	25	25	25
SAMPLE LENGTH, (mm)	60	60	60
WATER CONTENT, BEFORE TEST, (%)	27.4	26.2	26.0
NORMAL (CONSOLIDATION) STRESS, (kPa)	10	20	40
WATER CONTENT, AFTER TEST, (%)	33.1	31.0	30.2
DISPLACEMENT RATE, mm/min	0.0048	0.0048	0.0048
TIME TO FAILURE, min	82	325	350
PEAK SHEAR STRESS, (kPa)	12.24	18.67	28.95
HORIZONTAL DISPLACEMENT AT PEAK, (mm)	0.39	1.56	1.68
RESIDUAL SHEAR STRESS, (kPa)	8.64	14.36	21.80
HORIZONTAL DISPLACEMENT AT RESIDUAL, (mm)	14.03	11.49	8.98
DRY DENSITY, initial, Mg/m^3	1.58	1.60	1.60
WET DENSITY, initial, Mg/m^3	2.01	2.02	2.02

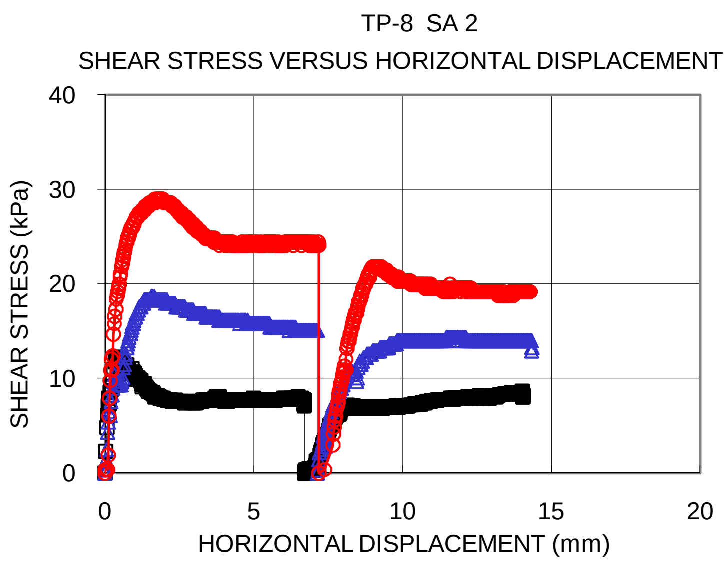
TEST NOTES:

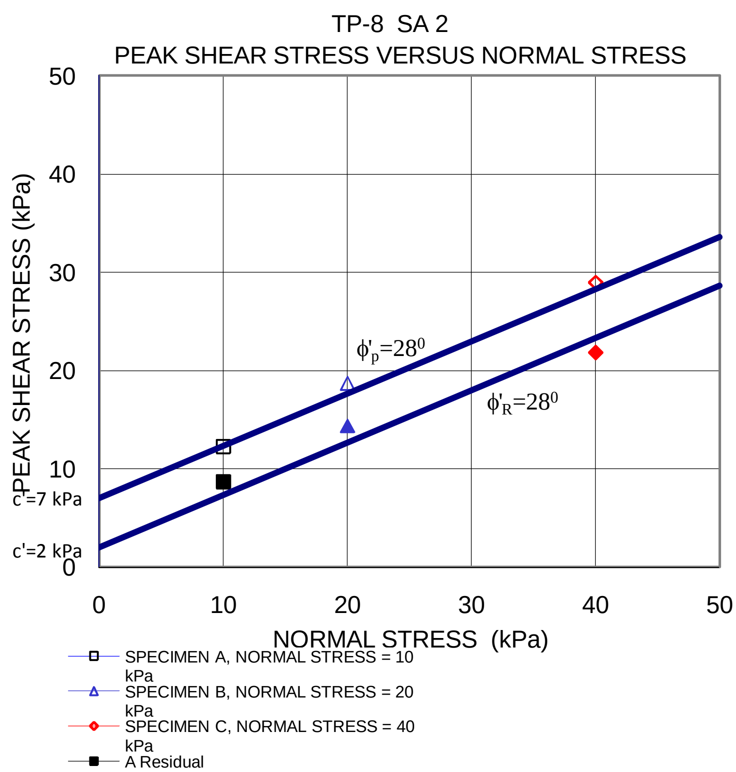
Date: 11/20/2008
Project No. 08-1111-0031

Golder Associates

Prepared By: LFG
Checked By: MM

CONSOLIDATED DRAINED DIRECT SHEAR TEST **Undisturbed Clay Fill**	**FIGURE C5** **(Sheet 2 of 3)**





Date: 11/20/2008

Project No. 08-1111-0031

Golder Associates

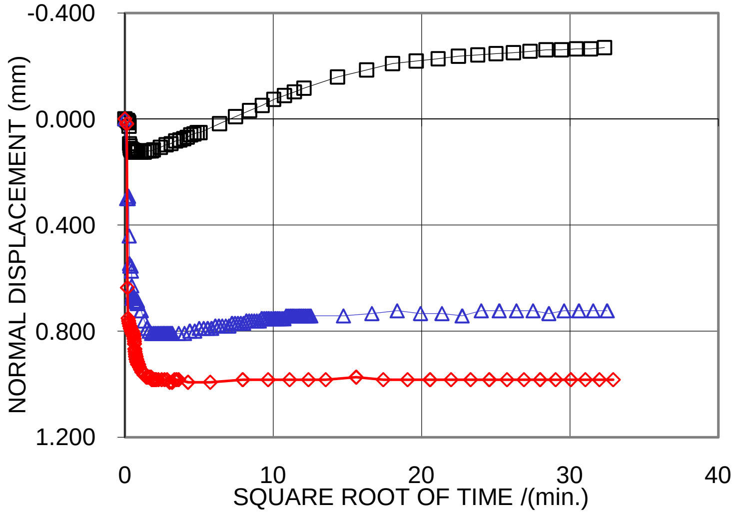
Prepared By: LFG

Checked By: MM

CONSOLIDATED DRAINED DIRECT SHEAR TEST Undisturbed Clay Fill	FIGURE C5 (Sheet 3 of 3)

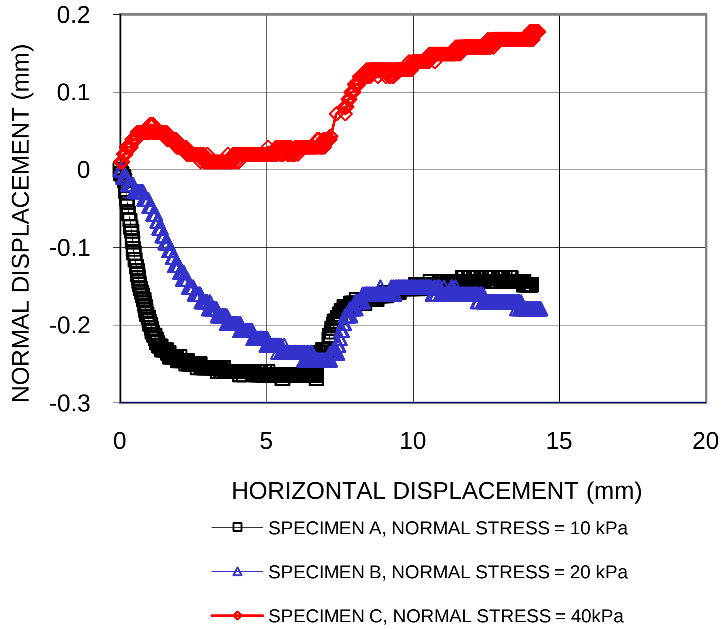
TP-8 SA 2

NORMAL DISPLACEMENT VERSUS SQUARE ROOT OF TIME



TP-8 SA 2

NORMAL DISPLACEMENT VERSUS HORIZONTAL DISPLACEMENT



Date:	11/20/2008		Prepared By:	LFG
Project No.	08-1111-0031	**Golder Associates**	Checked By:	MM

CONSOLIDATED DRAINED DIRECT SHEAR TEST

Remoulded Clay Fill

FIGURE C6
(Sheet 1 of 3)

TEST STAGE	A	B	C
TEST PIT NUMBER	8	8	8
SAMPLE NUMBER	1	1	1
SAMPLE DEPTH, (m)	0.5	0.5	0.5
SAMPLE HEIGHT, (mm)	30.00	29.42	29.98
SAMPLE LENGTH, (mm)	60.00	60.00	60.00
WATER CONTENT, BEFORE TEST, (%)	22.5	22.5	22.5
NORMAL (CONSOLIDATION) STRESS, (kPa)	10	20	40
WATER CONTENT, AFTER TEST, (%)	31.5	28.7	29.51
DISPLACEMENT RATE, mm/min	0.0048	0.0048	0.0048
TIME TO FAILURE, min	212	692	967
PEAK SHEAR STRESS, (kPa)	10.69	42.43	31.59
HORIZONTAL DISPLACEMENT AT PEAK, (mm)	1.02	3.32	4.64
RESIDUAL SHEAR STRESS, (kPa)	7.56	34.34	27.28
HORIZONTAL DISPLACEMENT AT RESIDUAL, (mm)	8.30	9.48	9.28
DRY DENSITY, initial, Mg/m^3	1.52	1.52	1.52
WET DENSITY, initial, Mg/m^3	1.863	1.863	1.863

TEST NOTES:

Specimens prepared at 24.2% moisture content and 1500 kg/m^3 dry density.

Date: 12/01/2008
Project No. 08-1111-0031

Golder Associates

Prepared By: LFG
Checked By: MM

CONSOLIDATED DRAINED DIRECT SHEAR TEST

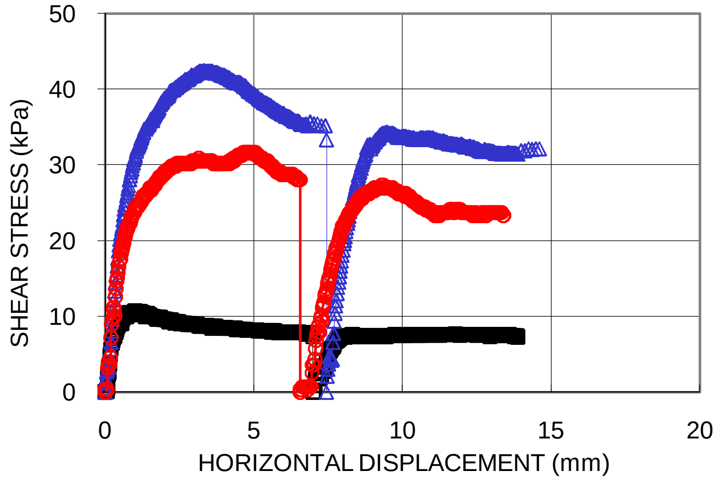
Remoulded Clay Fill

FIGURE C6

(Sheet 2 of 3)

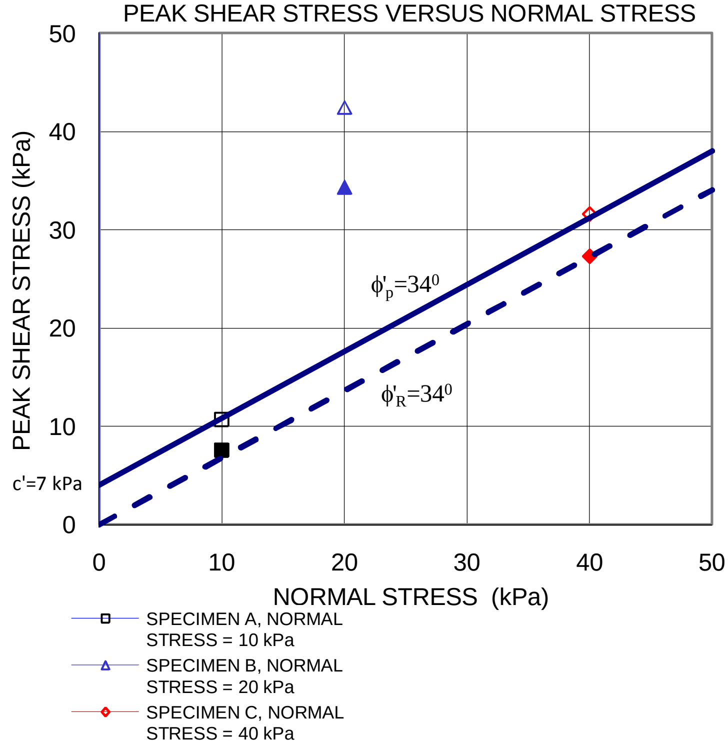
TP-8 SA 1

SHEAR STRESS VERSUS HORIZONTAL DISPLACEMENT



TP-8 SA 1

PEAK SHEAR STRESS VERSUS NORMAL STRESS

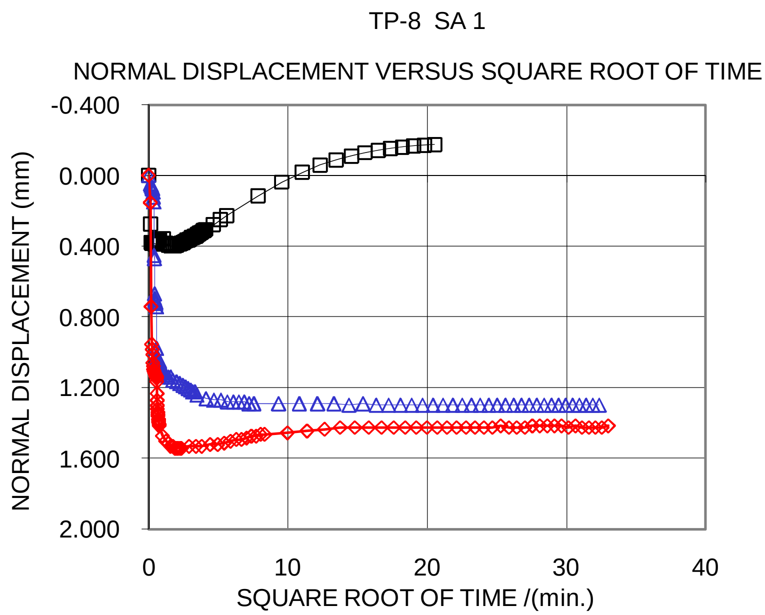


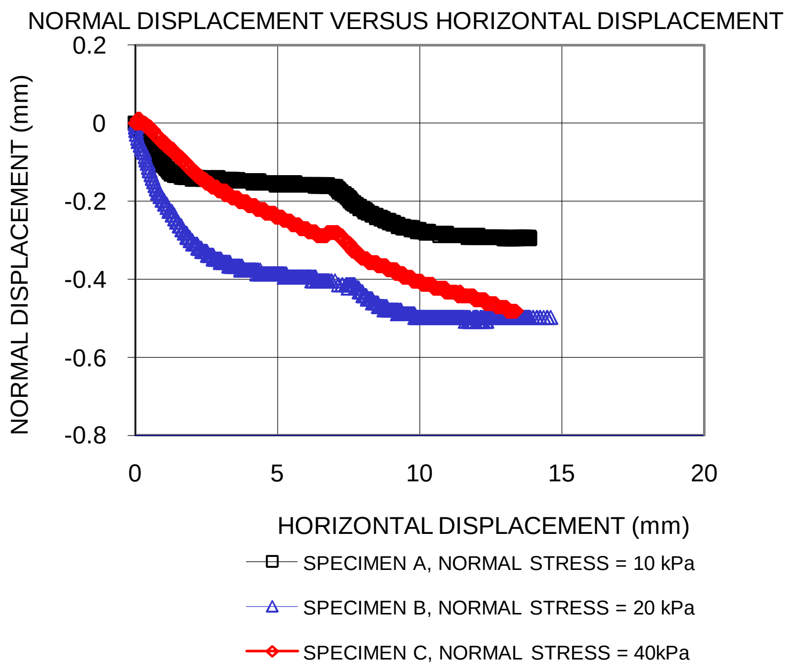
Date:	12/01/2008		Prepared By:	LFG
Project No.	08-1111-0031	**Golder Associates**	Checked By:	MM

CONSOLIDATED DRAINED DIRECT SHEAR TEST

Remoulded Clay Fill

FIGURE C6
(Sheet 3 of 3)





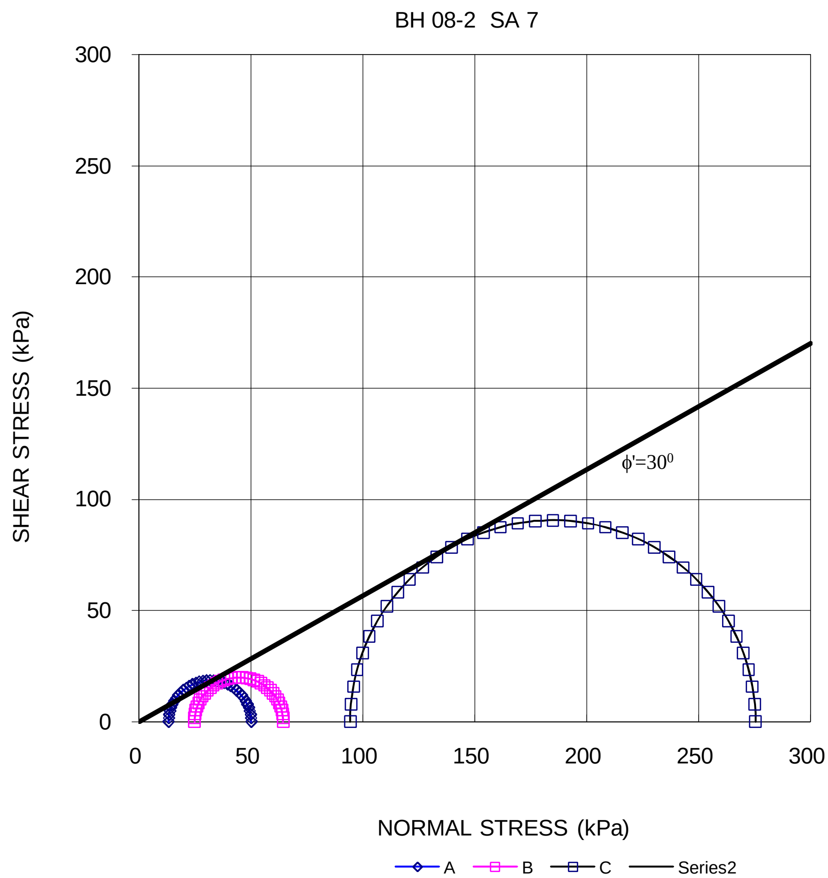
Date:	12/01/2008		Prepared By:	LFG
Project No.	08-1111-0031	**Golder Associates**	Checked By:	MM

CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS Undisturbed Clay Fill				FIGURE C7 (Sheet 1 of 4)
TEST STAGE	A	B	C	
BOREHOLE NUMBER	08-2	08-2	08-2	
SAMPLE	7	7	7	
SPECIMEN DIAMETER, cm	5.01	4.99	4.99	
SPECIMEN HEIGHT, cm	10.14	10.16	10.15	
WATER CONTENT BEFORE CONSOLIDATION, %	31.4	30.8	28.2	
CELL PRESSURE, σ_3, kPa	645.0	395.0	610.0	
BACK PRESSURE, kPa	625.0	345.0	485.0	
PORE PRESSURE PARAMETER "B"	0.96	0.99	0.99	
CONSOLIDATION PRESSURE, σc, kPa	20.0	50.0	125.0	
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	0.6	6.3	5.8	
WATER CONTENT AFTER CONSOLIDATION, %	31.1	27.0	24.7	
AVERAGE RATE OF STRAIN, %/hr	0.5	0.5	0.5	
TIME TO FAILURE, DAYS	1	1	1	
WATER CONTENT AFTER TEST, %	28.5	29.5	26.6	
MAX. DEVIATOR STRESS, $(\sigma_1\text{-}\sigma_3)$, kPa	51.1	53.0	192.4	
AXIAL STRAIN AT $(\sigma_1\text{-}\sigma_3)$ MAXIMUM, %	17.8	19.4	5.1	
MAX EFFECTIVE PRINCIPAL STRESS RATIO, (σ_1/σ_3) MAXIMUM	3.8	2.6	2.9	
DEVIATOR STRESS AT (σ_1/σ_3) MAXIMUM, kPa	36.8	39.7	181.0	
AXIAL STRAIN AT (σ_1/σ_3) MAXIMUM, %	1.6	3.3	4.0	
PORE PRESSURE PARAMETER, Af, AT $(\sigma_1\text{-}\sigma_3)$ MAXIMUM	-0.20	0.23	0.11	
PORE PRESSURE PARAMETER, Af, AT (σ_1/σ_3) MAXIMUM	0.18	0.63	0.17	
NATURAL WATER CONTENT, %	23.8	24.1	22.6	
DRY DENSITY, Mg/m^3	1.66	1.66	1.69	
FILTER DRAINS USED, y/n	y	y	y	
TEST NOTES:				
CHANGED RATE OF STRAIN, %/hr	-	-	-	
AXIAL STRAIN WHERE RATE OF STRAIN WAS CHANGED, %	-	-	-	
FAILURE PLANE NUMBER	1.0	1.0	1.0	
ANGLE OF FAILURE, DEGREES	65.0	55.0	65.0	

Date: 02/27/2009 Prepared By: MM

Project No. 08-1111-0031 **Golder Associates** Checked By: MK

CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS **Undisturbed Clay Fill**	**FIGURE C7** **(Sheet 2 of 4)**



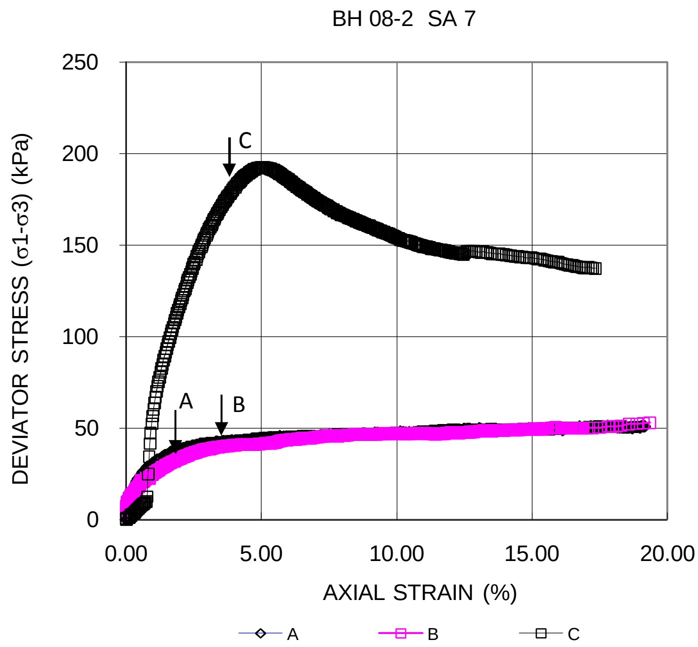
Date: 02/27/2009
Project No. 08-1111-0031

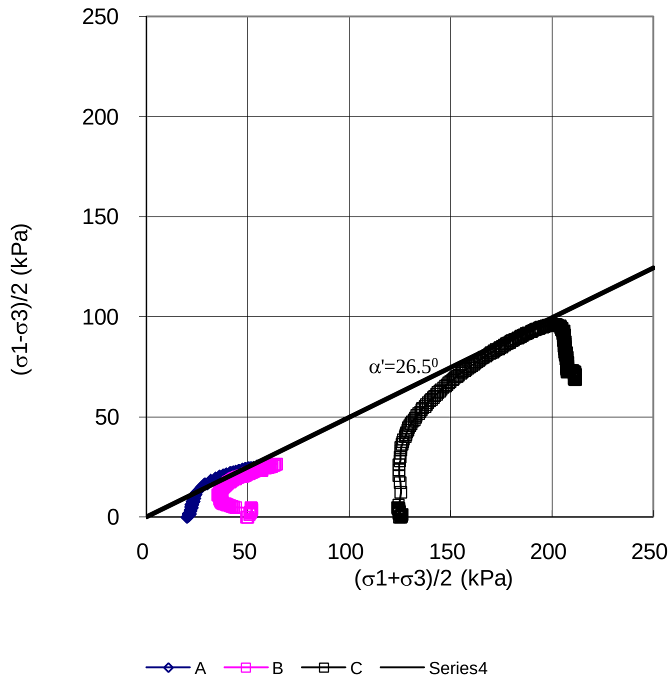
Golder Associates

Prepared By: MM
Checked By: MK

CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
Undisturbed Clay Fill

FIGURE C7
(Sheet 3 of 4)



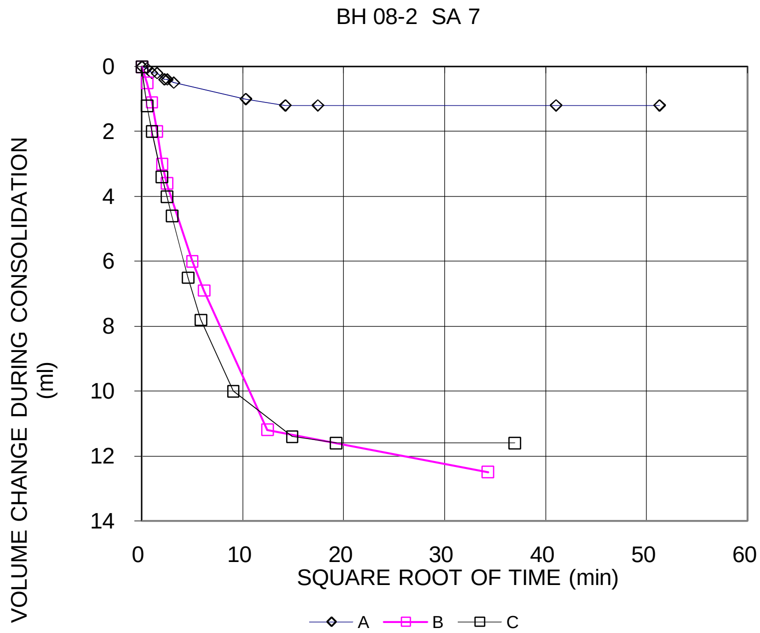


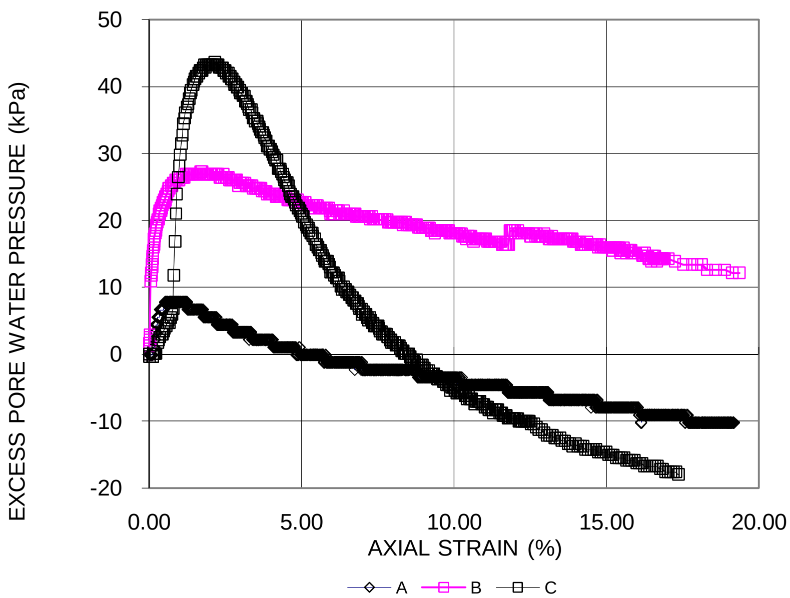
Date: 02/27/2009
Project No. 08-1111-0031

Golder Associates

Prepared By: MM
Checked By: MK

CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS Undisturbed Clay Fill	FIGURE C7 (Sheet 4 of 4)





Date:	02/27/2009		Prepared By:	MM
Project No.	08-1111-0031	**Golder Associates**	Checked By:	MK

CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
Undisturbed Clay Fill

FIGURE C8
(Sheet 1 of 4)

TEST STAGE	A	B
TEST PIT NUMBER	3	3
SAMPLE	3	3
SPECIMEN DIAMETER, cm	5.09	5.00
SPECIMEN HEIGHT, cm	10.17	10.17
WATER CONTENT BEFORE CONSOLIDATION, %	29.8	32.8
CELL PRESSURE, σ_3, kPa	220.0	165.0
BACK PRESSURE, kPa	205.0	135.0
PORE PRESSURE PARAMETER "B"	0.99	0.96
CONSOLIDATION PRESSURE, σc, kPa	15.0	30.0
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	1.4	3.1
WATER CONTENT AFTER CONSOLIDATION, %	28.9	30.7
AVERAGE RATE OF STRAIN, %/hr	0.5	0.5
TIME TO FAILURE, DAYS	1	1
WATER CONTENT AFTER TEST, %	30.8	30.6
MAX. DEVIATOR STRESS, $(\sigma_1-\sigma_3)$, kPa	52.6	57.8
AXIAL STRAIN AT $(\sigma_1-\sigma_3)$ MAXIMUM, %	12.2	10.8
MAX EFFECTIVE PRINCIPAL STRESS RATIO, (σ_1/σ_3) MAXIMUM	5.4	3.7
DEVIATOR STRESS AT (σ_1/σ_3) MAXIMUM, kPa	26.3	50.1
AXIAL STRAIN AT (σ_1/σ_3) MAXIMUM, %	2.4	4.5
PORE PRESSURE PARAMETER, Af, AT $(\sigma_1-\sigma_3)$ MAXIMUM	-0.06	0.06
PORE PRESSURE PARAMETER, Af, AT (σ_1/σ_3) MAXIMUM	0.34	0.22
NATURAL WATER CONTENT, %	28.8	31.8
DRY DENSITY, Mg/m^3	1.53	1.46
FILTER DRAINS USED, y/n	y	y
TEST NOTES:		
CHANGED RATE OF STRAIN, %/hr	-	-
AXIAL STRAIN WHERE RATE OF STRAIN WAS CHANGED, %	-	-
FAILURE PLANE NUMBER	1.0	1.0
ANGLE OF FAILURE, DEGREES	55.0	65.0

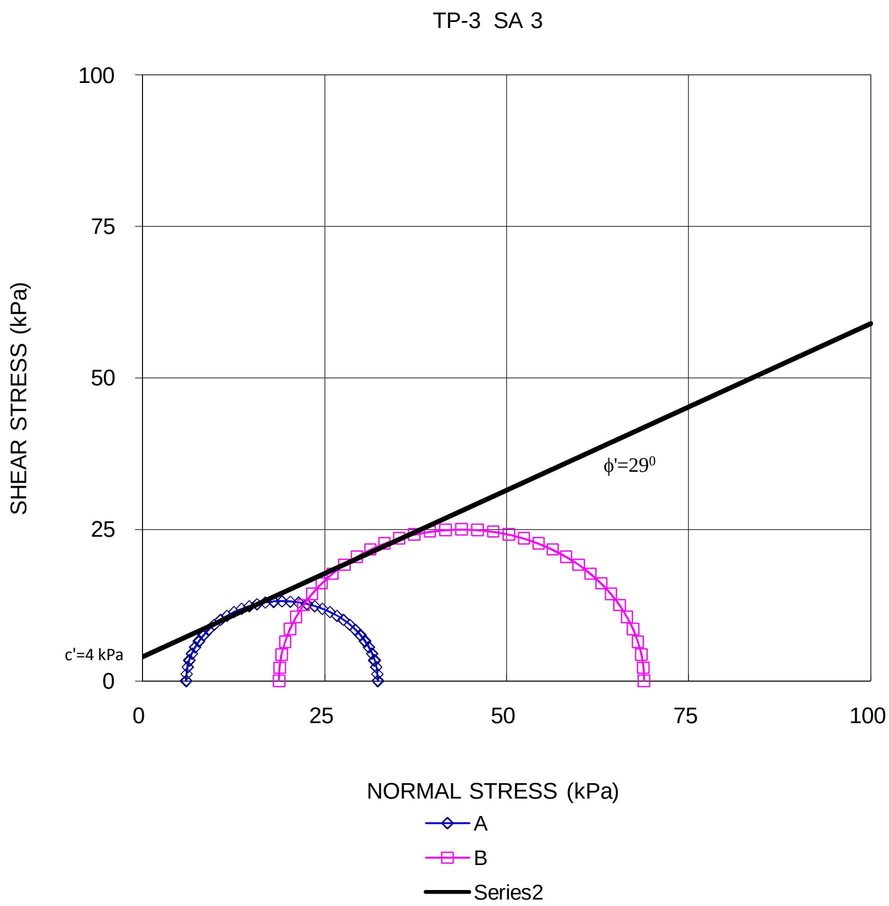
Date: 12/29/2008
Project No. 08-1111-0031

Golder Associates

Prepared By: MM
Checked By: RO

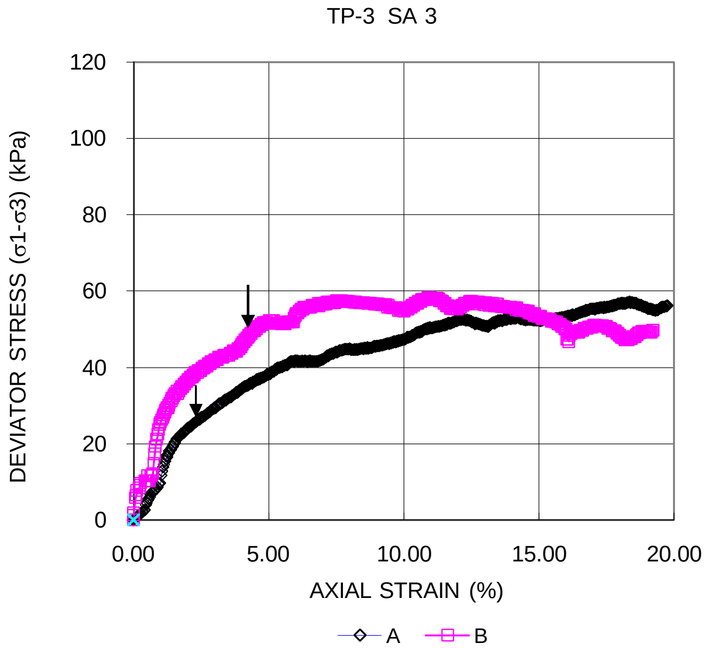
CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
Undisturbed Clay Fill

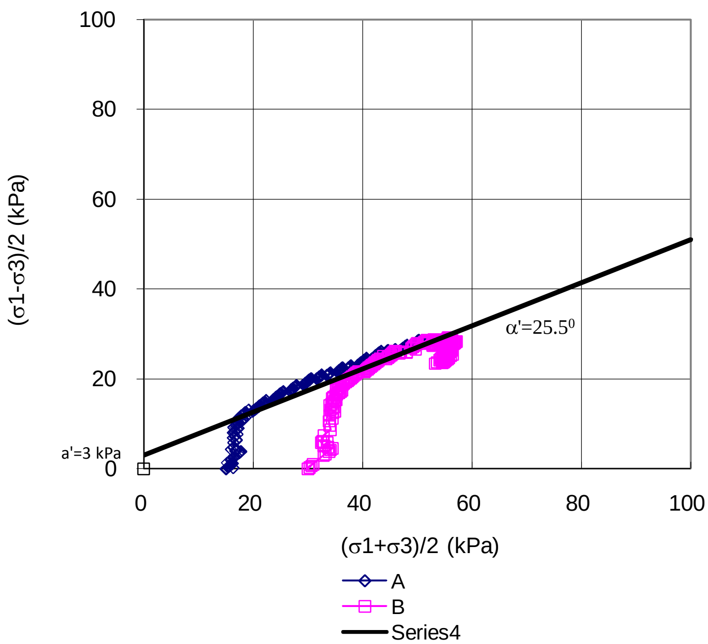
FIGURE C8
(Sheet 2 of 4)



Date:	12/29/2008		Prepared By:	MM
Project No.	08-1111-0031	**Golder Associates**	Checked By:	RO

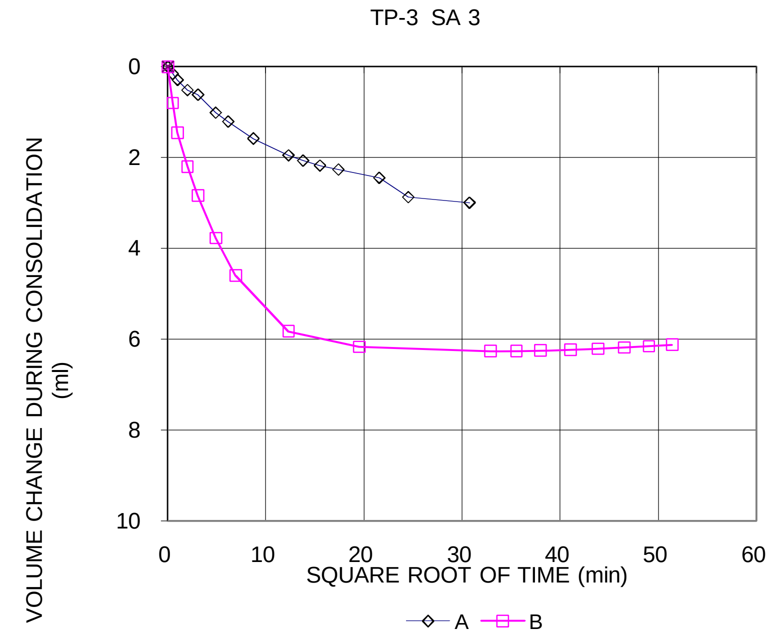
CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS Undisturbed Clay Fill	**FIGURE C8 (Sheet 3 of 4)**

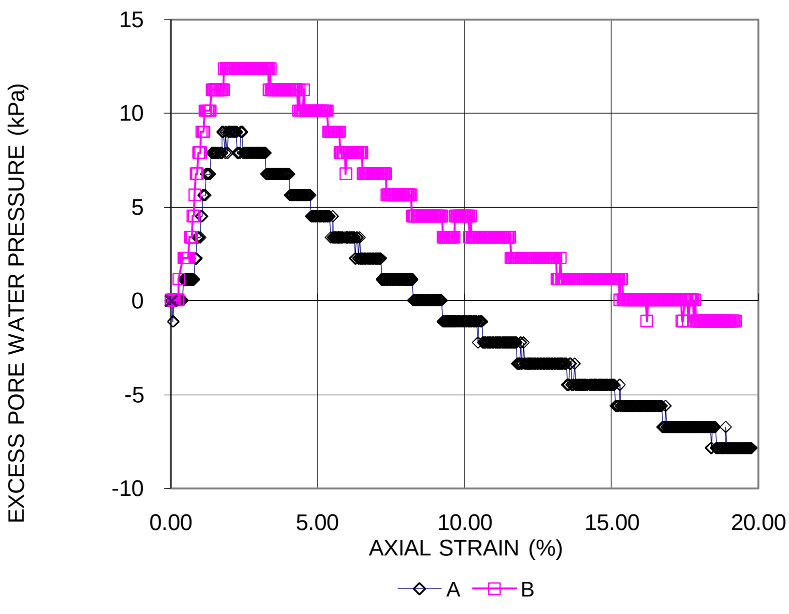




Date:	12/29/2008		Prepared By:	MM
Project No.	08-1111-0031	**Golder Associates**	Checked By:	RO

CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS Undisturbed Clay Fill	FIGURE C8 (Sheet 4 of 4)





Date: 12/29/2008
Project No. 08-1111-0031

Golder Associates

Prepared By: MM
Checked By: RO

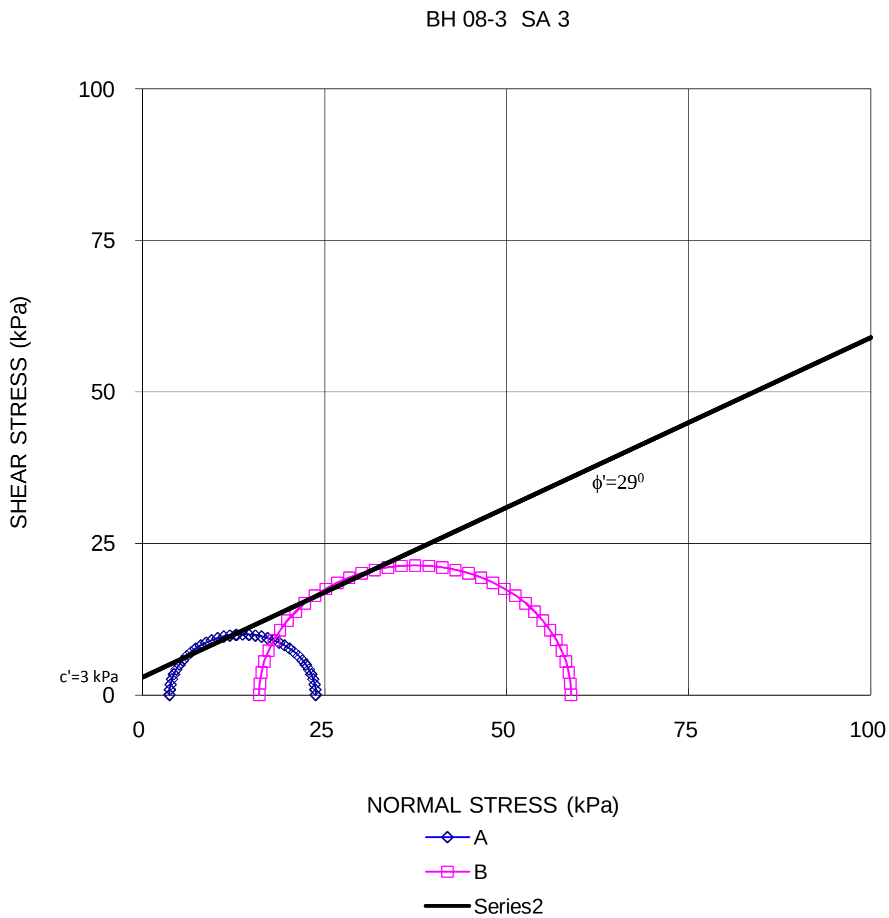
CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS Undisturbed Clay Fill			**FIGURE C9** **(Sheet 1 of 4)**
TEST STAGE	A	B	
BOREHOLE NUMBER	08-3	08-3	
SAMPLE	3	3	
SPECIMEN DIAMETER, cm	5.03	5.00	
SPECIMEN HEIGHT, cm	10.15	10.10	
WATER CONTENT BEFORE CONSOLIDATION, %	30.7	31.6	
CELL PRESSURE, σ_3, kPa	355.0	370.0	
BACK PRESSURE, kPa	345.0	345.0	
PORE PRESSURE PARAMETER "B"	0.99	0.96	
CONSOLIDATION PRESSURE, σc, kPa	10.0	25.0	
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	0.5	2.4	
WATER CONTENT AFTER CONSOLIDATION, %	30.4	30.1	
AVERAGE RATE OF STRAIN, %/hr	0.5	0.5	
TIME TO FAILURE, DAYS	1	1	
WATER CONTENT AFTER TEST, %	29.7	29.4	
MAX. DEVIATOR STRESS, (σ_1-σ_3), kPa	49.7	51.9	
AXIAL STRAIN AT (σ_1-σ_3) MAXIMUM, %	14.2	6.2	
MAX EFFECTIVE PRINCIPAL STRESS RATIO, (σ_1/σ_3) MAXIMUM	6.5	3.7	
DEVIATOR STRESS AT (σ_1/σ_3) MAXIMUM, kPa	18.2	42.8	
AXIAL STRAIN AT (σ_1/σ_3) MAXIMUM, %	1.3	3.0	
PORE PRESSURE PARAMETER, Af, AT (σ_1-σ_3) MAXIMUM	-0.29	0.05	
PORE PRESSURE PARAMETER, Af, AT (σ_1/σ_3) MAXIMUM	0.37	0.21	
NATURAL WATER CONTENT, %	25.7	26.1	
DRY DENSITY, Mg/m^3	1.58	1.59	
FILTER DRAINS USED, y/n	y	y	
TEST NOTES:			
CHANGED RATE OF STRAIN, %/hr	-	-	
AXIAL STRAIN WHERE RATE OF STRAIN WAS CHANGED, %	-	-	
FAILURE PLANE NUMBER	1.0	1.0	
ANGLE OF FAILURE, DEGREES	55.0	60.0	

Date: 01/25/2008 Prepared By: MM

Project No. 08-1111-0031 **Golder Associates** Checked By: RO

CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
Undisturbed Clay Fill

FIGURE C9
(Sheet 2 of 4)



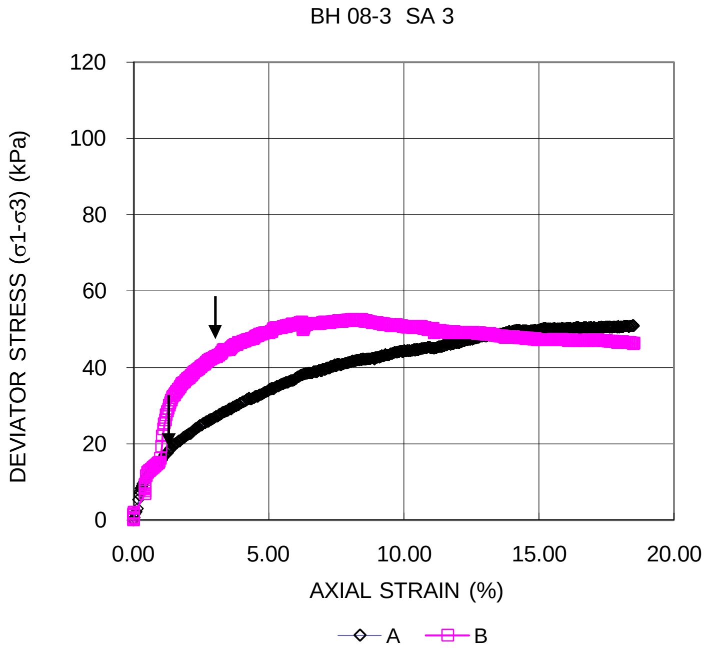
Date: 12/29/2008
Project No. 08-1111-0031

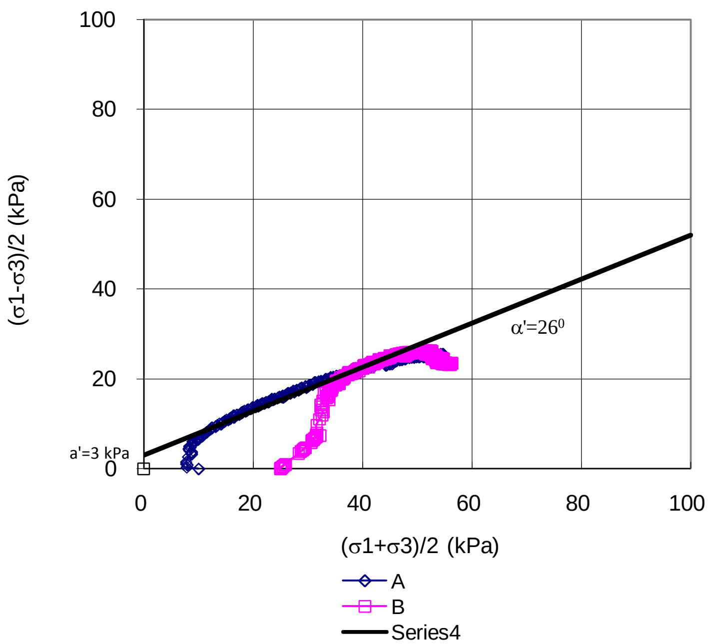
Golder Associates

Prepared By: MM
Checked By: RO

CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
Undisturbed Clay Fill

FIGURE C9
(Sheet 3 of 4)

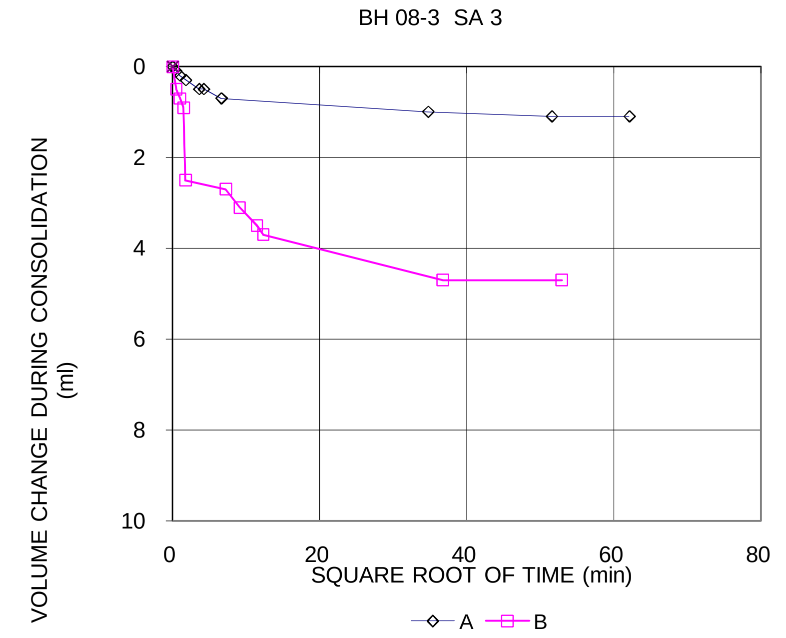


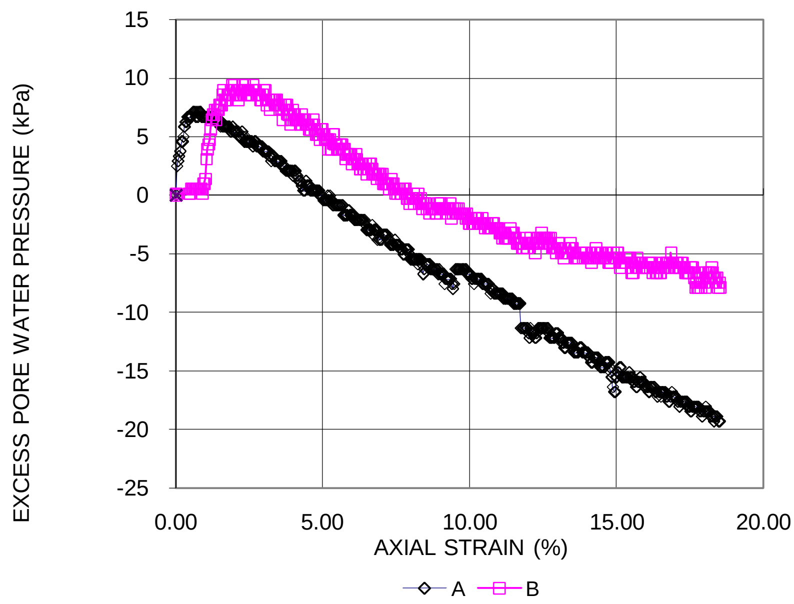


Date:	12/29/2008		Prepared By:	MM
Project No.	08-1111-0031	**Golder Associates**	Checked By:	RO

CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
Undisturbed Clay Fill

FIGURE C9
(Sheet 4 of 4)



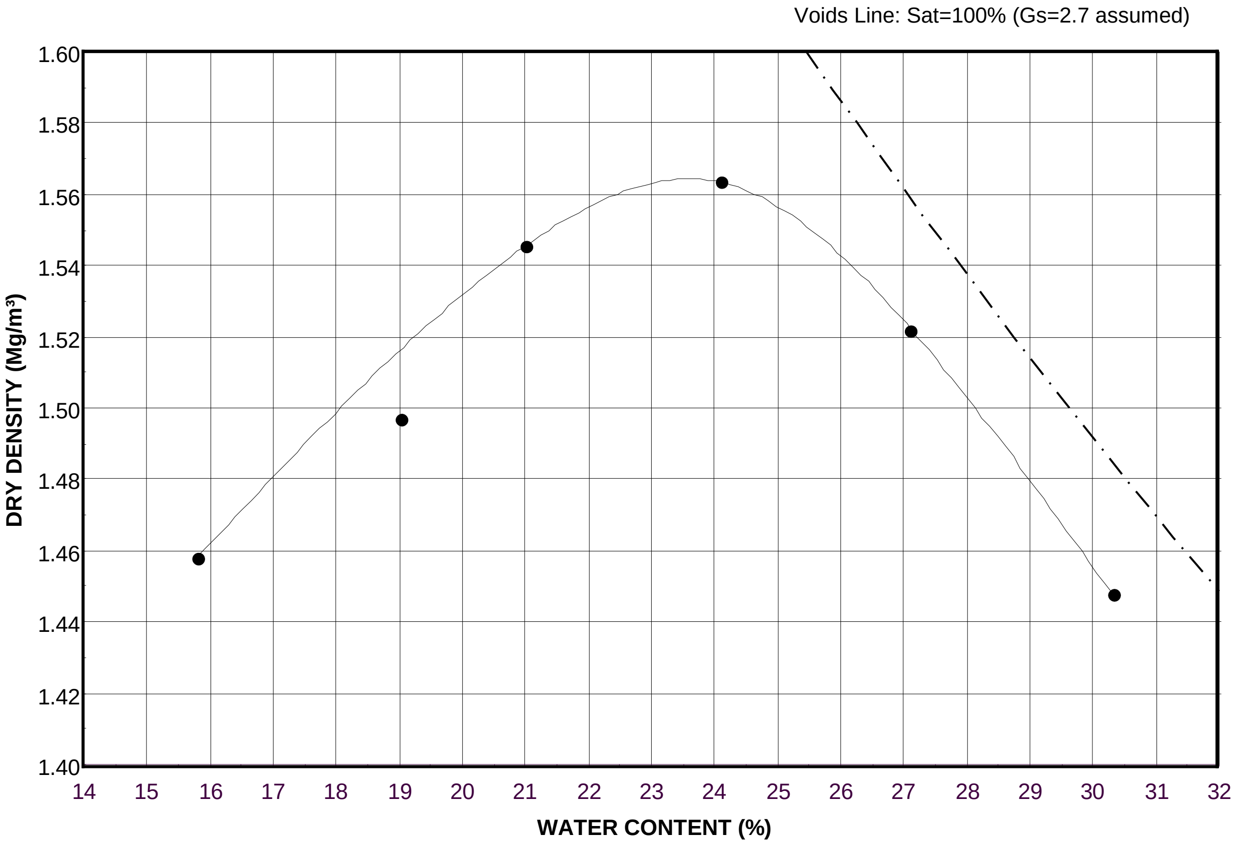


Date:	12/29/2008		Prepared By:	MM
Project No.	08-1111-0031	**Golder Associates**	Checked By:	RO

LABORATORY COMPACTION TEST

Clay Fill

FIGURE C10



Standard
Proctor Test Results

Test Pit:
TP-4

Sample:
1

Max Dry Density:
1.564 Mg/m³

Optimum Water
Content: 23.8%

Natural Water
Content: 23.1%

Project Number: 08-1111-0031
Checked By: MWK / JPD

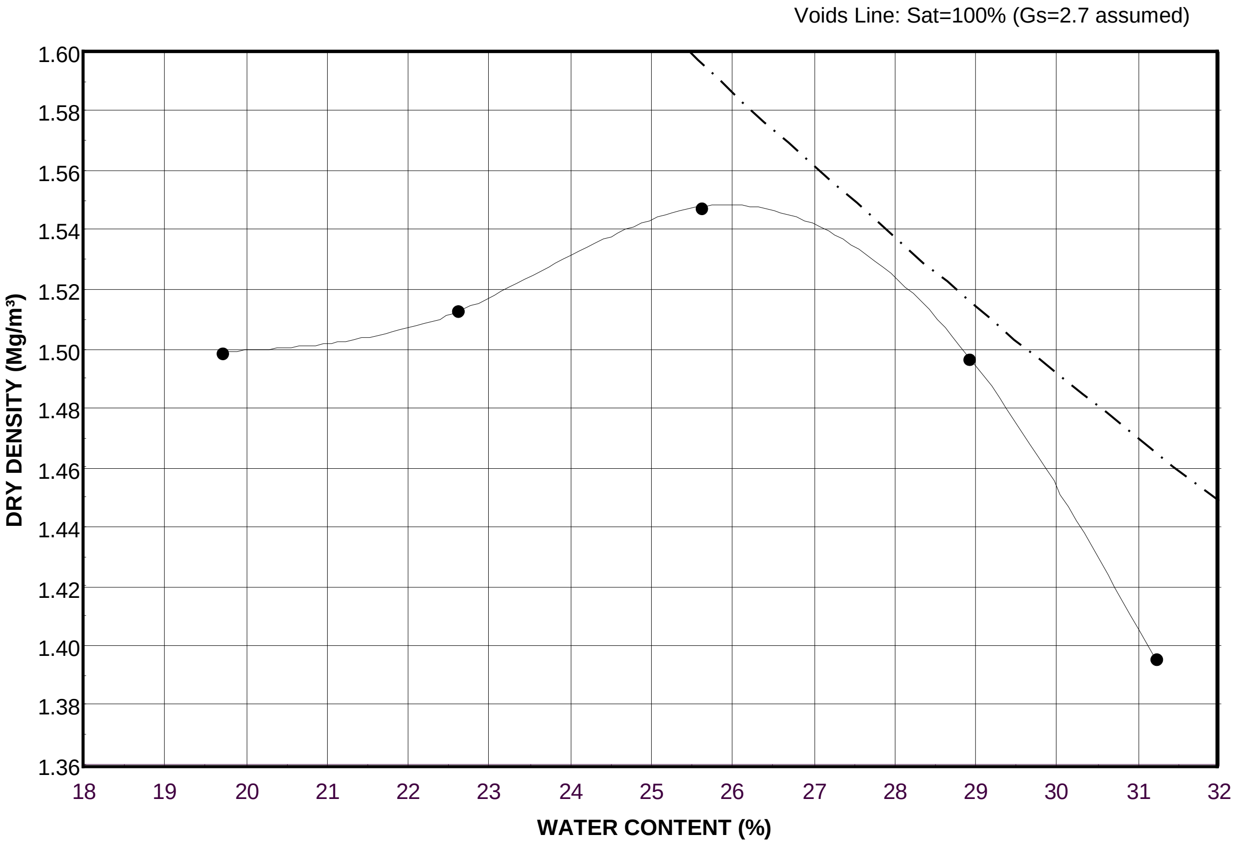
Golder Associates

LABID: 09-186
Date: 25-May-09

LABORATORY COMPACTION TEST

Clay Fill

FIGURE C11



Standard
Proctor Test Results

Test Pit:
TP-2

Sample:
1

Max Dry Density:
1.548 Mg/m³

Optimum Water
Content: 26.1%

Natural Water
Content: 23.8%

OEDOMETER CONSOLIDATION SUMMARY

Clay Fill

FIGURE C12

(Sheet 1 of 4)

SAMPLE IDENTIFICATION

Project Number	08-1111-0031	Sample Number	3
Borehole Number	08-1	Sample Depth, m	1.52-2.13

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	9		
Date Started	11/25/2008		
Date Completed	12/09/2008		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m^3	19.94
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m^3	15.90
Area, cm^2	31.47	Specific Gravity, measured	2.79
Volume, cm^3	59.89	Solids Height, cm	1.106
Water Content, %	25.45	Volume of Solids, cm^3	34.80
Wet Mass, g	121.80	Volume of Voids, cm^3	25.09
Dry Mass, g	97.09	Degree of Saturation, %	98.5

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	1.903	0.721	1.903				
4.72	1.972	0.784	1.938	swell			
9.59	1.927	0.743	1.950	7	1.15E-01	4.88E-03	5.50E-05
19.20	1.922	0.738	1.925	101	7.77E-03	2.73E-04	2.08E-07
38.88	1.908	0.725	1.915	86	9.04E-03	3.74E-04	3.31E-07
77.88	1.884	0.704	1.896	37	2.06E-02	3.23E-04	6.53E-07
155.81	1.854	0.677	1.869	26	2.85E-02	2.02E-04	5.65E-07
314.85	1.821	0.647	1.837	35	2.05E-02	1.09E-04	2.19E-07
622.26	1.776	0.606	1.798	20	3.43E-02	7.68E-05	2.58E-07
1244.69	1.720	0.555	1.748	34	1.91E-02	4.73E-05	8.83E-08
2489.96	1.651	0.493	1.685	21	2.87E-02	2.92E-05	8.20E-08
1244.69	1.671	0.511	1.661				
314.85	1.723	0.558	1.697				
77.88	1.780	0.610	1.752				
19.37	1.834	0.658	1.807				
4.72	1.872	0.693	1.853				

Note:
k calculated using cv based on t_{90} values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.87	Unit Weight, kN/m^3	20.54
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m^3	16.16
Area, cm^2	31.47	Specific Gravity, measured	2.79
Volume, cm^3	58.91	Solids Height, cm	1.106
Water Content, %	27.11	Volume of Solids, cm^3	34.80
Wet Mass, g	123.41	Volume of Voids, cm^3	24.11
Dry Mass, g	97.09		

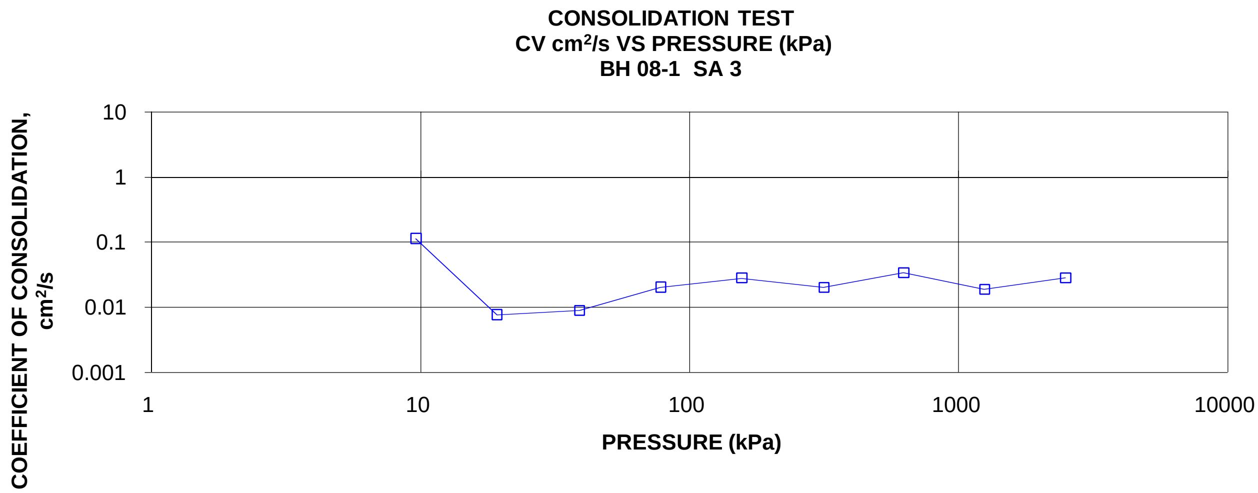
Prepared By: LFG

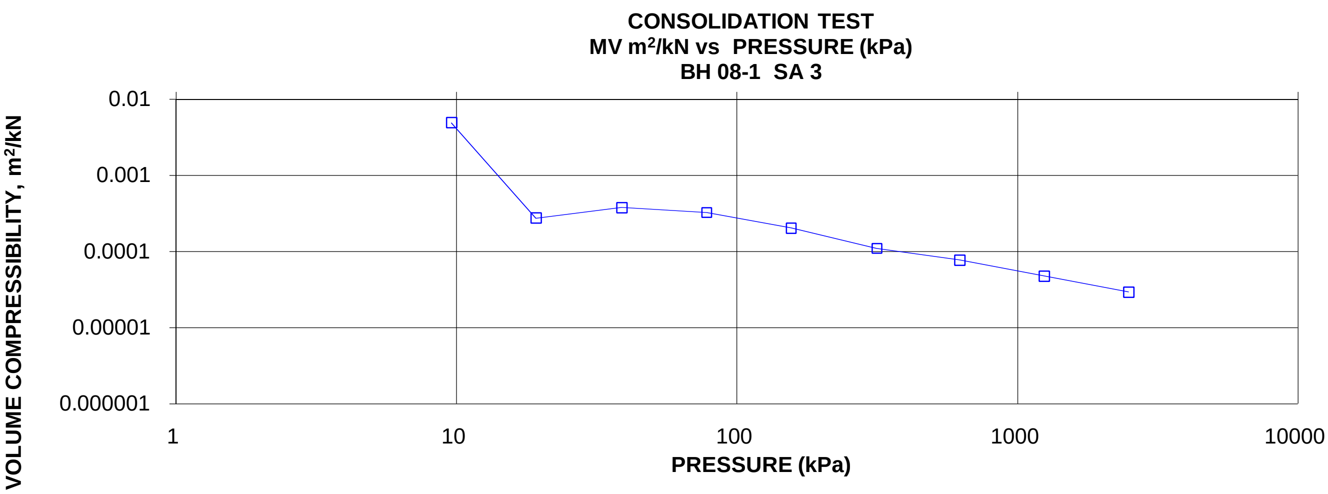
Golder Associates

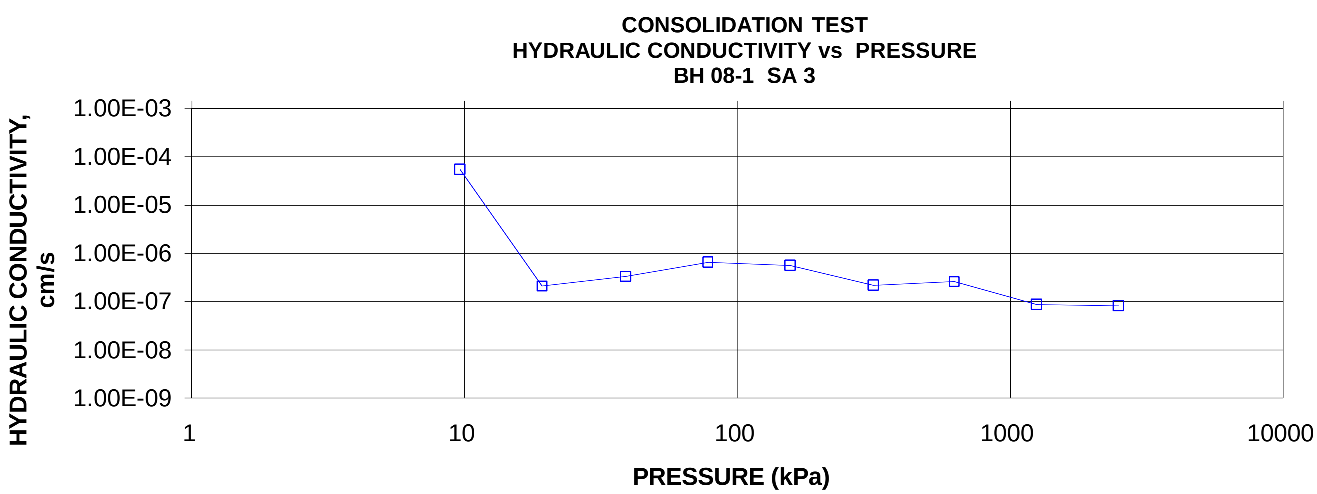
Checked By: MM

OEDOMETER CONSOLIDATION SUMMARY
ClayFill

FIGURE C12
(Sheet 2 of 4)



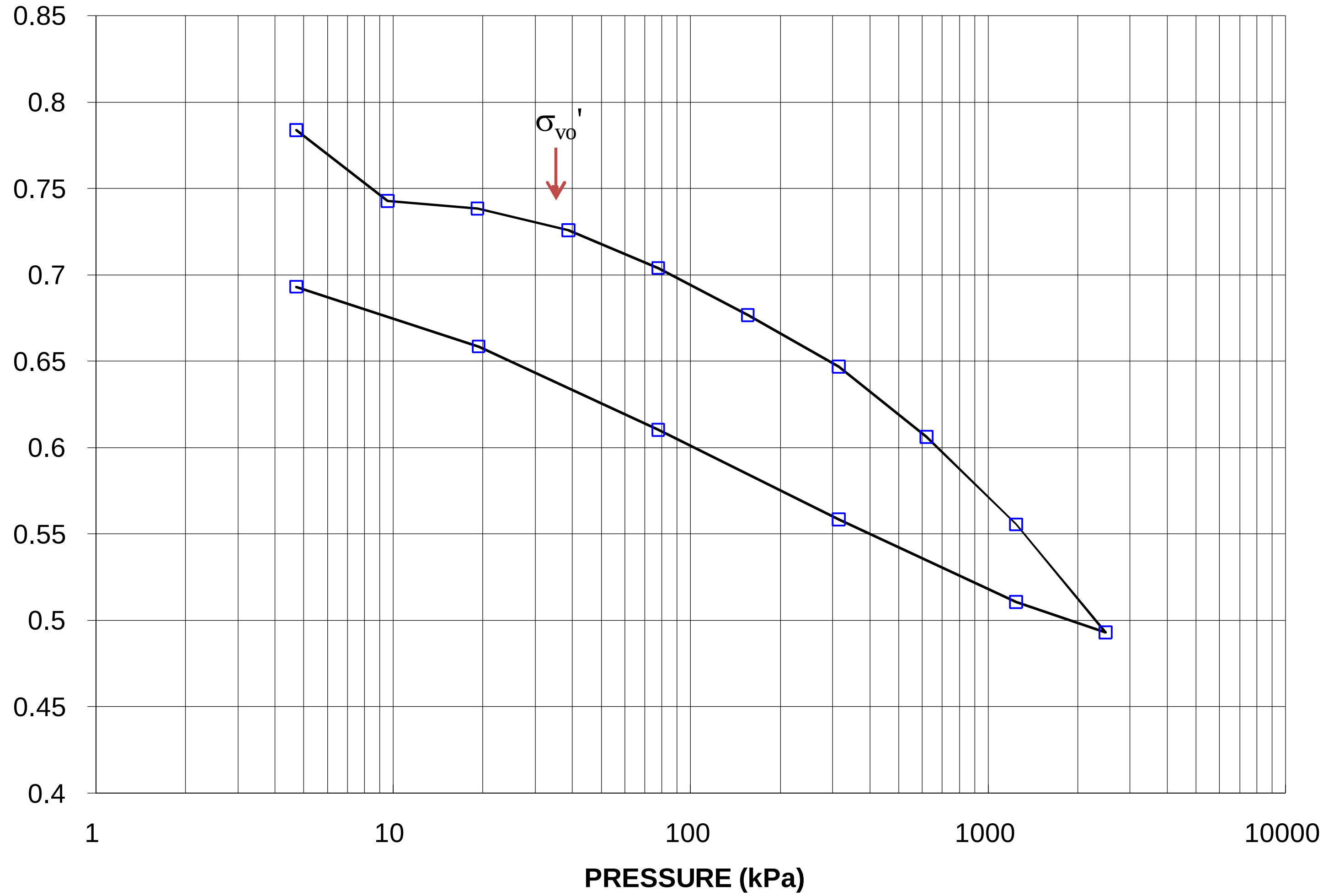




Project No. 08-1111-0031

Prepared By: LFG

CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH 08-01 SA 3



Golder Associates

Checked By: MM

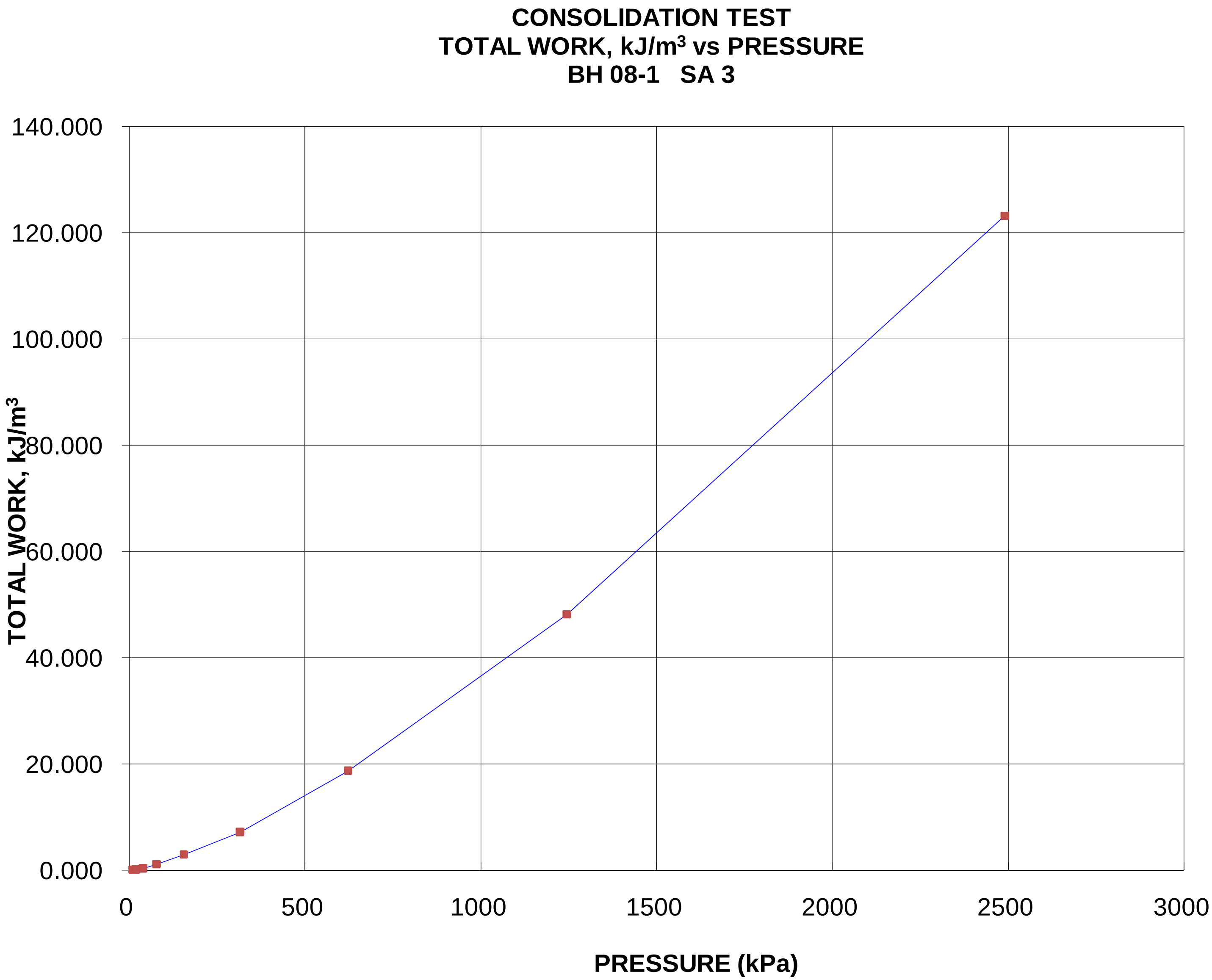
CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE C12
(Sheet 3 OF 4)

Project No. 08-1111-0031
Prepared By: LFG

Golder Associates

Checked By: MM



CONSOLIDATION TEST
TOTAL WORK VS. PRESSURE

FIGURE C12
(Sheet 4 of 4)

OEDOMETER CONSOLIDATION SUMMARY

Clay Fill

FIGURE C13
(Sheet 1 of 4)

SAMPLE IDENTIFICATION

Project Number	08-1111-0031	Sample Number	7
Borehole Number	08-2	Sample Depth, m	6.1-6.7

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	3		
Date Started	11/12/2008		
Date Completed	11/27/2008		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.54	Unit Weight, kN/m^3	19.35
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	15.15
Area, cm^2	31.62	Specific Gravity, measured	2.80
Volume, cm^3	80.28	Solids Height, cm	1.401
Water Content, %	27.67	Volume of Solids, cm^3	44.30
Wet Mass, g	158.38	Volume of Voids, cm^3	35.98
Dry Mass, g	124.05	Degree of Saturation, %	95.4

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	2.539	0.812	2.539				
4.76	2.536	0.810	2.537	2	6.82E-01	2.57E-04	1.72E-05
9.56	2.536	0.810	2.536	2	6.82E-01	8.21E-06	5.48E-07
19.52	2.545	0.816	2.540	2	6.84E-01	-3.64E-04	-2.44E-05
38.75	2.545	0.816	2.545	16	8.58E-02	6.14E-06	5.17E-08
77.55	2.530	0.806	2.537	18	7.58E-02	1.49E-04	1.11E-06
154.89	2.494	0.780	2.512	24	5.57E-02	1.83E-04	1.00E-06
309.65	2.447	0.746	2.471	12	1.08E-01	1.20E-04	1.26E-06
619.21	2.386	0.703	2.417	41	3.02E-02	7.76E-05	2.30E-07
1239.25	2.305	0.645	2.346	89	1.31E-02	5.15E-05	6.61E-08
2477.06	2.207	0.575	2.256	128	8.43E-03	3.12E-05	2.58E-08
1239.25	2.227	0.589	2.217				
309.65	2.297	0.639	2.262				
77.55	2.375	0.695	2.336				
19.52	2.453	0.751	2.414				
4.76	2.505	0.788	2.479				

Note:
k calculated using cv based on t_{90} values.
Sample swelled under 38.75kPa

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

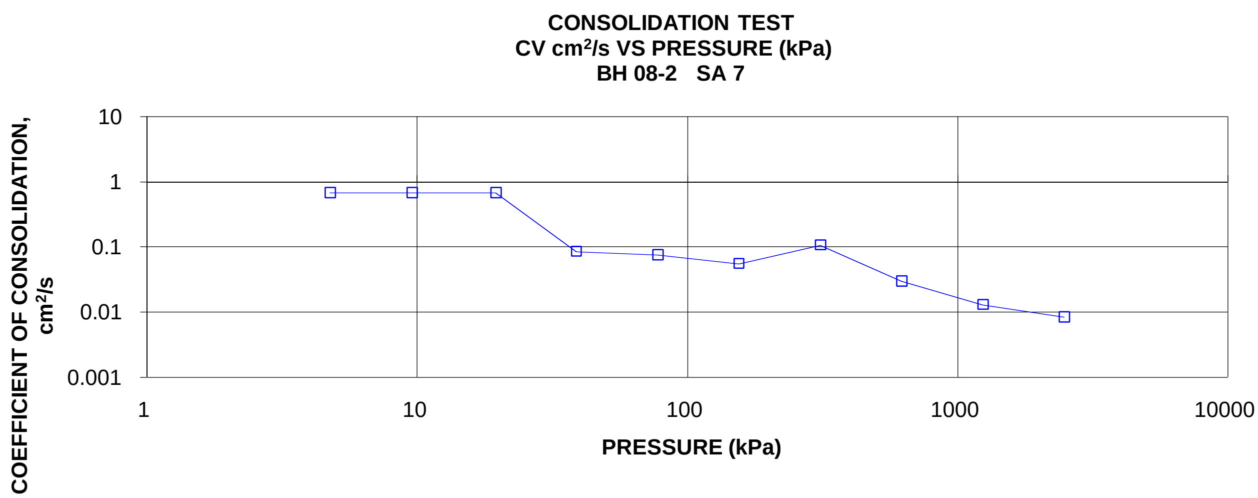
Sample Height, cm	2.51	Unit Weight, kN/m^3	19.87
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	15.36
Area, cm^2	31.62	Specific Gravity, measured	2.80
Volume, cm^3	79.21	Solids Height, cm	1.401
Water Content, %	29.40	Volume of Solids, cm^3	44.30
Wet Mass, g	160.52	Volume of Voids, cm^3	34.90
Dry Mass, g	124.05		

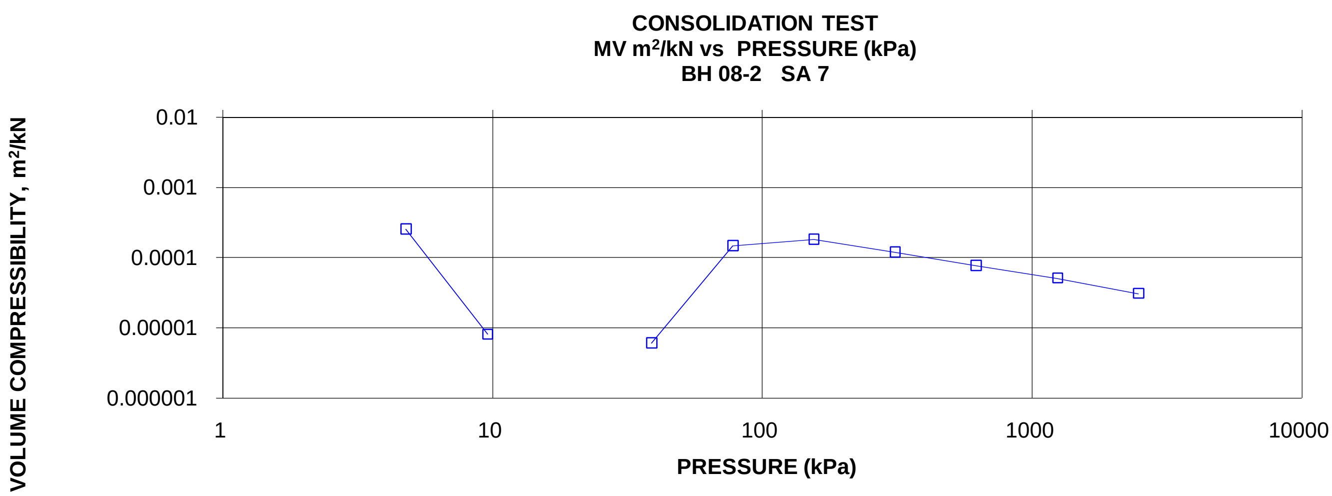
Prepared By: LFG

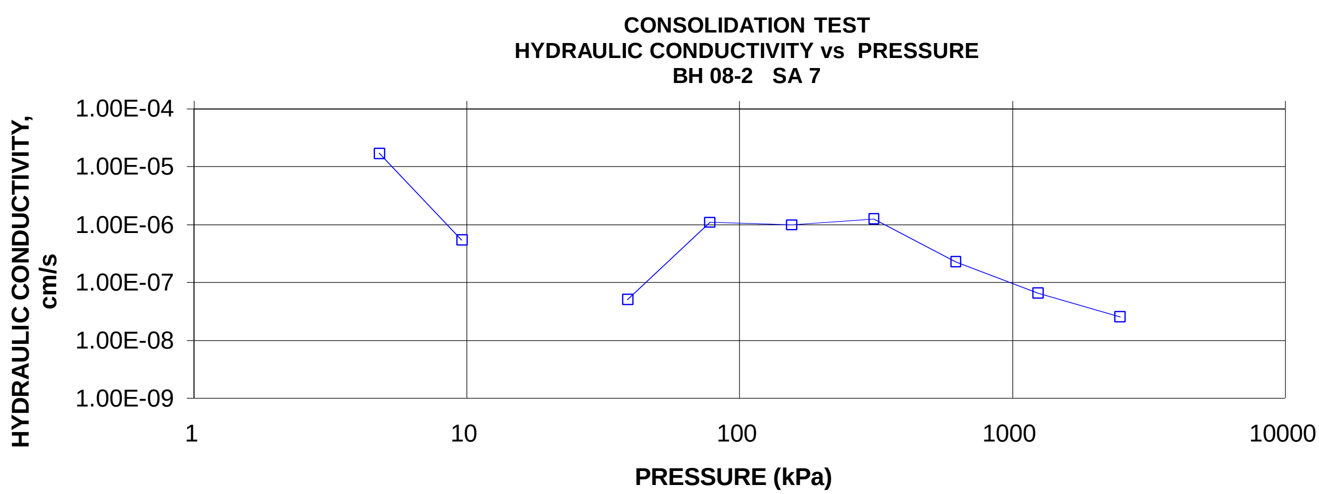
Golder Associates

Checked By: MM

OEDOMETER CONSOLIDATION SUMMARY Clay Fill	FIGURE C13 (Sheet 2 of 4)

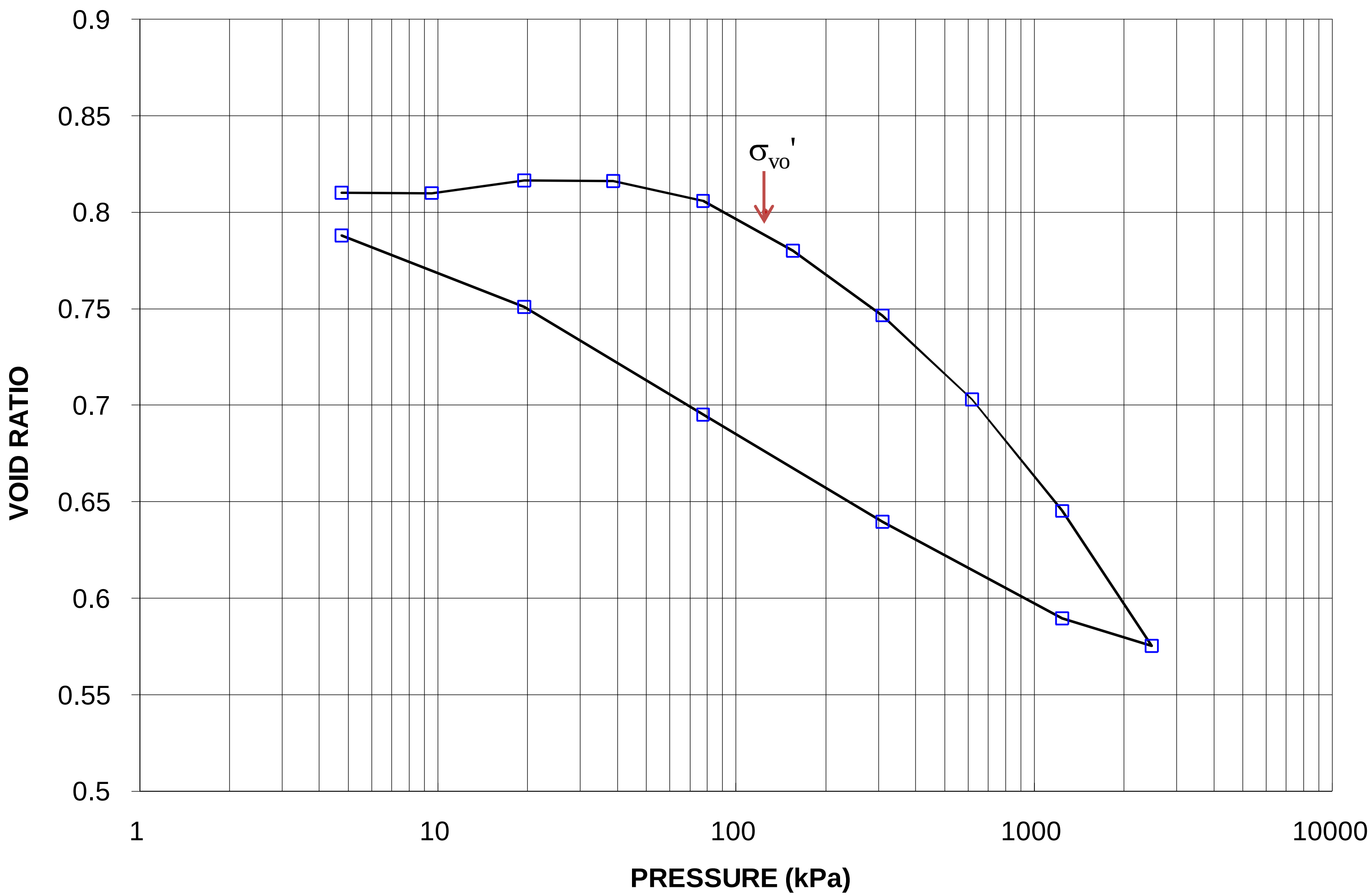






Project No.08-1111-0031
Prepared By: LFG

CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH 08-02 SA 7



Golder Associates

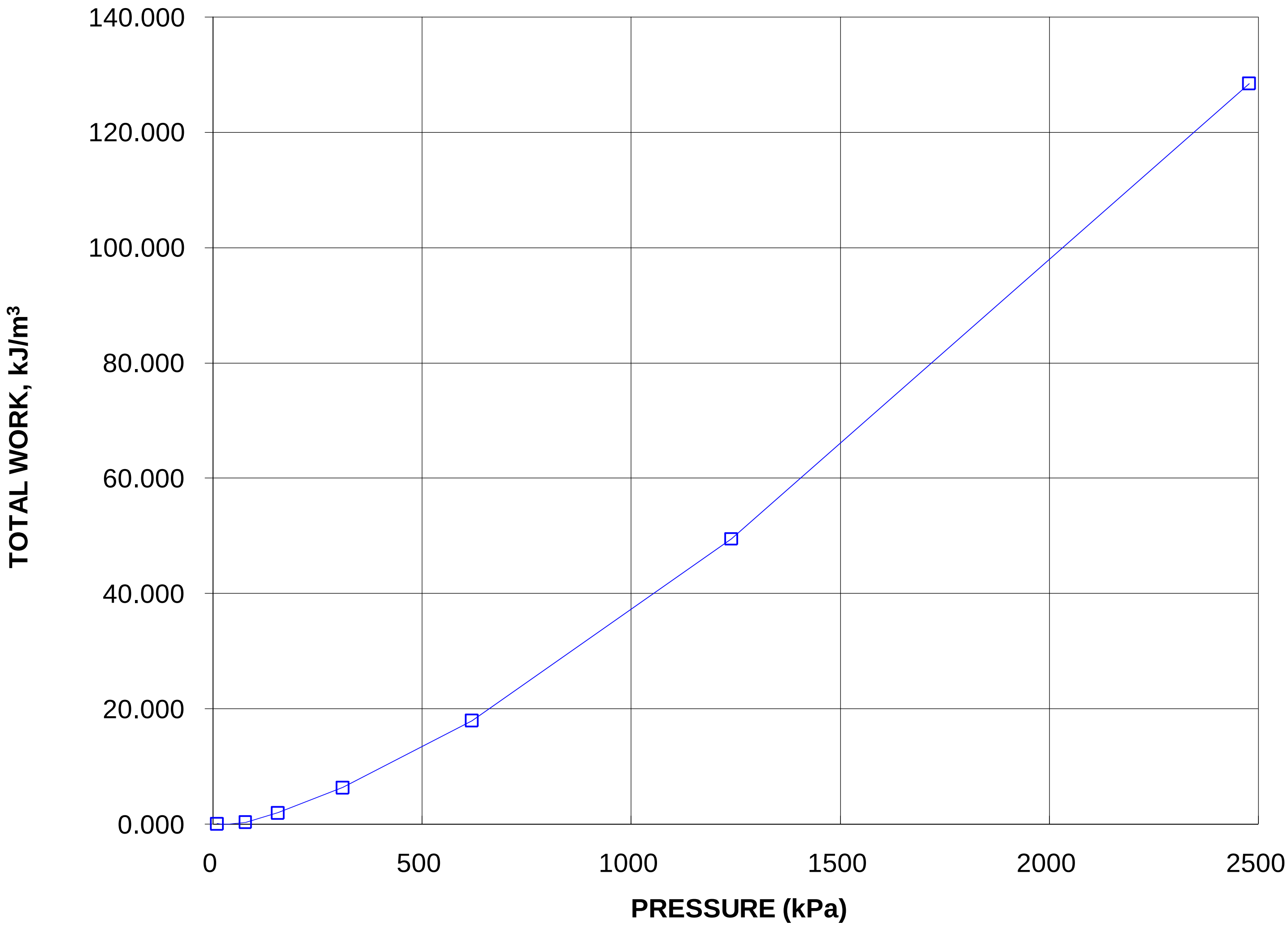
CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE C13
(SHEET 3 OF 4)

Project No.08-1111-0031
Prepared By: LFG

Golder Associates

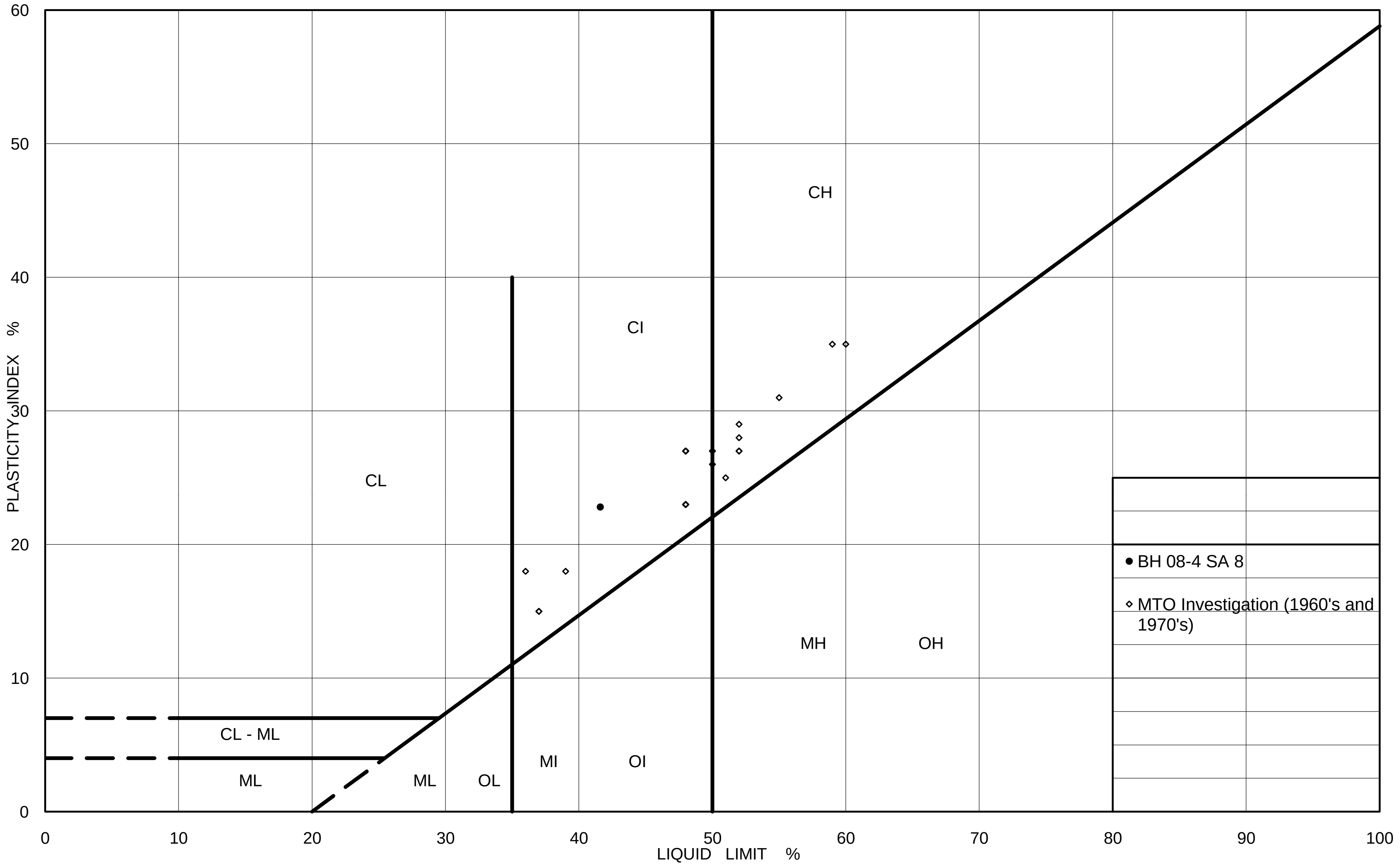
CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs PRESSURE
BH 08-02 SA 7



Checked By: MM

CONSOLIDATION TEST
TOTAL WORK VS. PRESSURE

FIGURE C13
(SHEET 4 OF 4)



Ministry of Transportation

Ontario

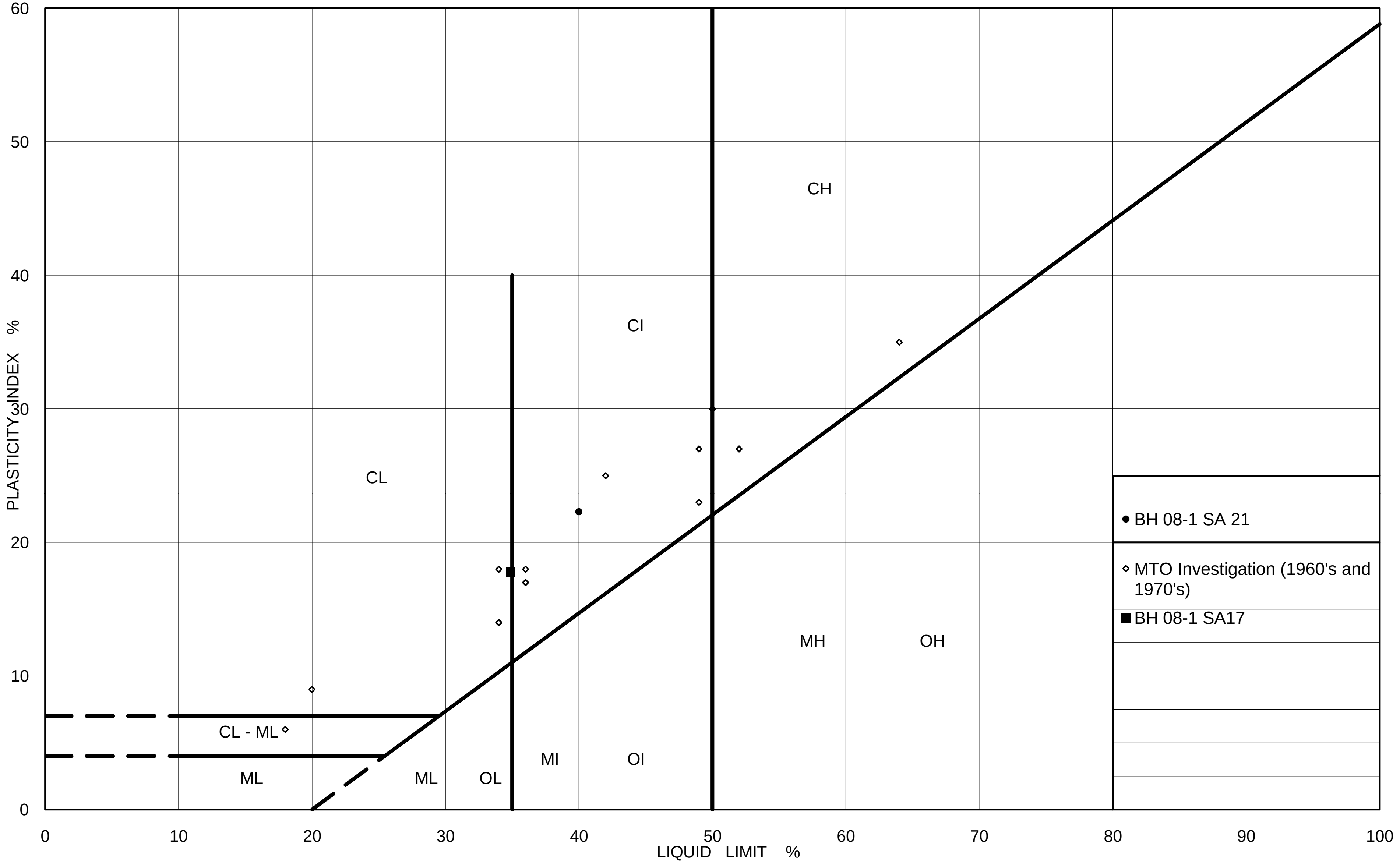
PLASTICITY CHART
Upper Silty Clay to Clay (Crust)

Figure No. C14

Project No. 08-1111-0031

Checked By: MWK

Oct 75, FF-S-21



Ministry of Transportation Ontario	PLASTICITY CHART Lower Clayey Silt to Silty Clay	Figure No. C15 Project No. 08-1111-0031 Checked By: MWK

OEDOMETER CONSOLIDATION SUMMARY

Lower Clayey Silt to Silty Clay

FIGURE C16

(Sheet 1 of 4)

SAMPLE IDENTIFICATION

Project Number	08-1111-0031	Sample Number	17
Borehole Number	08-1	Sample Depth, m	19.8-20.4

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	3		
Date Started	11/12/2008		
Date Completed	11/27/2008		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.54	Unit Weight, kN/m^3	19.93
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	15.95
Area, cm^2	31.62	Specific Gravity, measured	2.76
Volume, cm^3	80.28	Solids Height, cm	1.497
Water Content, %	24.91	Volume of Solids, cm^3	47.32
Wet Mass, g	163.14	Volume of Voids, cm^3	32.96
Dry Mass, g	130.61	Degree of Saturation, %	98.7

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	2.539	0.696	2.539				
4.86	2.539	0.696	2.539	1	1.37E+00	4.05E-05	5.43E-06
9.55	2.533	0.692	2.536	4	3.41E-01	5.04E-04	1.68E-05
19.44	2.524	0.686	2.528	25	5.42E-02	3.58E-04	1.90E-06
38.70	2.506	0.674	2.515	27	4.97E-02	3.60E-04	1.75E-06
77.64	2.477	0.655	2.491	44	2.99E-02	2.92E-04	8.57E-07
154.93	2.436	0.628	2.456	41	3.12E-02	2.09E-04	6.40E-07
313.21	2.381	0.591	2.408	49	2.51E-02	1.38E-04	3.38E-07
621.25	2.312	0.545	2.346	23	5.07E-02	8.80E-05	4.37E-07
1241.33	2.221	0.484	2.266	34	3.20E-02	5.79E-05	1.82E-07
2479.90	2.132	0.424	2.176	49	2.05E-02	2.83E-05	5.69E-08
1241.33	2.138	0.428	2.135				
313.21	2.179	0.456	2.158				
77.64	2.232	0.491	2.205				
19.44	2.282	0.525	2.257				
4.86	2.319	0.550	2.301				

Note:
k calculated using cv based on t_{90} values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	2.32	Unit Weight, kN/m^3	21.10
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	17.46
Area, cm^2	31.62	Specific Gravity, measured	2.76
Volume, cm^3	73.34	Solids Height, cm	1.497
Water Content, %	20.80	Volume of Solids, cm^3	47.32
Wet Mass, g	157.78	Volume of Voids, cm^3	26.02
Dry Mass, g	130.61		

Prepared By: LFG

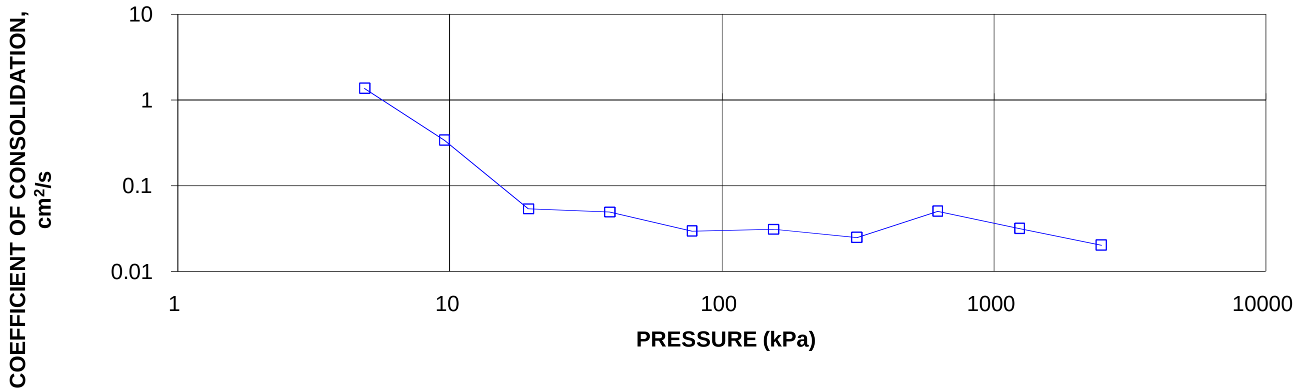
Golder Associates

Checked By: MM

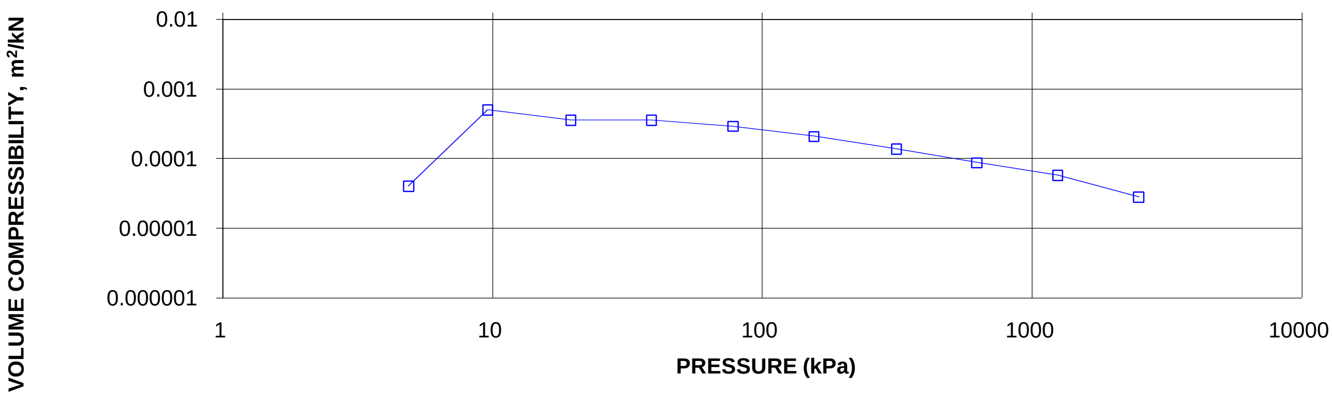
OEDOMETER CONSOLIDATION SUMMARY
Lower Clayey Silt to Silty Clay

FIGURE C16
(Sheet 2 of 4)

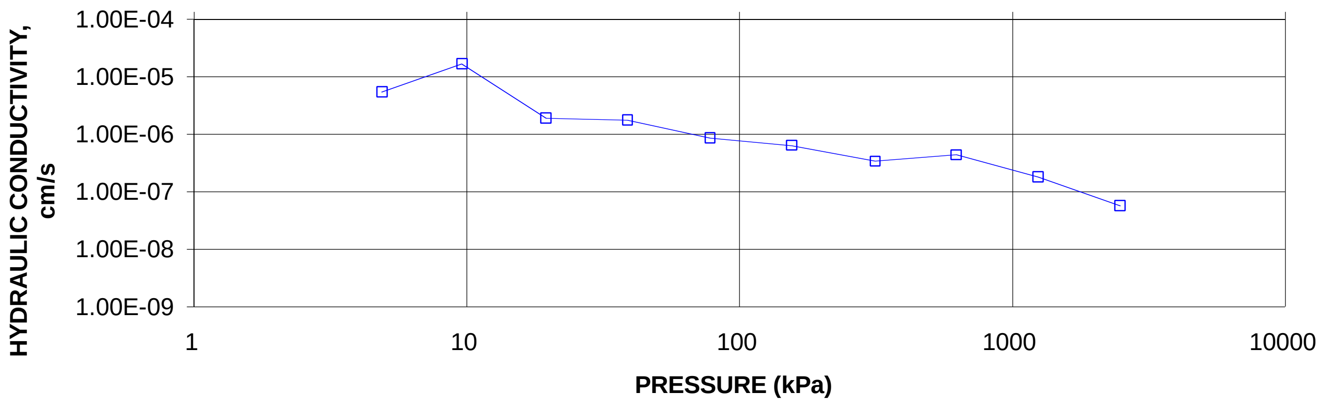
CONSOLIDATION TEST
CV cm²/s VS PRESSURE (kPa)
BH 08-1 SA 17



CONSOLIDATION TEST
MV m²/kN vs PRESSURE (kPa)
BH 08-1 SA 17



CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH 08-1 SA 17

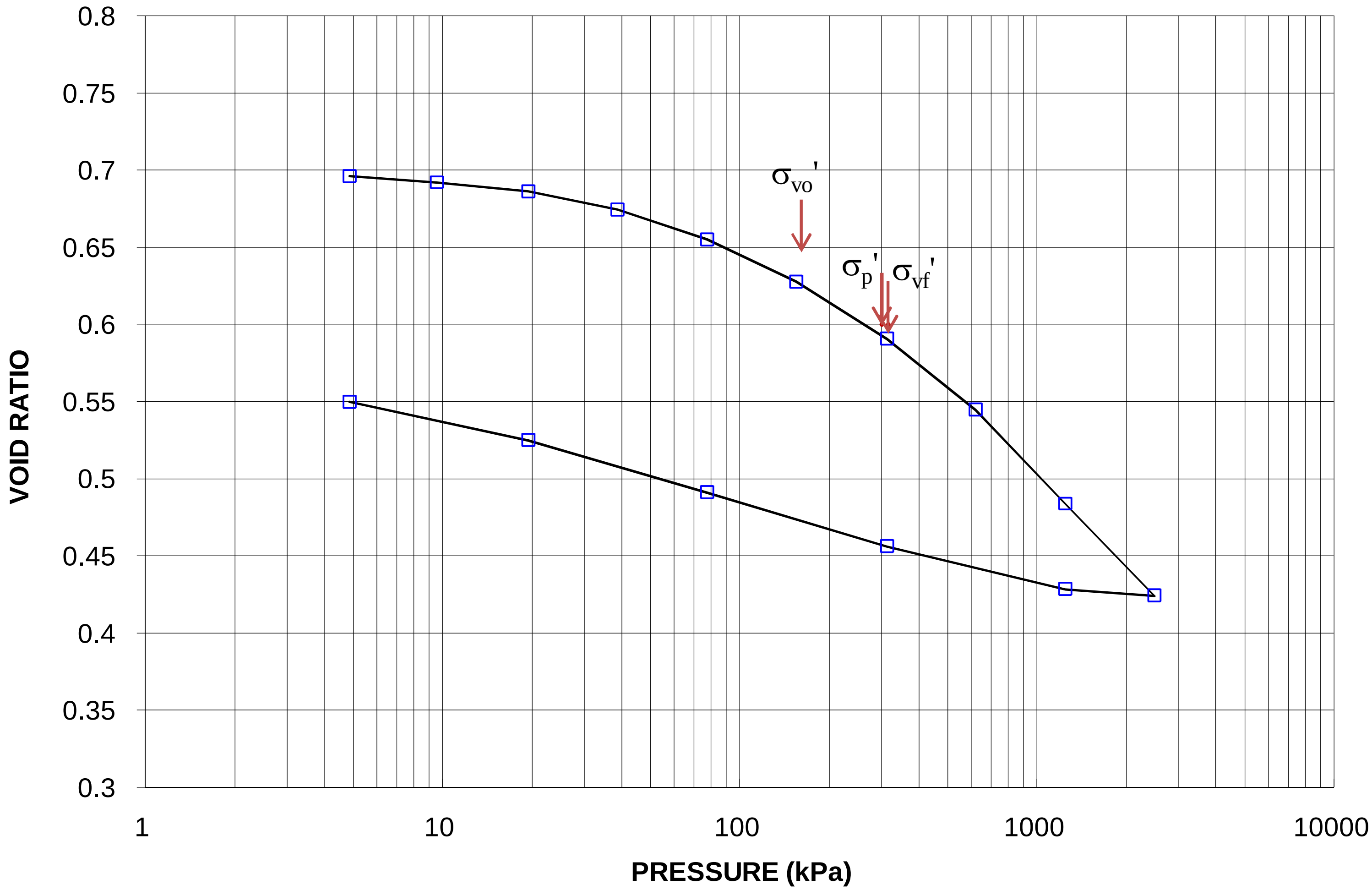


Project No. 08-1111-0031
Prepared By: LFG

Golder Associates

Checked By: MM

CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH 08-1 SA 17



Project No.08-1111-0031

Prepared By: LFG

Golder Associates

Checked By: MM

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

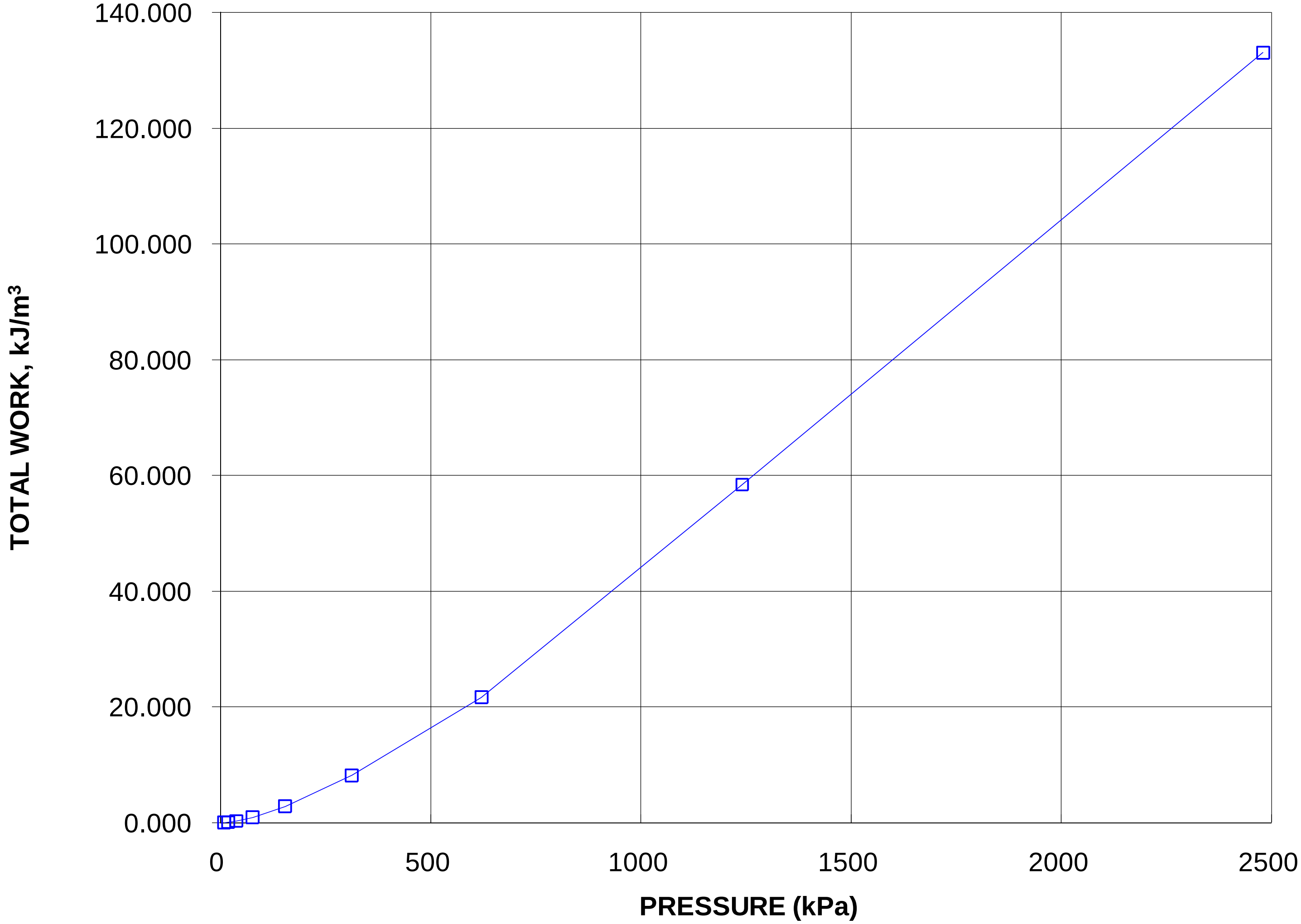
FIGURE C16
(Sheet 3 of 4)

Project No.08-1111-0031

Prepared By: LFG

Golder Associates

CONSOLIDATION TEST
TOTAL WORK, kJ/m³ vs PRESSURE
BH 08-01 SA 17



Checked By: MM

CONSOLIDATION TEST
TOTAL WORK VS. PRESSURE

FIGURE C16
(Sheet 4 of 4)

OEDOMETER CONSOLIDATION SUMMARY

Lower Clayey Silt to Silty Clay

FIGURE C17

(Sheet 1 of 4)

SAMPLE IDENTIFICATION

Project Number	08-1111-0031	Sample Number	21
Borehole Number	08-1	Sample Depth, m	27.5-29.5

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	2		
Date Started	08/11/2008		
Date Completed	25/11/2008		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.54	Unit Weight, kN/m^3	19.21
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	14.71
Area, cm^2	31.57	Specific Gravity, measured	2.79
Volume, cm^3	80.28	Solids Height, cm	1.368
Water Content, %	30.57	Volume of Solids, cm^3	43.18
Wet Mass, g	157.28	Volume of Voids, cm^3	37.11
Dry Mass, g	120.46	Degree of Saturation, %	99.2

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	2.543	0.859	2.543				
4.84	2.544	0.860	2.544	5	2.74E-01	-8.12E-05	-2.18E-06
9.61	2.547	0.862	2.545	1	1.37E+00	-2.14E-04	-2.89E-05
19.31	2.545	0.861	2.546	11	1.25E-01	8.51E-05	1.04E-06
38.81	2.539	0.856	2.542	12	1.14E-01	1.17E-04	1.31E-06
77.76	2.524	0.845	2.531	29	4.68E-02	1.53E-04	7.04E-07
155.06	2.493	0.823	2.508	112	1.19E-02	1.55E-04	1.81E-07
309.67	2.449	0.790	2.471	383	3.38E-03	1.13E-04	3.75E-08
619.93	2.340	0.711	2.394	653	1.86E-03	1.37E-04	2.50E-08
1240.21	2.193	0.603	2.266	924	1.18E-03	9.38E-05	1.08E-08
2481.31	2.068	0.512	2.130	103	9.34E-03	3.94E-05	3.60E-08
1240.21	2.077	0.518	2.072				
309.67	2.131	0.558	2.104				
77.76	2.200	0.609	2.165				
19.47	2.266	0.657	2.233				
4.84	2.323	0.699	2.295				

Note:
k calculated using cv based on t_{90} values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	2.32	Unit Weight, kN/m^3	20.30
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	16.11
Area, cm^2	31.57	Specific Gravity, measured	2.79
Volume, cm^3	73.34	Solids Height, cm	1.368
Water Content, %	26.01	Volume of Solids, cm^3	43.18
Wet Mass, g	151.79	Volume of Voids, cm^3	30.16
Dry Mass, g	120.46		

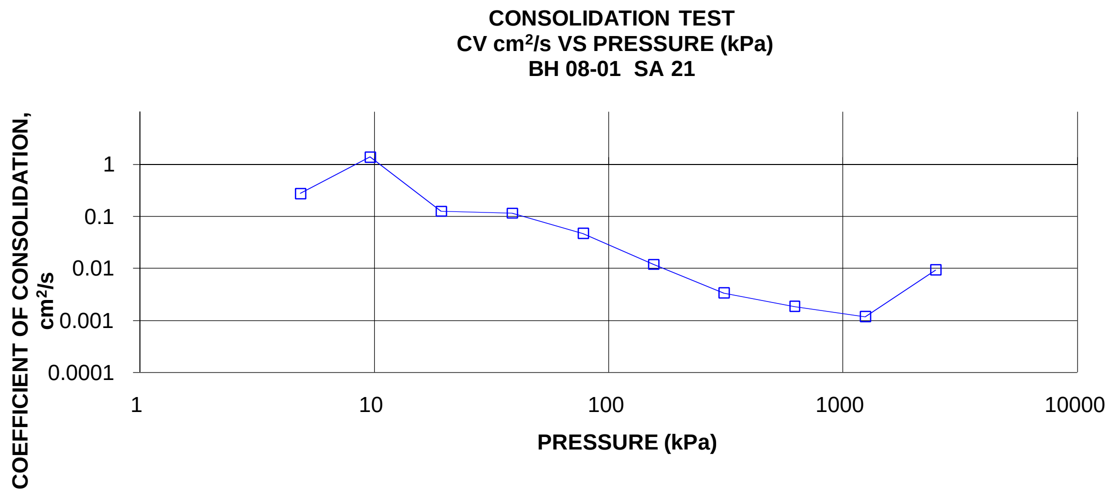
Prepared By: LFG **Golder Associates** Checked By: MM

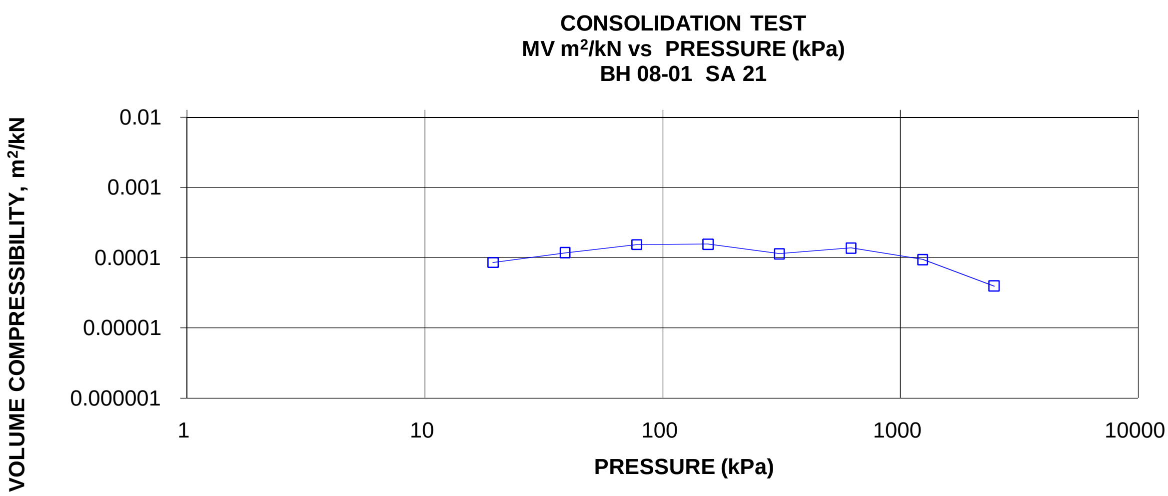
OEDOMETER CONSOLIDATION SUMMARY

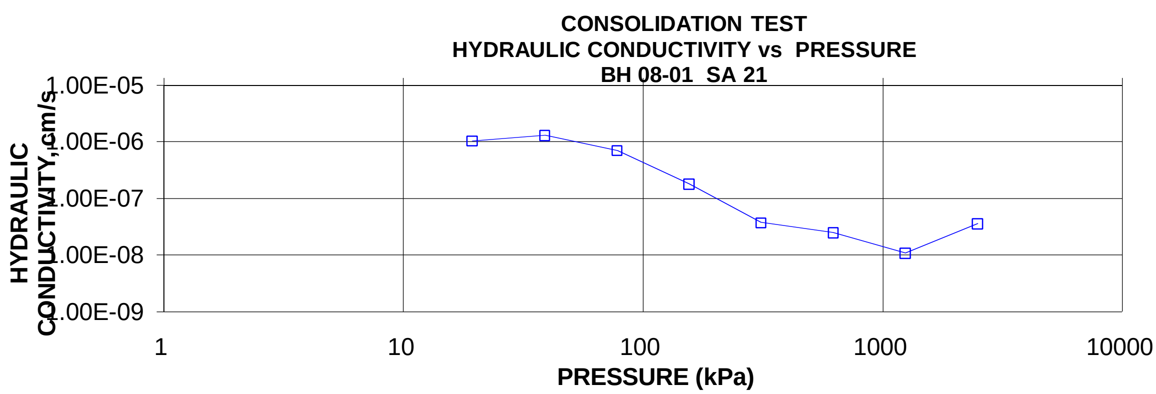
Lower Clayey Silt to Silty Clay

FIGURE C17

(Sheet 2 of 4)







Project No. 08-1111-0031

Prepared By: LFG

Golder Associates

Checked By: MM

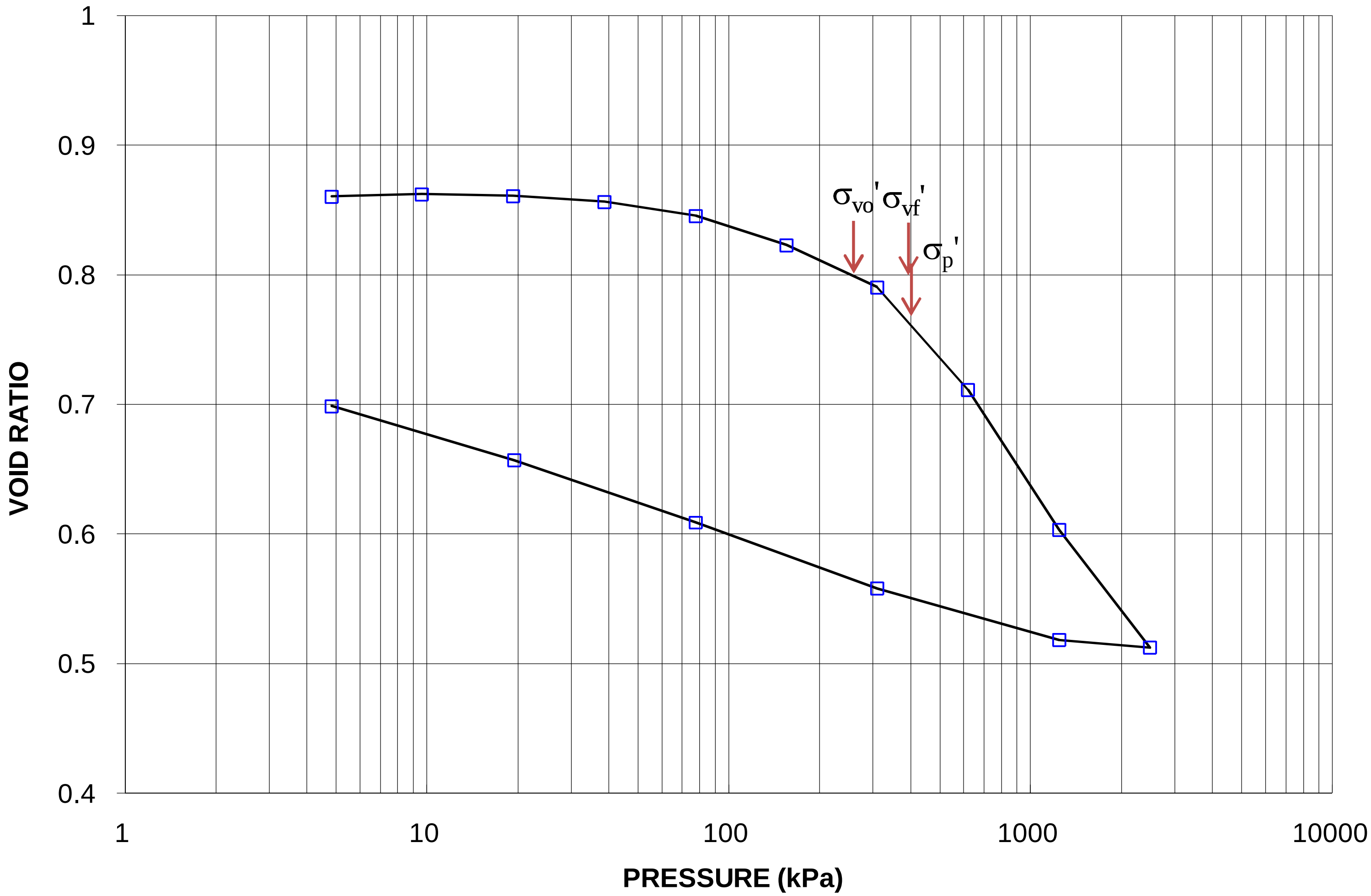
Project No.08-1111-0031

Prepared By: LFG

Golder Associates

Checked By: MM

CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH 08-01 SA 21



CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

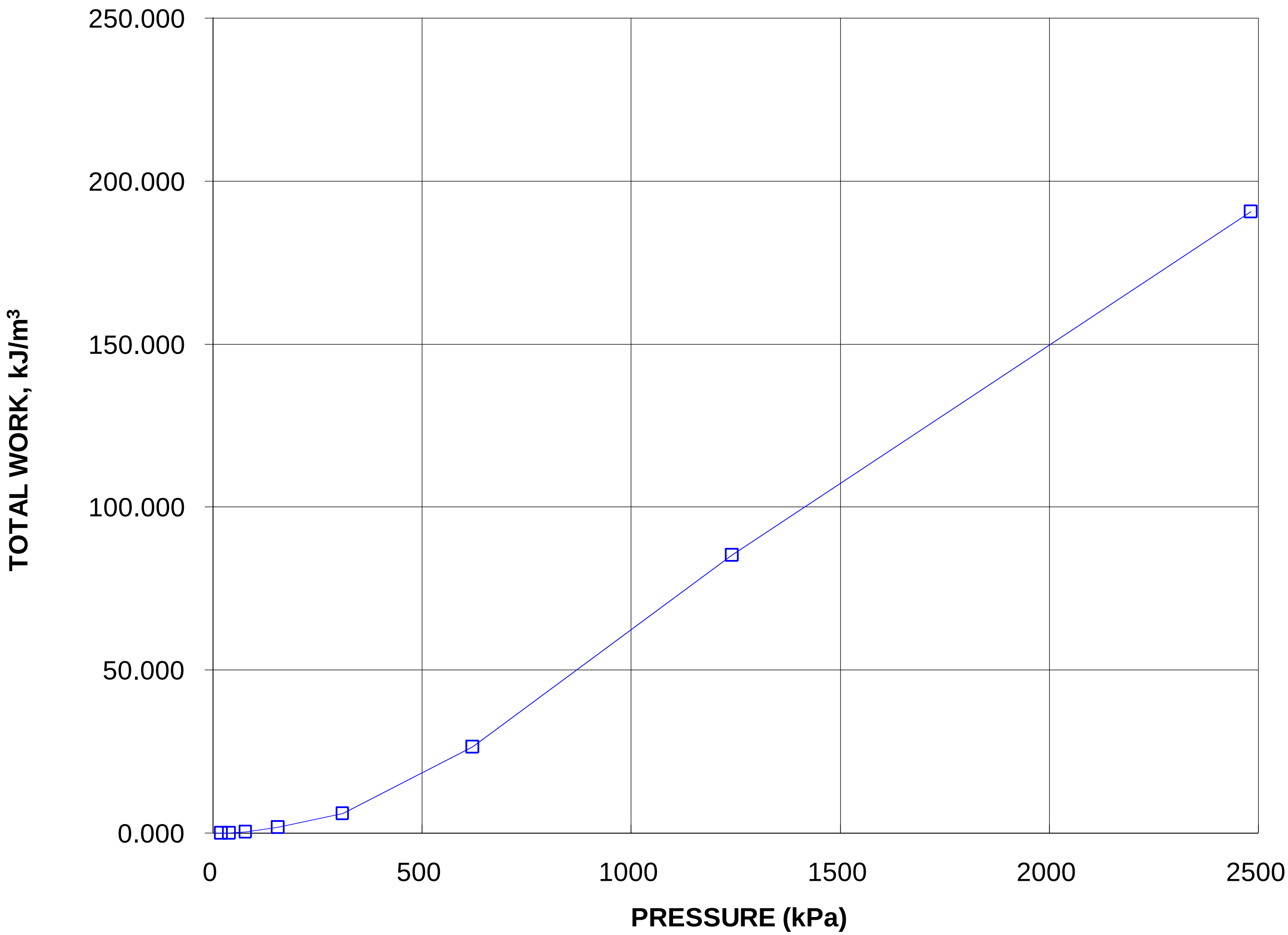
FIGURE C17
(Sheet 3 of 4)

Project No.08-1111-0031
Prepared By: LFG

Golder Associates

Checked By: MM

CONSOLIDATION TEST
TOTAL WORK, kJ/m³ vs PRESSURE
BH 08-01 SA 21



CONSOLIDATION TEST
TOTAL WORK VS. PRESSURE

FIGURE C17
(Sheet 4 of 4)

APPENDIX D

Previous Investigation Results

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

30L-45

MEMORANDUM

TO: Mr. T. J. Kovich, (2)
Regional Materials Engineer,
Central Region,
3501 Dufferin St., Downsview.

FROM: Foundations Office,
Design Services Branch,
West Bldg., Downsview.

ATTENTION:

DATE: July 17, 1972.

OUR FILE REF.

IN REPLY TO JUL 19 1972

SUBJECT:

FOUNDATION INVESTIGATION REPORT
For
Failure of Approach Embankments
Overhead Structure at the Crossing of
Hwy. #140 and C.N.R.
~~Township of Humberstone~~, County of ~~Welland~~
W.O. 72-11025 --- W.P. 60-6?-02
Contract ~~72-2~~ 70-212

30L-45
GEOCRES No.

Attached we are forwarding to you our detailed foundation investigation report on the subsoil conditions existing at the above-mentioned site.

We believe that the factual data and recommendations contained therein will prove adequate for your design requirements. Should additional information be required, please do not hesitate to contact our Office.

A. G. Stermac,
PRINCIPAL FOUNDATIONS ENGINEER.

AGS/ao
Attch.
cc: Messrs. F. G. Allen
D. W. Farren
A. Rutka
B. R. Davis
D. M. Hopper
P. J. Harvey
C. R. Robertson (Attn: D. Waller - 2)
G.C.E. Burkhardt
B. J. Giroux
G. A. Wrong
B. A. Singh

Foundations Files
Documents

TABLE OF CONTENTS

FOUNDATION INVESTIGATION REPORT
For
Failure of Approach Embankments
Overhead Structure at the Crossing of
Hwy. #140 and C.N.R.
Township of Humberstone, County of Welland
W.O. 72-11025 -- W.P. 60-68-02

1. INTRODUCTION:

This Office carried out a sub-surface investigation for the then proposed structure at the crossing of Hwy. #140 and the C.N.R., in the Township of Humberstone, County of Welland, during October and November, 1968. Recommendations pertaining to the design of foundations, as well as the stability and settlement considerations associated with the approach fills were presented in Report No. W.J. 68-F-73, dated December 4, 1968.

The south and north approach fills to the structure were constructed in May, 1971. On July 5, 1971, major instability occurred along the south approach; instability also developed along the north approach during the latter part of August of the same year. Following these failures the Foundations Office was requested to carry out an investigation of sufficient scope to aid in the assessment of what remedial measures need be taken to ensure the stability of the approaches. The request was presented by Mr. T. J. Kovich, Regional Materials Engineer, Central Region.

Visual observations have been made by both personnel from the Central Region Materials Section as well as the Foundations Office. In addition, sub-surface investigations have been carried out, at two different periods, in order to assess the physical properties of the fill and the parent subsoil.

.......2

This report presents all the visual and factual data accumulated, prior to and following the failure. In addition, remedial measures are proposed which should ensure the long-term stability of the failed sections.

2. DESCRIPTION OF THE SITE AND GEOLOGY:

The site is located some 200 feet east of the intersection of Forkes Rd. and Kleinsmith Rd., approximately 2 miles east of Welland Junction. At this location the C.N.R. tracks, which run parallel to Forkes Rd., are about 100 feet to the north. The tracks are elevated about 4 feet above the surrounding ground level on a 25 feet wide embankment. Forkes Rd. is a two-lane, paved county road; the profile grade of this road is about 1 to 2 feet above the surrounding terrain. Shallow ditches run along both sides of Forkes Rd., as well as the C.N.R. embankment.

The surrounding area is generally flat-lying; the surficial drainage is very poor. The land to the north of the site is used for farming purposes, while the land to the south is, at present, abandoned.

Physiographically, the site is situated in the region known as the Haldimand Clay Plain. In this area the subsoil is composed of extensive, mainly glacial-lacustrine deposits, laid down in glacial Lake Warren, during the Wisconsinan Age. These deposits are composed of stratified silts and clays, and are generally underlain by a basal glacial till sheet, which in turn, is followed by dolomitic linestone bedrock of the Salina formation, Silurian period.

3. CONSTRUCTION DETAILS:

3.1) Structure Scheme:

The 42 feet wide overhead structure at the crossing of Hwy. #140 an the C.N.R. has three spans (40'-50'-40'). In the vicinity of the crossing the profile grades of the

C.N.R. and Hwy. #140 are at elevation 586 and 614, respectively. At these grades the maximum height of the approaches, in the longitudinal and transverse directions, are 26 and 32 feet, respectively.

The structure and related approaches have been constructed in an area where the predominant stratum is a stiff to hard clay to silty clay. The thickness of this deposit ranges from 73 to 82 feet. This cohesive stratum is underlain by a thin glacial till deposit then dolomite bedrock.

The two piers for the structure are founded on spread footings, located at elevation 575.5 - i.e., in the upper very stiff portion of the clay. It is understood that they were designed using an allowable bearing value of 2.5 t.s.f. The abutments are supported on hexagonal section (maximum dimension 16") pre-stressed concrete piles which supposedly are driven to bedrock. These piles were designed using an allowable load of 100 tons/pile.

The fill used to form the approaches was composed of a clay to silty clay, which was obtained from borrow pits located in close proximity to the site. This fill material is of similar geologic origin to that of the parent cohesive deposit across this site.

3.2) Observations During Placement of Fill:

Visual observations were made by Regional and District personnel during the placement of the fill to form the approaches to this structure, these are summarized in the paragraphs to follow.

Fill placement along the north approach commenced on May 4, 1971; the fill was placed to final grade. The fill placement along the south approach commenced on May 21, 1971; the height during this stage was approximately 4 to 5 feet below final grade. All the fill was placed directly on the existing terrain - i.e., the topsoil was not removed.

The surficial drainage, in the vicinity of the approaches, particularly the south, was, at the time of fill placement, generally poor. Numerous ponds of water existed in this area. The fill material placed in the lower portion of the embankment section, along the south approach, appeared to have a higher natural water content than the fill placed elsewhere on this site. These conditions would make it difficult to adequately compact the fill in the lower portion of the embankments, particularly along the south approach. Further, the availability of free water would tend to lead to softening of the clay fill with time.

4. DESCRIPTION OF FAILURE:

i) South Approach:

On July 5, 1971, major instability developed along the south approach, specifically between Stations 211 + 00 and 216 + 50. In this area the fill subsided about 2 feet. Longitudinal tension cracks, up to 3 feet wide, opened up within the main core. Further, bulging was noticed at the toe of the fill; the maximum extent of this bulge was 3 feet beyond the original geometry.

On September 14, 1971, the embankment was repaired. The revised sections, from Station 211 + 50 northerly to the south abutment incorporated 20 feet long mid-height berms. In addition, it was recommended that the surficial organic material, located at the toe of the original section, be sub-excavated to a minimum depth of 2 feet. The sub-excavation so formed was then to be backfilled with acceptable compacted earth material (refer to the memo written by Mr. M. Devata, Supervising Foundations Engineer, dated August 12, 1971).

A second failure occurred along this approach on October 1, 1971. The failure originally developed on the west side of the embankment, eventually enveloping the east side. The magnitude and extent of the subsidence, tension cracks and toe bulging were similar to those discussed previously.

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ii) North Approach:

The north ramp initially failed on August 30, 1971, approximately 1-1/2 months after the south ramp showed signs of distress. The degree of distress was, however, less than that along the south approach. The north approach was repaired on September 22, 1971, using the scheme adopted on the south approach. A second less severe failure occurred in the first week of January, 1972. The west side of this approach showed more distress than the east. Information,provided by District personnel, has indicated that the berm constructed on the east side, following the initial failure, was longer than that on the west.

5. FIELD AND LABORATORY INVESTIGATION:

5.1) General:

Four boreholes were put down for the original foundation investigation at this site, during October and November of 1968 (No.'s 1, 2, 3 and 4). Following the initial failure of the south approach fill in July, 1971, seven boreholes were put down in strategic areas (100 series numbering). In addition, six borings were put down (200 series numbering) in February, 1972, to investigate the reasons for the second failure. Representative samples were obtained during the various investigation phases. Groundwater level observations were carried out, throughout this period, in piezometers installed in both the fill and parent subsoil. In addition, the groundwater levels in the open boreholes, at the remaining locations were recorded.

The locations and elevations of all of the boreholes, which were surveyed by District #4 personnel, are shown on Drawing No. W.O. 72-11025A. A typical stratigraphical section across the site, inferred from the boring data, is plotted on this drawing.

All the samples were subjected to a visual examination in the field and subsequently in the laboratory. Following this examination, laboratory testing was carried out on selected representative samples. This testing is summarized on the borelog sheets and Figures 1 and 2 contained in the Appendix of this report.

5.2) <u>Subsoil and Bedrock Conditions:</u>

5.2.1) <u>Fill (Silty Clay to Clay)</u>:

A number of the borings, put down following the failures, penetrated the fill placed along the approaches; the maximum depth of fill encountered was of the order of 32 feet. The fill is composed of a clay to silty clay, with a trace of sand.

Atterberg limit testing, carried out on samples from the fill, indicate that the material has a plasticity in the intermediate to high range. The natural moisture content within the fill varied from 23 to [illegible] percent, in general, there is an increase in the lower portion of the fill immediately above its contact with original ground. The compaction characteristics of the fill material were determined by carrying out two laboratory standard Practor Compaction Tests; the results from this testing are summarized on Figure #1 in the Appendix of this report. The values obtained from this testing are summarized below.

Optimum Compaction Bulk Density - 122 to 122.5 p.c.f.
Optimum Compaction Water Content - 24%

Referring to thise values it can be seen that in many areas throughout the fill, particularly in the lower zones, the inplace moisture content is considerably higher than the optimum compaction water content. This is graphically illustrated on Figure #3 appended to this report.

The undrained shear strength properties of the fill were determined in the field as well as in the laboratory. This testing gave values which ranged from 1,200 p.s.f. to greater than 2,000 p.s.f. This would indicate that the consistency of the major portion of the fill ranges from stiff to hard. The standard penetration testing carried out gave 'N' values which corroborate the range in consistency quoted above.

.........7

A laboratory programme was carried out to determine the engineering properties of the cohesive fill in terms of effective stresses. This was done by carrying out a series of isotropical consolidated undrained triaxial compression tests, in which the excess pore water was monitored (C.I.U. test). The results of this testing, which are plotted on Figure #2, are summarized below.

Apparent Effective Cohesive Intercept (c') - 0-120 p.s.f.
Apparent Effective Angle of Internal Friction (ø') - 23°

5.2.2) Clay to Silty Clay:

The fill is underlain by a clayey topsoil, approximately 1 foot thick. The topsoil is followed by a 73 to 82 feet thick stratum, composed of a clay to silty clay with a trace of sand and gravel. The upper 15 to 20 feet of the deposit is brown in colour; it is considered that this zone has been desiccated. Numerous layers and seams of sand and silt, up to 3 inches thick, are present throughout the stratum.

The physical properties of the overall stratum, as determined by field and laboratory testing, are summarized on the borelog sheets; a brief resume follows.

Atterberg limit tests carried out on samples of the cohesive material indicate that it is inorganic with a plasticity in the intermediate to high range. The consistency of the overall stratum, as determined by the undrained shear strength testing, varies from hard to very stiff, in the upper 15 to 20 feet (desiccated zone), decreasing to very stiff to stiff with depth.

Effective stress testing was carried out on a sample from the parent cohesive stratum using the procedure outlined in sub-section 5.2.1). The results of this testing, which are plotted on Figure #3, are summarized below:

Apparent Effective Cohesive Intercept (c') - 280 p.s.f.
Apparent Angle of Internal Friction (ø') - 25°

The compressibility characteristics of this subsoil

.......8

were determined by laboratory consolidation testing, the results of which were summarized in report W.J. 68-F-73. This testing indicated that the clay is preconsolidated by about 2 to 4 t.s.f. in excess of the existing overburden pressure. It is estimated that the upper 15 to 20 feet of the stratum (desiccated crust) is preconsolidated by something in excess of 5 t.s.f.

5.2.3) Lower Deposits:

The cohesive stratum is underlain by a basically cohesive glacial till composed of a clayey silt with sand and gravel. The thickness of the glacial till varies from 1 to 6 feet. The standard penetration resistance or 'N' values vary from 29 blows/ft. to well over 100 blows/ft., indicating that the consistency of the cohesive deposit ranges from very stiff to hard.

The glacial till is underlain by a grey dolomite bedrock. The surface of the bedrock was encountered between elevations 497 and 500; which corresponds to depths of from 79 to 85 feet below existing ground surface.

6. GROUNDWATER CONDITIONS:

Groundwater level observations have been carried out during the period of the investigation in i) sealed piezometers installed in the fill as well as in the cohesive stratum, and ii) the open holes at the remaining boring locations. These observations are recorded on the borelog sheets and summarized on Drawing No. 72-11025A. The results indicate that, prior to placement of the fill, the groundwater level in the cohesive stratum ranged from elevation 576 to 579 - i.e., some 3 to 5 feet below ground surface. The piezometric groundwater level in the glacial till, underlying the clayey silt stratum, ranged from elevation 554 to 558 - i.e., some 25 feet below ground level. These observations would indicate that there is some downward drainage from the upper cohesive stratum down into the glacial till deposit.

Following placement of fill the water level, in the parent cohesive subsoil, rose to elevations between 588 and 603.

The variation is an indication of the build-up in excess pore water pressure due to the fill loading.

Water level observations, carried out in piezometers installed in the fill have given an erratic pattern. The results would seem to indicate that the upper portion of the fill is dry. The water level in some isolated areas of the lower zone of the fill (immediately above the topsoil) rose to about elevation 598 - i.e., a level some 17 feet above the original ground surface (refer to B.H. #210). This is probably due to the fact that this zone was in communication with free water during fill placement.

7. <u>DISCUSSION AND RECOMMENDATIONS</u>:

7.1) <u>Reasons for Failure</u>:

As discussed in detail in Section 4) the south approach exhibited more distress than the north. This being the case, the discussion contained herein will pertain primarily to the former. The instability could have originated as either a deep-seated rotational type of failure within the parent cohesive foundation subsoil, or alternatively a failure confined to the new fill. These two possibilities will be discussed in detail in the following paragraphs.

i) <u>Deep-Seated Failure in Foundation Subsoil</u>:

Stability analyses, carried out prior to the original construction of the embankment (refer to Report W.O. 68-F-73), have indicated that fills of the order of 30 feet in height will be stable, with respect to a deep-seated failure within the foundation subsoil, provided i) standard 2:1 slopes are employed and ii) suitable earth fill is used and it is properly compacted. These computations were carried out using a total stress approach where the analyses are based on the undrained shear strength of the fill and parent cohesive subsoil, as well as the magnitude of the induced loading. The pre-failure undrained shear strength profile for the parent subsoil is plotted on Figure #1 of the aforementioned report. In addition, the stability was checked in terms of effective stresses. In this method the stability is governed by the stress-strain characteristics of the fill and parent cohesive stratum as well as the buildup and eventual dissipation of excess pore water pressure due to load application. These computations also provided an adequate factor of safety with respect to a deep-seated failure (F.S. $\geqq$ 1.3).

The borings, put down in the affected areas following failure, have indicated that the shear strength pattern throughout the parent cohesive stratum has remained basically unaltered. If the failure was deep-seated, then, in the critical areas bounded by the surface of the failure envelope, the silty clay should have been remoulded due to shear deformation; this would have led to some reduction in strength in these zones. Since this was not found to be the case, it is inferred that the instability must have been attributed to something other than a failure within the parent cohesive foundation subsoil.

ii) Failure Within New Fill:

If the failures are not of a deep-seated nature, then they must have originated within the new fill. As mentioned in Subsection 3.2) the fill, in the lower portion of the embankments, was placed and compacted in a wet environment. Further, the topsoil was not stripped. It is inferred that these factors probably led to the formation of a softened zone which encompasses the lower 3 to 4 feet of the fill as well as the natural topsoil cover. The failure surface would then tend to be located within this zone, which would have been a path of least resistance. This mode of failure will be discussed in detail in the paragraphs to follow.

The stability of a critical section along the south approach (at Station 214 +50), prior to the original failure in July 1971, was studied in detail using the effective stress approach developed by Messrs. Bishop and Morgenstern.*

*Bishop, A.W. and Morgenstern, N., "Stability Coefficients for Earth Slopes," Geotechnique, Vol. 10, No. 4, 1960.

The following were assumed for computational purposes.

a) <u>Fill Details (Immediately Prior to Failure):</u>

Fill Height - 24 ft. (4 feet below proposed final grade).

Average Slope - 2-1/4:1

b) <u>Engineering Parameters (Predicted From Laboratory Testing Results):</u>

	Fill	
	Lower Softened Zone	Remaining
Apparent Effective Cohesive Intercept (C')	0	120 p.s.f.
Apparent Effective Angle of Internal Friction (Ø')	23°	23°

Average Pore Pressure Ratio (r_u) = 0.25

where $r_u = \frac{\Delta u}{\gamma H}$

Δ u - excess pore water pressure (p.s.f.)

γ - bulk unit weight of fill (p.c.f.)

H - height of fill (ft.)

The results of the computations have indicated that the fill section itself was in a limiting state of equilibrium (F.S. $\leqq$ 1.0) during this critical period. As such it is believed that the failure could have originated within this lower softened zone of the fill-topsoil complex.

An extension of these computations have indicated that, in order to ensure the long-term stability of this section, when constructed to final grade (height 28 feet), the side slopes should be constructed no steeper than 3-1/2:1 overall. In these computations it was assumed that a minimum factor of safety of 1.3 should be obtained to ensure the stability of the section being investigated.

7.2) <u>Recommended Remedial Measures:</u>

In order to ensure the long-term stability of the approach fills at this site it will be necessary to employ flatter overall slopes; this could be accomplished by constructing counter balancing mid-height berms. In addition, it is recommended

.....12

that a reinforced zone, composed of either rock fill or a granular type of material, be placed at the toe of the reconstructed section. The final selection of the material to be used in this toe zone is to be decided upon by the Central Regional Materials Section. This reinforced zone, which should extend a minimum of 2-1/2 feet into the fill and 2-1/2 feet into natural ground (maximum thickness 5 feet), will serve two main purposes; namely, it will,

i) provide a zone of higher shear strength and thus improve the stability of any potential failure surfaces passing through this area, and

ii) confine any softened material located beneath the core of the embankments, thus preventing the tendency for such soils to undergo large lateral strains.

Stability analyses were carried out to determine what berm lengths would be required, for various fill heights, when such a composite section is employed. These analyses were based on a method developed by N. Janbu*. Using this method, the critical surface need not be cylindrical in shape, instead it may assume any general configuration and thus maximize its length within zones of relative weakness. Based on these computations a revised geometry is recommended for sections along the south approach extending from Station 211 + 00 northerly to 217 + 00; these are shown on Drawing No. 72-11025B. Referring to this drawing, it can be seen that:

i) the maximum length of berm recommended is 35 feet (Stations 216 + 00 and 217 + 00 where the height of fill is of the order of 32 feet),

ii) all slopes are 2:1. The berm, however, should slope towards the top at 20:1.

iii) the reinforced toe is to extend from Station 212 + 00 to 217 + 00. The recommended dimensions of this toe are shown on the sections presented on Drawing 72-11025B.

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*Janbu, N. "Stability Analysis of Slopes with Dimensionless Parameters," Harvard Soil Mechanics Series, No. 46, 1954.

In order to relieve the build up of excess hydrostatic groundwater pressure positive drainage measures should be provided within the reinforced toe.

The west side of the north approach should be reconstructed using the procedures outlined for the south approach. The berm lengths and extent of the reinforced toe, should be based on the requirements specified for the various fill heights along the south approach. As discussed in Subsection 4. ii) the east side of the north approach appeared to be stable following the second failure. This is inferred to be due to the fact that the berms constructed on the east side, following the first failure, were longer than those constructed on the west. This being the case it is believed that initially the reinforced toe section need not be installed along the east side. We would recommend, however, that this area be kept under observation during the reconstruction period. If any signs of distress are noticed, they should be brought to the attention of this Office so that additional remedial measures can be initiated to ensure the overall stability of this section of the north approach.

All loosened and disturbed fill material, located in areas affected by mass slumping and major tension cracks, along both approaches, should be removed prior to placing new fill in these areas. If so desired, this excavated material could be used to flatten the outer portion of the bermed slopes.

This report should be read in conjunction with a letter, dated August 12, 1971, which was written by Mr. M. Devata, Supervising Foundations Engineer, and addressed to Mr. D. Waller, Construction Engineer, District No. 4 (Hamilton).

8. <u>MISCELLANEOUS:</u>

The field work, performed during the periods of July 13 to 20, 1971, and February 3 to 17, 1972, was carried out under the supervision of Mr. S. A. Ahmad, Project Foundations Engineer.

The equipment used was owned and operated by Master Soil Investigation Ltd. and Dominion Soil Investigation Ltd., both of Toronto.

This report was written by Mr. B. T. Darch, Senior Foundations Engineer and reviewed by Mr. M. Devata, Supervising Foundations Engineer.

B.T. Darch

B. T. Darch, P. Eng.

REGISTERED PROFESSIONAL ENGINEER
B. T. DARCH
PROVINCE OF ONTARIO

M. Devata

BTD/ao
M. Devata, P. Eng.

July 17, 1972.

APPENDIX I

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 1 (68-F-73)

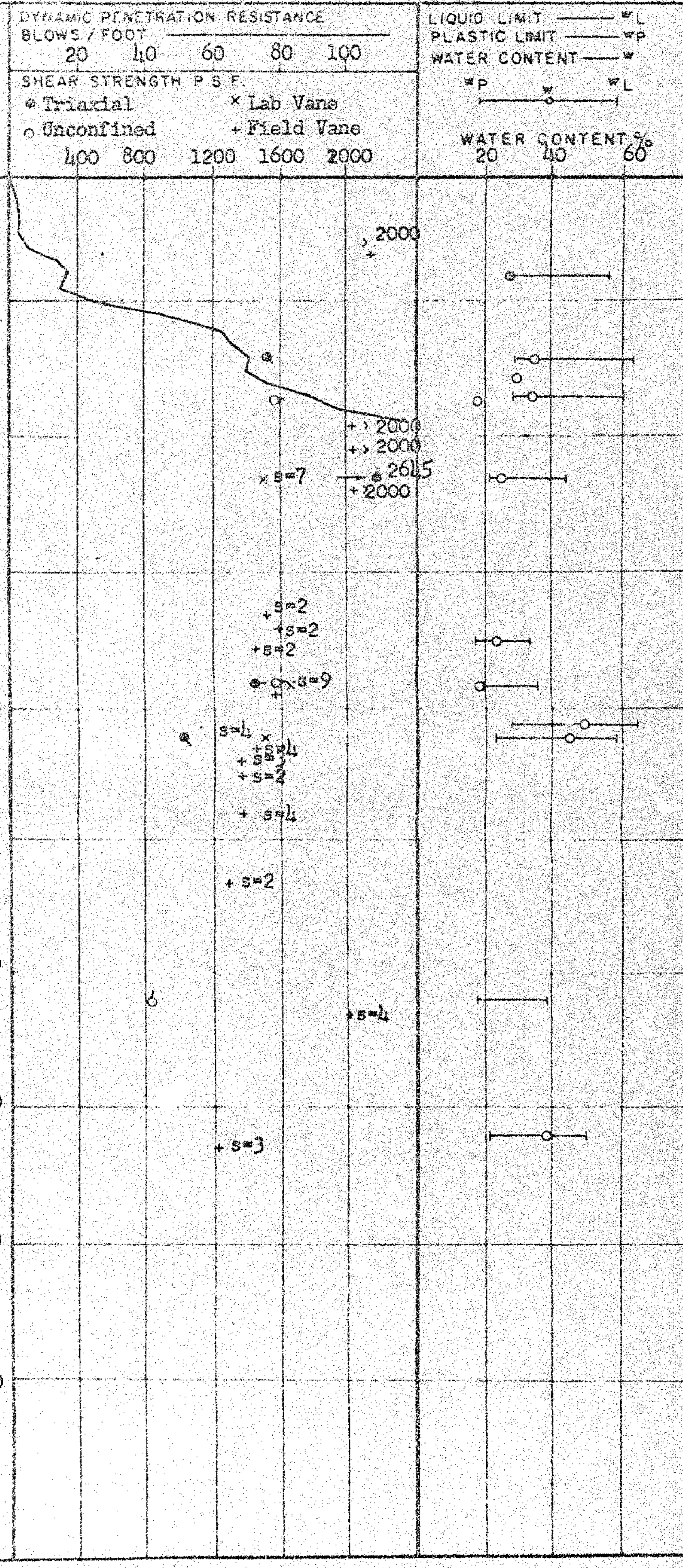
FOUNDATION SECTION

JOB 72-11025 | LOCATION Sta. 216+50 ℄ East Side Hwy. o/s 25' Rt. | ORIGINATED BY WH

W.P. 60-68-02 | BORING DATE Oct. 17 - Nov. 1, 1968 | COMPILED BY WH

DATUM Geodetic | BOREHOLE TYPE Cont. Flight auger & diamond drill | CHECKED BY

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS Gr.Sa.Si.C
579.0	Ground Level							
0.0	Silty clay to clay with trace of sand & gravel with occ. silt layers up to 3" thick throughout. Pocket of silty sand with gravel 3" thick at elev. 539		1A	SS	17			▽ 576
			2	SS	24	570		0 1 29 70
			3	SS	24			
			4	TW	PH		116	0 1 33 66
			5	SS	17			
			6	3" TW	PH		114	Piez.# Tip.elev.56
			7	SS	21	560		
			8	TW	PH		127.5	▽ 554.
			9	SS	20	550		
			10	TW	PM			
546.0 33.0	Clayey silt zone		10A	TW	PM			
			11	SS	9			
			11A	TW	PM		127	
540.0 39.0	Brown to reddish brown Stiff to very stiff.		12A	TW	PM	540		
			12	TW	PM		111	0 1 22 77
			13	SS	16	530		
			14	TW	PM			
						520		
			15	TW	PH		121	
						510		
			16	SS	12			
500.5 78.5	(Glacial Till) Very dense sand with gravel & fragments of bedrock.		17	SS	40/4"	500		Piez.#
79.5	Dolomite bedrock with gypsum lenses. Occ. layers of calcareous shale.		18	BXL RC	80% Rec.			Tip Elev. 449
494.6 84.4	End of Borehole					490		



DEPARTMENT OF HIGHWAYS – ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 2 (68-F-73)

FOUNDATION SECTION

JOB 72-11025

W.P. 60-68-02

DATUM Geodetic

LOCATION Sta. 219+10 Ø East Side Hwy. o/s 38' Lt.

BORING DATE Oct. 23-29, 1968

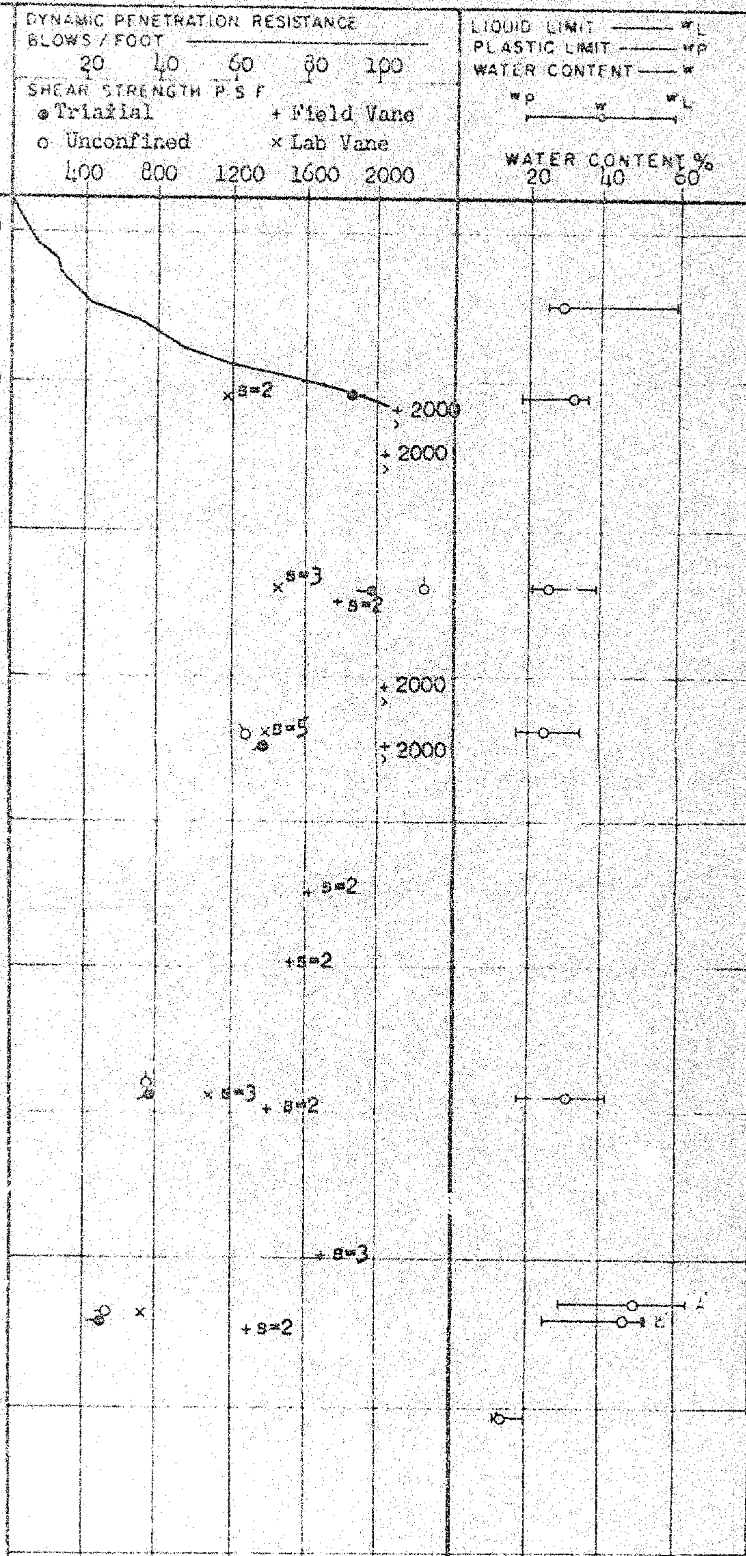
BOREHOLE TYPE Cont. flight auger & diamond drill

ORIGINATED BY WH

COMPILED BY WH

CHECKED BY [illegible]

ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BULK DENSITY P.C.F.	REMARKS Gr.Sa.Si.Cl.
582.0	Ground Level						
0.0	Clayey silt to clay with traces of sand & gravel. Occ. very thin grey silt seams containing clear gypsum crystals above elev. 559.	1	SS	26	580		
		2	SS	21			0 1 (99)
		3	SS	21			
		4	TW	PH	570	116	
		5	SS	14			
		6	TW	PH			
		7	SS	24	560		
							558
		8	TW	PH		128	
		9A	SS	10	550		
		10	TW	PH		125	
					540		
		11	TW	PH			
		12	SS	12	530		
	Brown to reddish brown						
		13	TW	PH	520	121	
	Firm to very stiff						
510.0 72.0	Occ. thin sand seams up to ½" thick	14	SS	9	510		
		15	TW	PH		112 108	
500.0 82.0	Glac. Till-clayey silt with sand, gravel and gypsum. Very stiff.	16	TW	PH	500		Piez. #3 Tip. Elev. 500
497.5 84.5	Dolomite bedrock with a bed of gypsum 2' thick. Sound. Grey.	17	SS	29			12 34 39 15
493.0		18	BXL RC	76% Rec			
89.0	End of Borehole				490		



DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 3 (68-F-73)

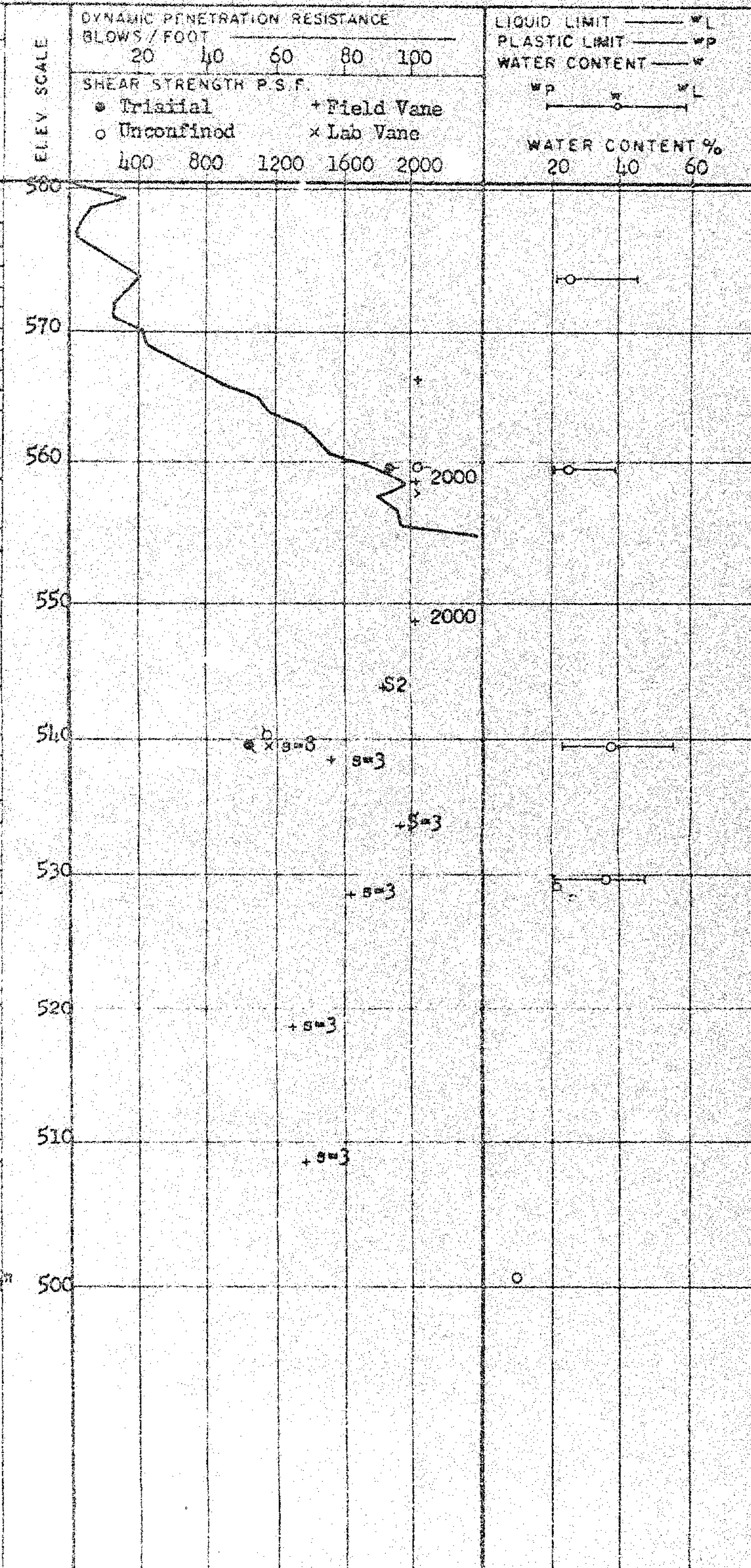
FOUNDATION SECTION

JOB 72-11025 LOCATION Sta. 217+53 ⌀ East Side Hwy. o/s 73' Rt. ORIGINATED BY WH

W P 60-68-02 BORING DATE Oct. 28-29, 1968 COMPILED BY WH

DATUM Geodetic BOREHOLE TYPE Cont. Flight Auger CHECKED BY

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FOOT	BULK DENSITY γ P.C.F.	REMARKS Gr.Sa.Si.Cl
580.5	Ground Level					
0.0	Roadway Fill Clayey silt with some sand & gravel. V.Stiff. dark grey.					
576.0 4.5		1	SS	17		▼ 575.5
		2	SS	36		
	Silty clay to clay with trace of sand occ. very thin grey silt seams containing clear gypsum crystals above elev. 554.	3	SS	27		
		4	TW	PH		
		5	SS	13		
		6	TW	PH	127	0 1 54 45
		7	SS	22		
		8	3" TW	PH		
		9	SS	7		
		10	3"TW	PH	125 111	
535.0 45.5	Occasional silt layers up to 3" thick.	11	SS	11		
		12	3"TW	PH		0 0 90 10 (silt seam)
525.0 55.5	Brown to reddich brown	13	SS	-		
	Firm to very stiff	14	TW	PH		
502.5 78.0	Glacial Till - clayey silt with sand & grav. Hard - Brown.	15	SS	70/6"		
500.0 80.5	End of borehole Probable Bedrock					



DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 4 (68-F-73)

FOUNDATION SECTION

JOB 72-11025 | LOCATION Sta. 221+00 ℄ East Side Hwy. o/s 5' Lt. | ORIGINATED BY WH

W P 60-68-02 | BORING DATE Oct. 30-31, 1968 | COMPILED BY WH

DATUM Geodetic | BOREHOLE TYPE Diamond Drill - NX Casing | CHECKED BY

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
582.0	Ground Level
0.0	Silty clay to clay with trace of sand & gravel occ. very thin grey silt seams containing clear gypsum crystals above elev. 560 Brown to reddish brown Stiff to hard
534.0 48.0	Occ. layers of silt up to 3" thick
520.0 62.0	
504.0 78.0	Glacial till-clayey silt with sand & grav. Hard. Brown to grey.
497.3 84.7	End of Borehole Probable Bedrock

SAMPLES

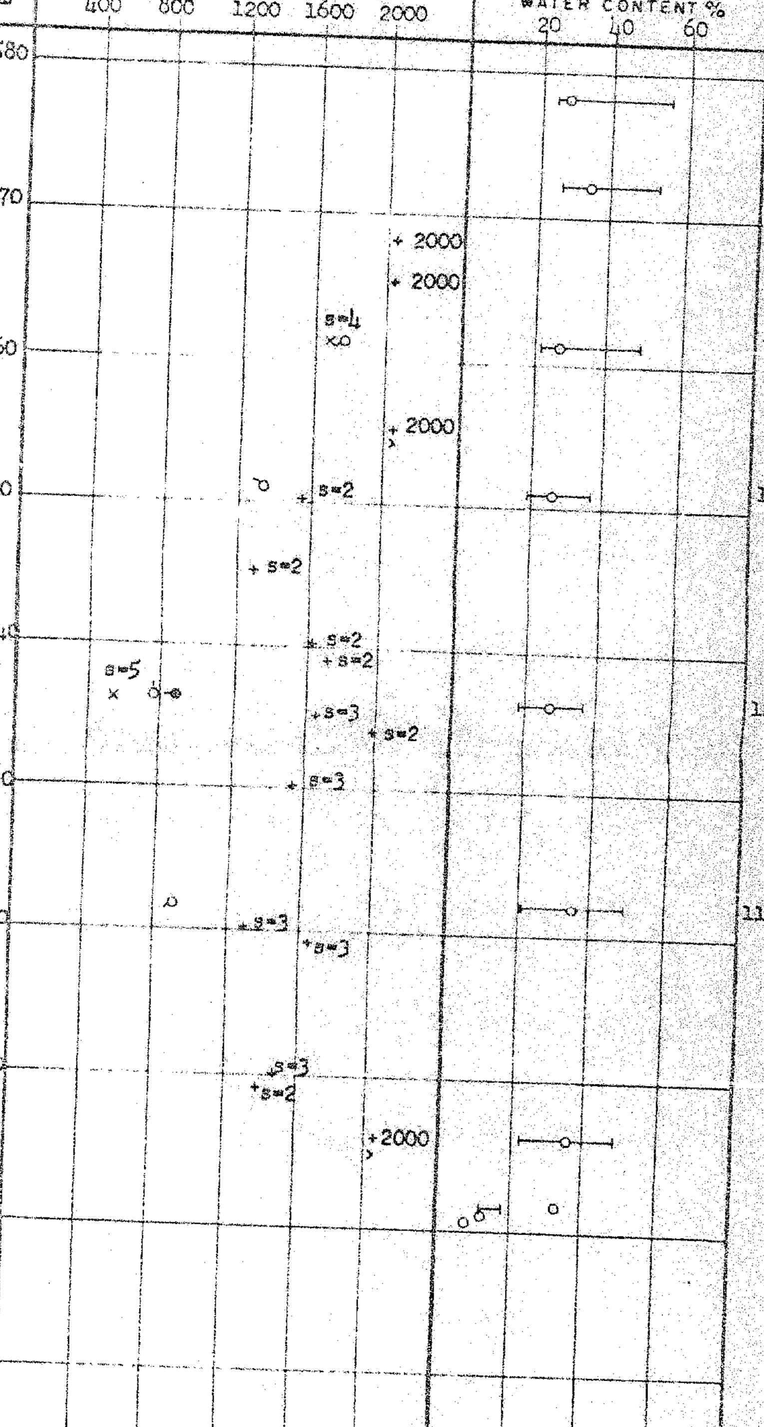
NUMBER	TYPE	BLOWS / FOOT
1	SS	76
2	SS	31
3	SS	19
4	TW	PM
5	SS	17
6	SS	17
6A	SS	13
7	TW	PM
8	SS	14
9	TW	PM
10	TW	PM
11	SS	19
12	TW	PM
13	SS	12
14	TW	PM
15	SS	81
16	SS	64/3"

ELEV SCALE

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT

SHEAR STRENGTH P.S.F.
⊕ Triaxial + Field Vane
○ Unconfined Comp. × Lab Vane

LIQUID LIMIT — w_L
PLASTIC LIMIT — w_P
WATER CONTENT — w



BULK DENSITY γ P.C.F.	REMARKS Gr.Sa.Si
	▼ 579 0=1 30
124	
127	
123	0 2 61 3
117	

7

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS
DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 101

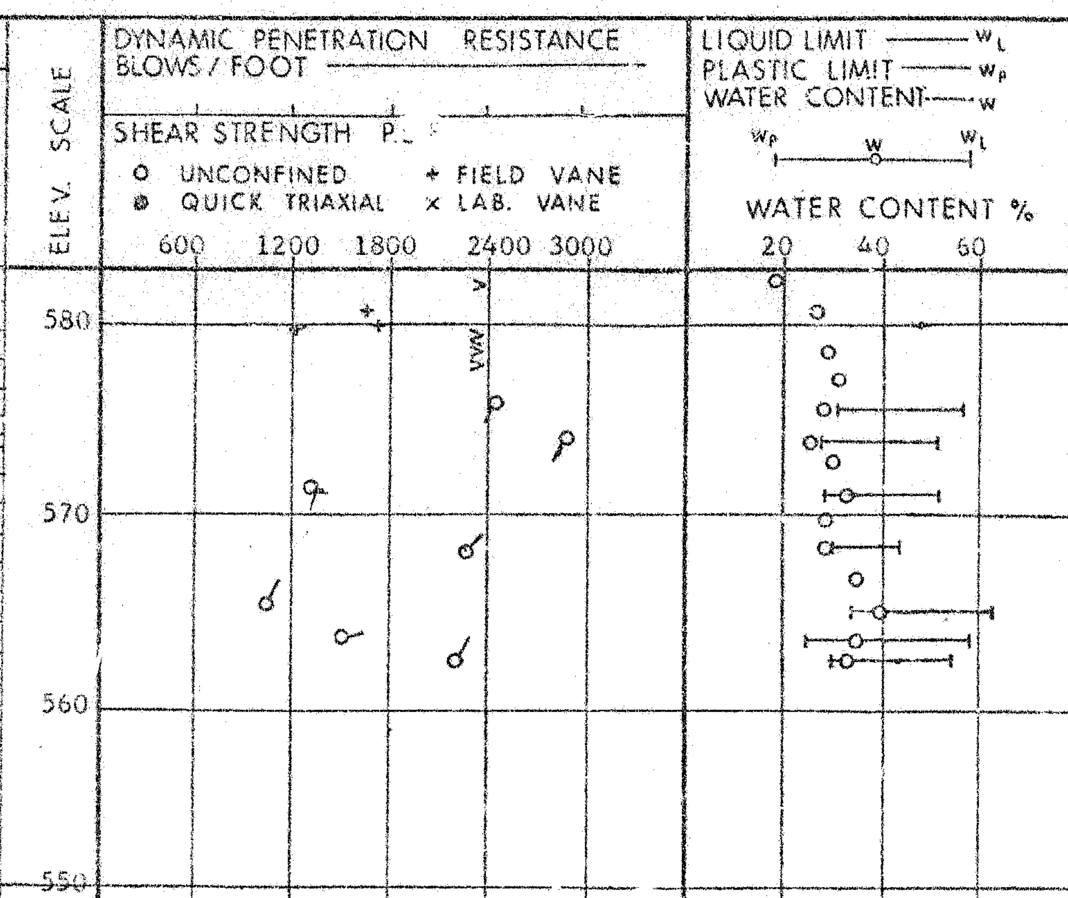
FOUNDATION SECTION

JOB 72-11025
LOCATION C.N.R. & Forkes Road. Sta. 214 + 60 32' Rt. Hwy. #140
ORIGINATED BY S.A.

W.P. 60-68-02
BORING DATE July 13, 1971
COMPILED BY W.V.U.

DATUM Geodetic
BOREHOLE TYPE NX Casing
CHECKED BY O.E.

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	BLOWS / FOOT	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
582.8	(Reddish-Brown)	1	SS	9		
		2	SS	14		▼ El.579.0
	Silty clay to clay,	3	SS	14		
	trace of sand.	4	SS	7		
		5	TW	E.	125	
	(Grey-Brown)	6	TW	PM	127	
		7	SS	14		
		8	TW	PM	121	
		9	SS	13		
	Stiff to very stiff.	10	TW	PM	124.5	
		11	SS	13		
		12	TW	PM	114	
		13	TW	PM	118	
		14	TW	PM	120	
		15	TW	PM		
		16	TW	PM		
551.2		17	TW	PM		
31.6	End of borehole.					



FORM OB-M[illegible]26 (REV. 1971)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS
DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 102

FOUNDATION SECTION

JOB 72-11025
W.P. 60-68-02
DATUM Geodetic

LOCATION Forkes Road & C.N.R., Sta. 214 + 60 32' Rt. Hwy. 14
BORING DATE July 14-15, 1971
BOREHOLE TYPE NX-AX Casing (Dry Boring)

ORIGINATED BY S.A.
COMPILED BY W.V.U.
CHECKED BY [illegible]

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL
603.3	Silty clay to clay, trace of sand (Fill) (Grey-Brown) Stiff to very stiff.		1	SS	10	600		Dry
			2	SS	8			
			3	SS	7			
			4	TW	PM			
			5	TW	PM		122	
			6	TW	PM		120	
			7	TW	PM		122	
			8	TW	PM	590	122	
			9	TW	PM		124.5	
			10	TW	PM		121	
			11	TW	PM		127	
			12	TW	PM		124	
			13	TW	PM			
579.3			14	SS	---	580	121	
			15					
24.0	End of borehole.							

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT

SHEAR STRENGTH P.S.F.
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB. VANE
600 1200 1800 2400 3000

LIQUID LIMIT — w_L
PLASTIC LIMIT — w_P
WATER CONTENT — w

WATER CONTENT %
20 40 60

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS
DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 102 A

FOUNDATION SECTION

JOB 72-11025
W.P. 60-68-02
DATUM Geodetic

LOCATION Forkes Road & C.N.R., Sta. 214+60 32' Rt. Hwy. #140
BORING DATE July 15, 1971
BOREHOLE TYPE NX

ORIGINATED BY S.A.
COMPILED BY W.V.U.
CHECKED BY

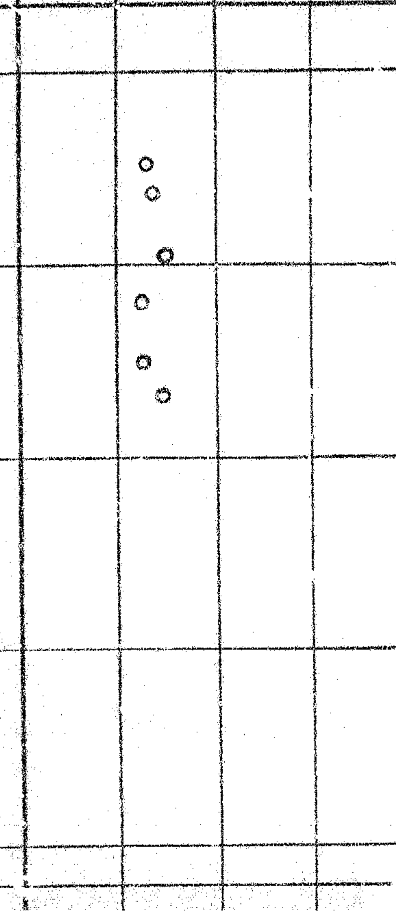
SOIL PROFILE ELEV / DEPTH	DESCRIPTION	SAMPLES NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	REMARKS GR.SA.SI.CL
603.3						
	Silty clay to clay, trace of sand - Fill. Reddish-Brown. Stiff to very stiff.				600	
		1	SS	20		
		2	SS	12		
		3	SS	4	590	
		4	SS	9		
		5	SS	10		
		6				
		7	SS	15		
578.8		8	SS	12	580	
577.8	Clayey Topsoil.	9	SS	33		El. 577.0
25.5	Silty clay to clay, trace of sand. Occasional silt and sand layers up to 3" thick. Stiff to very stiff.	10	SS	11		
		11	SS	57		
		12	SS	27	570	
		13	SS	13		
		14	SS	12		
557.8		15	SS	15	560	

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT

SHEAR STRENGTH P.S.F.
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB. VANE
600 1200 1800 2400 3000

LIQUID LIMIT — w_L
PLASTIC LIMIT — w_P
WATER CONTENT — w

WATER CONTENT %
20 40 60



BULK DENSITY γ P.C.F.

FORM OB-MT-26 (REV, 1971)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 103

FOUNDATION SECTION

JOB 72-11025

W.P. 60-68-02

DATUM Geodetic

LOCATION Forkes Road & C.N.R., Sta. 214+50 18' Lt. Hwy. #140

BORING DATE July 15, 1971

BOREHOLE TYPE NX Casing

ORIGINATED BY S.A.

COMPILED BY W.V.U.

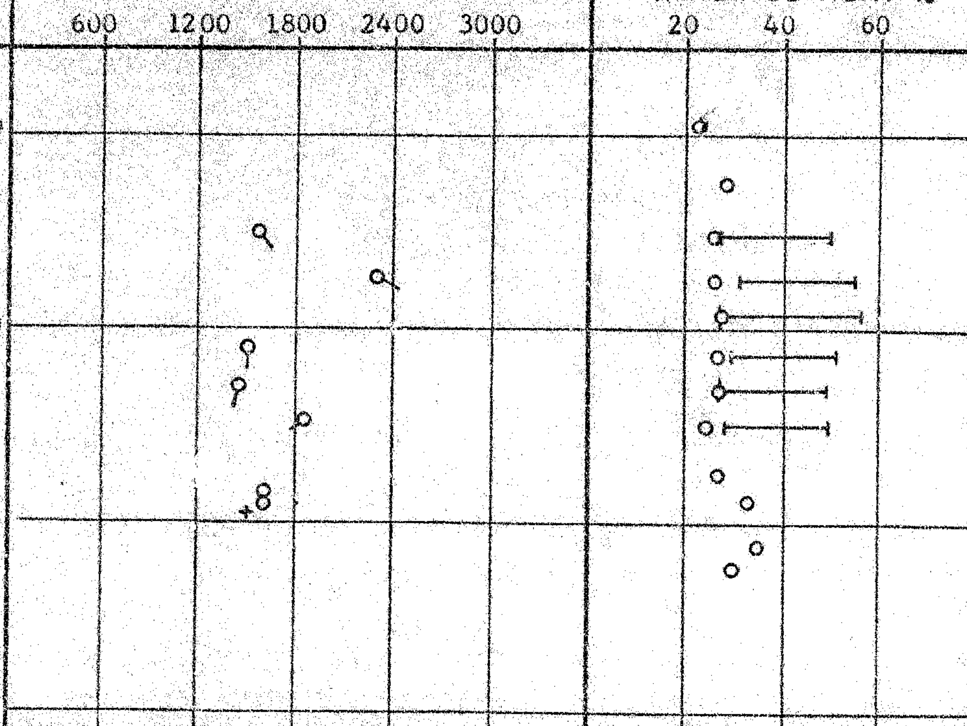
CHECKED BY

ELEV. DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
604.6	Clay to silty clay, trace of sand - fill. Reddish-Brown. Stiff to very stiff.	1	SS	18	600		
		2	SS	15			
		3	TW	PM		125	
		4	TW	PM		124.5	
		5	TW	PM	590	120	
		6	TW	PM		124.5	
		7	TW	PM		124.0	
		8	TW	PM		125	
582.1 / 22.5		9	TW	PM			
580.1	Clayey Topsoil.	10	TW	PM	580		
24.5	Clay to silty clay, trace of sand. Grey-Brown. Stiff to very stiff.	11	TW	PM			El. 579.0
		12	TW	PM			
		13	TW	PM			
		14	TW	PM			
569.1		15	TW	PM	570		
35.5	End of borehole.						

DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT

SHEAR STRENGTH P.S.F.: o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, x LAB. VANE

LIQUID LIMIT — w_L; PLASTIC LIMIT — w_P; WATER CONTENT — w



DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 103 A

FOUNDATION SECTION

JOB 72-11025 LOCATION Forkes Road & C.N.R., Sta. 214+50 18' Lt. Hwy. #140 ORIGINATED BY S.A.

W.P. 60-68-02 BORING DATE July 19, 1971 COMPILED BY W.V.U.

DATUM Geodetic BOREHOLE TYPE NX Casing CHECKED BY [illegible]

ELEV. / DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT; SHEAR STRENGTH P.S.F. (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB. VANE) 600 1200 1800 2400 3000	LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w; WATER CONTENT % 20 40 60	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL
604.6	Clay to silty clay, trace of sand - fill		1	SS	21	600				
	Reddish-brown		2	SS	12					DRY
	Stiff to very stiff.		3	SS	16	590				
582.1			4	SS	15					
22.5	End of borehole.					580				

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 104

FOUNDATION SECTION

JOB 72-11025 LOCATION Forkes Road and C.N.R., Sta. 214+50 86' Lt. Hwy. #140 ORIGINATED BY S.A.

W.P. 60-68-02 BORING DATE July 20, 1971 COMPILED BY W.V.U.

DATUM Geodetic BOREHOLE TYPE NX Casing CHECKED BY

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS/FOOT; SHEAR STRENGTH P.S.F. (o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB. VANE) 600 1200 1800 2400 3000	LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w; WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
582.8	Clay to silty clay, trace of sand. Reddish-brown, Stiff to very stiff.		1	SS	8					W.L. July 20/7
			2	SS	13	580	2000 VVVVVV			
			3	SS	13					
			4	SS	11					
			5	SS	18					
			6	SS	21					
			7	TW	PM					
			8	SS	31					
			9	SS	20	570				
567.8			10	SS	11					
15.0	End of borehole.									

FORM OB-MT-26 (REV. 1971)　　OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS
DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 105

FOUNDATION SECTION

JOB 72-11025　LOCATION Forkes Road and C.N.R., Sta. 219+50 86' Lt. Hwy. 140　ORIGINATED BY S.A.
W.P. 60-68-02　BORING DATE July 20, 1971　COMPILED BY W.V.U.
DATUM Geodetic　BOREHOLE TYPE NX Casing　CHECKED BY OE.

Dynamic penetration resistance, blows/foot. Shear strength P.S.F. (600, 1200, 1800, 2400, 3000): o Unconfined, + Field vane, ● Quick triaxial, × Lab. vane. Liquid limit w_L, plastic limit w_P, water content w; water content % (20, 40, 60).

Elev. / Depth	Description	Strat. plot	Sample number	Type	Blows/foot	Elev. scale	Shear strength	Bulk density γ P.C.F.	Remarks GR.SA.SI.CL
600.0	Clay to silty clay, trace of sand (Fill) Reddish-brown. Stiff to very stiff.		1	SS	15				
			2	SS	12				
			3	TW	PM		2000	125	
			4	TW	PM			125	
			5	SS	12	590			
			6	TW	PM			125	
			7	SS	17				
			8	TW	PM			126	
			9	SS	22				
581.0 / 19.0			10	SS	19				
579.0 / 21.0	Clayey Topsoil.		11	SS	13	580			
	Clay to silty clay, trace of sand. Stiff to hard.		12	SS	23				
			13	TW	PM				W.L. July 20/71
			14	SS	35				
			15	SS	44				
			16	SS	15				
			17	TW	PM	570			
567.5			18	SS	17				
32.5	End of borehole.								

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

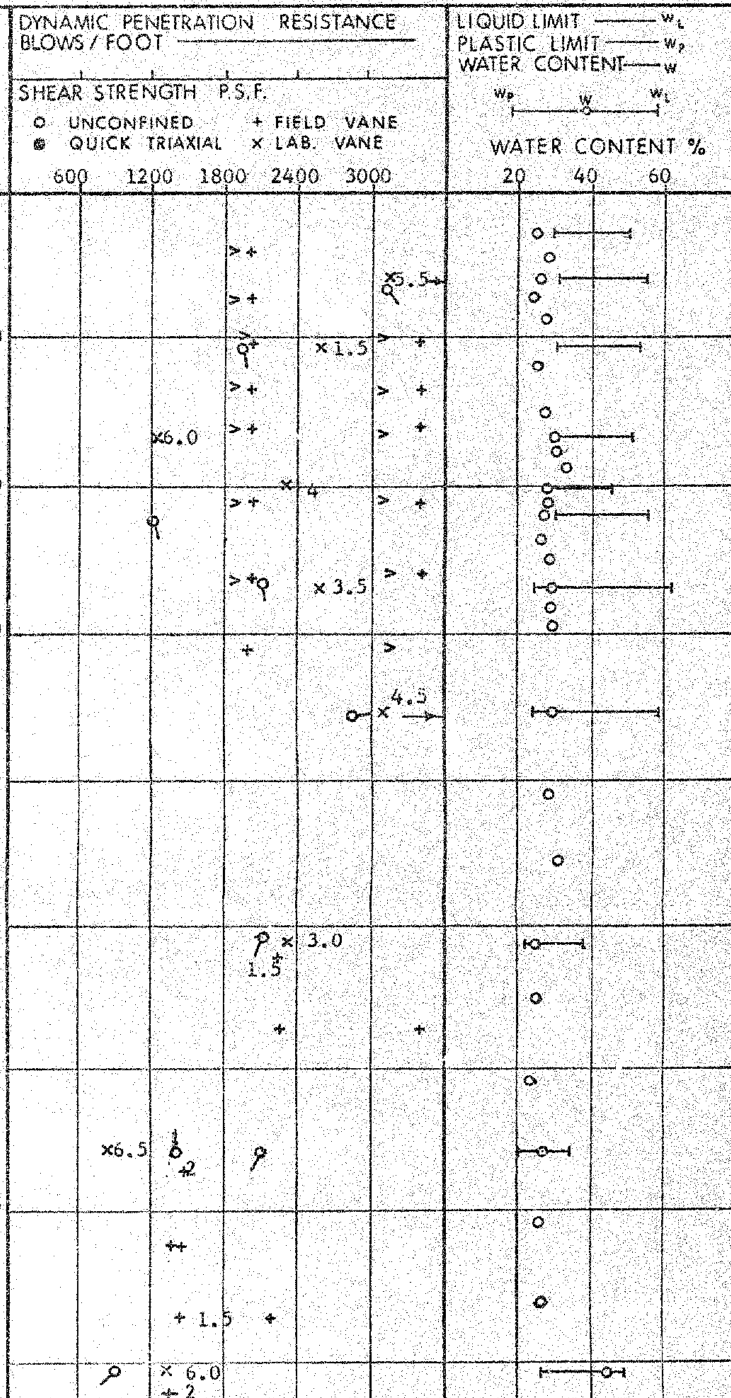
DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 206

FOUNDATION SECTION

JOB	72-11025	LOCATION	Hwy. #140 Sta. 221 + 40 O/S 3' Lt.	ORIGINATED BY	R.R.B.
W.P.	60-68-02	BORING DATE	February 3 and 4, 1972	COMPILED BY	R.R.B.
DATUM	Geodetic	BOREHOLE TYPE	C.M.E. Augering.	CHECKED BY	

ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FOOT	BULK DENSITY γ P.C.F.	REMARKS
609.8 / 0.0	Silty clay to clay, trace of sand (Fill) Greyish-brown. Stiff to very stiff.	1	SS	23		
		2	SS	18		
		3	TW	PH	128.5	
		4	SS	24		
		5	SS	14		
		6	TW	PH	124.0	
		7	SS	35		
		8	SS	14		
		9	TW	PH		
		10	SS	12		
		11	SS	9		
		12	TW	PH		
		13	SS	10		
		14	SS	13		
		15	TW	PH	123.0	
		16	SS	21		
		17	SS	18		
		18	TW	PH	124.5	P. 29'
		19	SS	20		
580.3	Clayey Topsoil.	20	SS	21		DRY
30.5	Silty clay to clay, trace of sand and gravel. (occasional silt and sand layers up to 3" thick) Stiff to very stiff.	21	TW	PH	122	
		22	SS	22		
		23	SS	14		
		24	TW	PH	125	
		25	SS	11		
		26	SS	16		
		27	TW	PH	125	
		28	SS	8		
		29	SS	10		P. 77.6
526.8		30	TW	PM	114	
83.0	End of borehole.					



20 / 15 – 5 / 10 % STRAIN AT FAILURE

FORM OB-MT-126 (REV. 1971)

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS
DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 207

FOUNDATION SECTION

JOB 72-1102.) LOCATION Hwy. #140 Sta. 221 + 46 O/S 60' Lt. ORIGINATED BY R.R.B.

W.P. 60-68-02 BORING DATE February 9, 1972 COMPILED BY R.R.B.

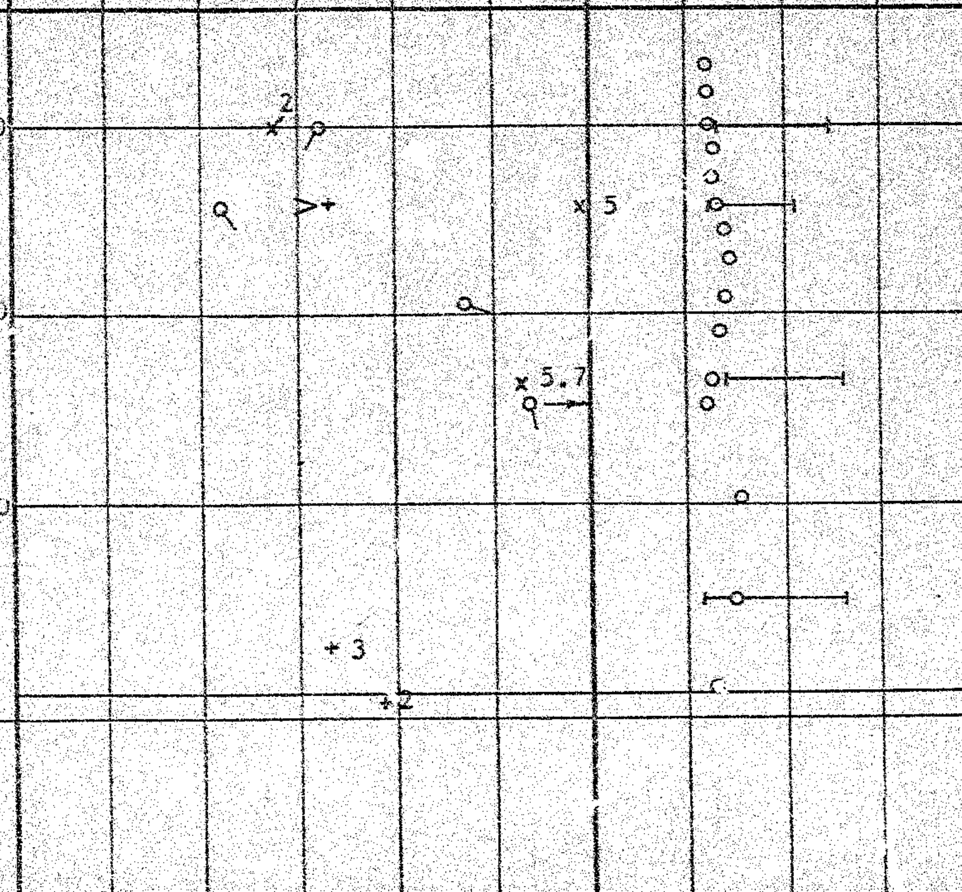
DATUM Geodetic BOREHOLE TYPE C.M.E. Augering CHECKED BY

ELEV. / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL
596.0 / 0.0	Silty clay to clay, trace of sand (Fill)						▽
	Reddish-Brown	1	SS	16			
	Stiff to very stiff.	2	SS	16			
		3	TW	PH	590	126.0	
		4	SS	17			
		5	SS	13			
		6	TW	PH		123.0	
		7	SS	14			
		8	SS	17			
		9	TW	PH		120.0	
579.5		10	SS	13	580		
578.0	Clayey Topsoil.	11	TW	PH			
18.0	Silty clay to clay, trace of sand.	12	TW	PH		126.0	
	(Occasional seams of silt and sand up to 3" thick)	13	SS	36			
	Very stiff.	14	SS	18	570		
		15	TW	PH		123.0	P. 33'
558.5		16	SS	11	560		
37.6	End of borehole.						

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT

SHEAR STRENGTH P.S.F.
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB. VANE
600 1200 1800 2400 3000

LIQUID LIMIT — w_L
PLASTIC LIMIT — w_P
WATER CONTENT — w
WATER CONTENT %
20 40 60



FORM 03-MT-•26 (REV. 1971) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS
DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 208

FOUNDATION SECTION

JOB 72-11025 LOCATION Hwy. #140 Sta. 212 + 00 O/S 15' Lt. ORIGINATED BY R.R.B.

W.P. 60-68-02 BORING DATE February 9, 1972 COMPILED BY R.R.B.

DATUM Geodetic BOREHOLE TYPE C.M.E. Augering & NX Casing Washbore CHECKED BY

ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	SHEAR STRENGTH P.S.F. (+ FIELD VANE)	REMARKS
602.7 / 0.0	Silty clay to clay, trace of sand (Fill) Reddish-Brown Stiff to very stiff.	1	SS	13	600		
		2	SS	11			
		3	SS	9			
		4	TW	PH			
		5	SS	16			
		6	SS	13			
		7	TW	PH			
		8	SS	29	590		
		9	SS	16			
		10	TW	PH			
		11	SS	16			
		12	SS	18			
		13	TW	PH			
		14	SS	22	580		
		15	SS	37			
		16	TW	PH			
575.5		17	SS	24			
574.5	Clayey Topsoil.	18	SS	36			P 31' DRY
28.2	Silty clay to clay, trace of sand (Occasional seams of silt and sand up to 3" thick throughout) (Grey-Brown) Stiff to very stiff.	19	TW	PH			
		20	SS	68	570		
		21	SS	24			
		22	TW	PH	560	>+	
		23	SS	18		+2	
		24	SS	12	550	+2	
		25	TW	PH		+2	
		26	SS	8	540	+2	
		27	SS	13		+4	
		28	TW	PH	530	+3	P 75'
		29	SS	11		>+	
519.7		30	SS	8	520	+3	
83.0	End of borehole.						

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT

SHEAR STRENGTH P.S.F.: O UNCONFINED; ● QUICK TRIAXIAL; + FIELD VANE; × LAB. VANE — 600 1200 1800 2400 3000

LIQUID LIMIT — w_L; PLASTIC LIMIT — w_P; WATER CONTENT — w; WATER CONTENT %

BULK DENSITY γ P.C.F.

REMARKS GR.SA.SI.CL.

20 / 15 - 5 / 10 % STRAIN AT FAILURE

OFFICE REPORT [illegible]N SOIL EXPLORATION

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS
DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 209

FOUNDATION SECTION

JOB 72-11025 LOCATION Hwy. #1[illegible] Sta. 212 + 00 O/S 7[illegible]' Rt. ORIGINATED BY R.R.B.

W.P. 60-68-02 BORING DATE February 14 & 15, 1972 COMPILED BY R.R.B.

DATUM Geodetic BOREHOLE TYPE C.M.E. Augered CHECKED BY [illegible]

ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT; SHEAR STRENGTH P.S.F. (o UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, x LAB. VANE)	LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w — WATER CONTENT %	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
592.5										
0.0	Silty clay to clay (Fill) Reddish-Brown Very stiff to hard.					590				
			1	SS	26					
			2	SS	47					
			3	SS	27					
			4	SS	24					
			5	TW	PH					
			6	SS	52					
			7	SS	13	580				
			8	TW	PH					
578.0	Softened Zone. Firm		9	SS	9					
577.0	Clayey Topsoil.		10	SS	32					
			11	TW	PH					
15.5	Silty clay to clay, trace of sand and gravel. Grey-Brown Very stiff to hard.		12	SS	40					
			13	SS	19	570				
			14	TW	PH					P 29½'
561.0			15	SS	19	560				
31.5	End of borehole.									

FORM OB-MT-■6 (REV. 1971) OFFICE REPOR■■N SOIL EXPLORATION

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS
DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 210

FOUNDATION SECTION

JOB 72-11025 | LOCATION Hwy. #140 Sta. 215 + 00 O/S 9' Lt. | ORIGINATED BY R.R.B.

W.P. 60-68-02 | BORING DATE February 15 and 16, 1972 | COMPILED BY R.R.B.

DATUM Geodetic | BOREHOLE TYPE | CHECKED BY

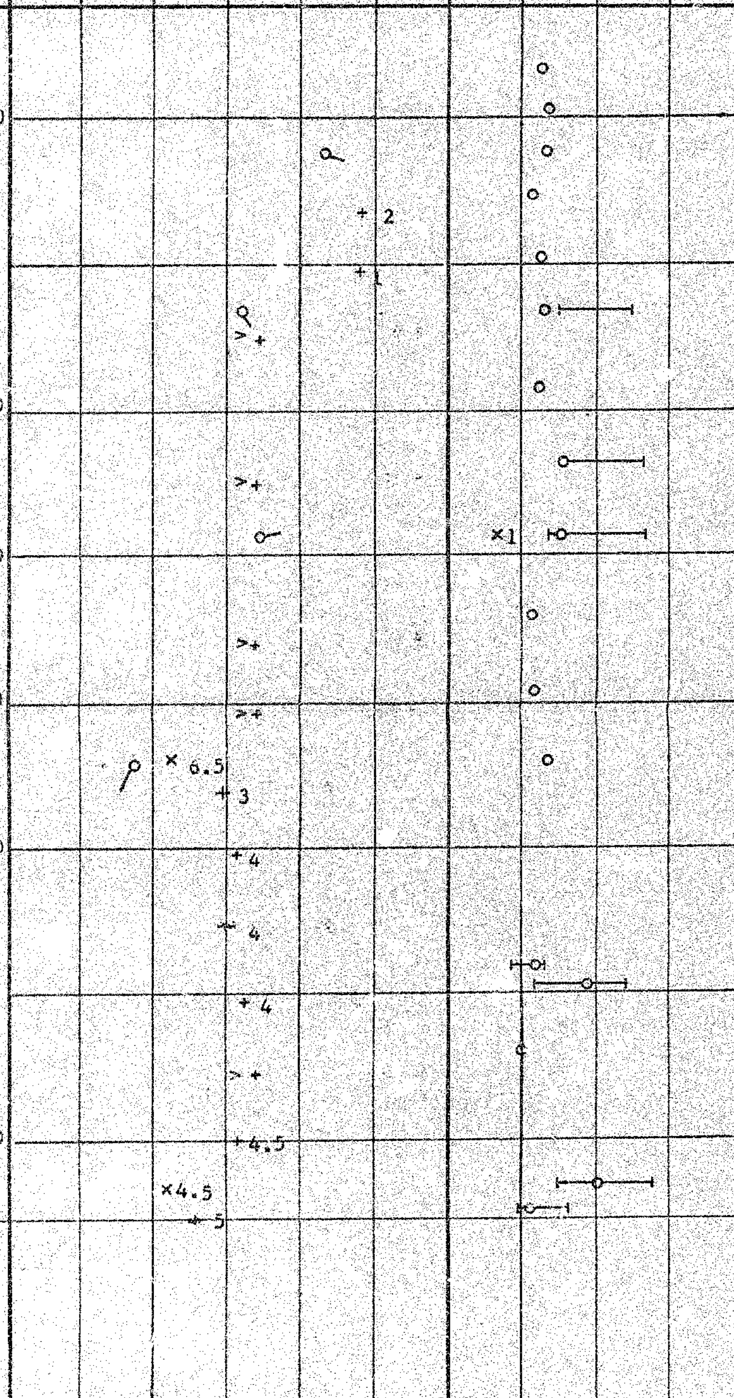
ELEV. DEPTH 607.4	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS GR.SA.SI.CL.
0.0	Silty clay to clay, trace of sand and gravel (Fill)	1	SS	15			
	Reddish-Brown.	2	SS	17	600		
	Stiff to very stiff.	3	TW	PH		125.0	
		4	SS	15			
		5	SS	15	590		
583.9		6	TW	PH		125.0	
582.9	Clayey Topsoil.						P 24'6"
24.5	Silty clay to clay, trace of sand and gravel.	7	SS	31	580		
		8	SS	16			
	(Occasional seams of silt and sand up to 3" thick)	9	TW	PH	570	123.0	
		10	SS	16			
	Stiff to very stiff.	11	SS	14	560		
		12	TW	PH		127	
		13	SS	10	550		
		14	SS	7			
		15	TW	PH	540	126	
		16	SS	20			
		17	SS	8	530		
524.4		18	TW	PH		121	
83.0	End of borehole.						

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT

SHEAR STRENGTH P.S.F.
o UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB. VANE

LIQUID LIMIT — w_L
PLASTIC LIMIT — w_P
WATER CONTENT — w

WATER CONTENT %



20
15 – 5 % STRAIN AT FAILURE
10

FORM OB-MT-?26 (REV. 1971) OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

DESIGN SERVICES BRANCH

RECORD OF BOREHOLE No. 211

FOUNDATION SECTION

JOB 72-11025 LOCATION Hwy. #140 Sta. 213 + 36 O/S 100 Ft. Rt. ORIGINATED BY R.R.B.

W.P. 60-68-02 BORING DATE February 17, 1972 COMPILED BY R.R.B.

DATUM Geodetic BOREHOLE TYPE C.M.E. Auger CHECKED BY

ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FOOT	ELEV. SCALE	BULK DENSITY γ P.C.F.	REMARKS GR. SA. SI. CL.
585.2	Ground level.						
0.0	Silty clay to clay, trace of sand & gravel (Fill)	1	SS				
		2	SS	34	580		
576.7	Very stiff.	3	SS	19			
8.5	Silty clay to clay, trace of sand and gravel.	4	TW	PH			El. 575.0
		5	SS	15			
	Grey-brown.	6	SS	22	570		
	Stiff to very stiff.	7	TW	PH		119	
		8	SS	17	560		
553.6		9	SS	16			
31.5	End of borehole.				550		

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT

SHEAR STRENGTH P.S.F.
o UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × LAB. VANE
600 1200 1800 2400 3000

LIQUID LIMIT — w_L
PLASTIC LIMIT — w_P
WATER CONTENT — w

WATER CONTENT %
20 40 60

STANDARD PROCTOR COMPACTION TEST RESULTS

72 - 11025

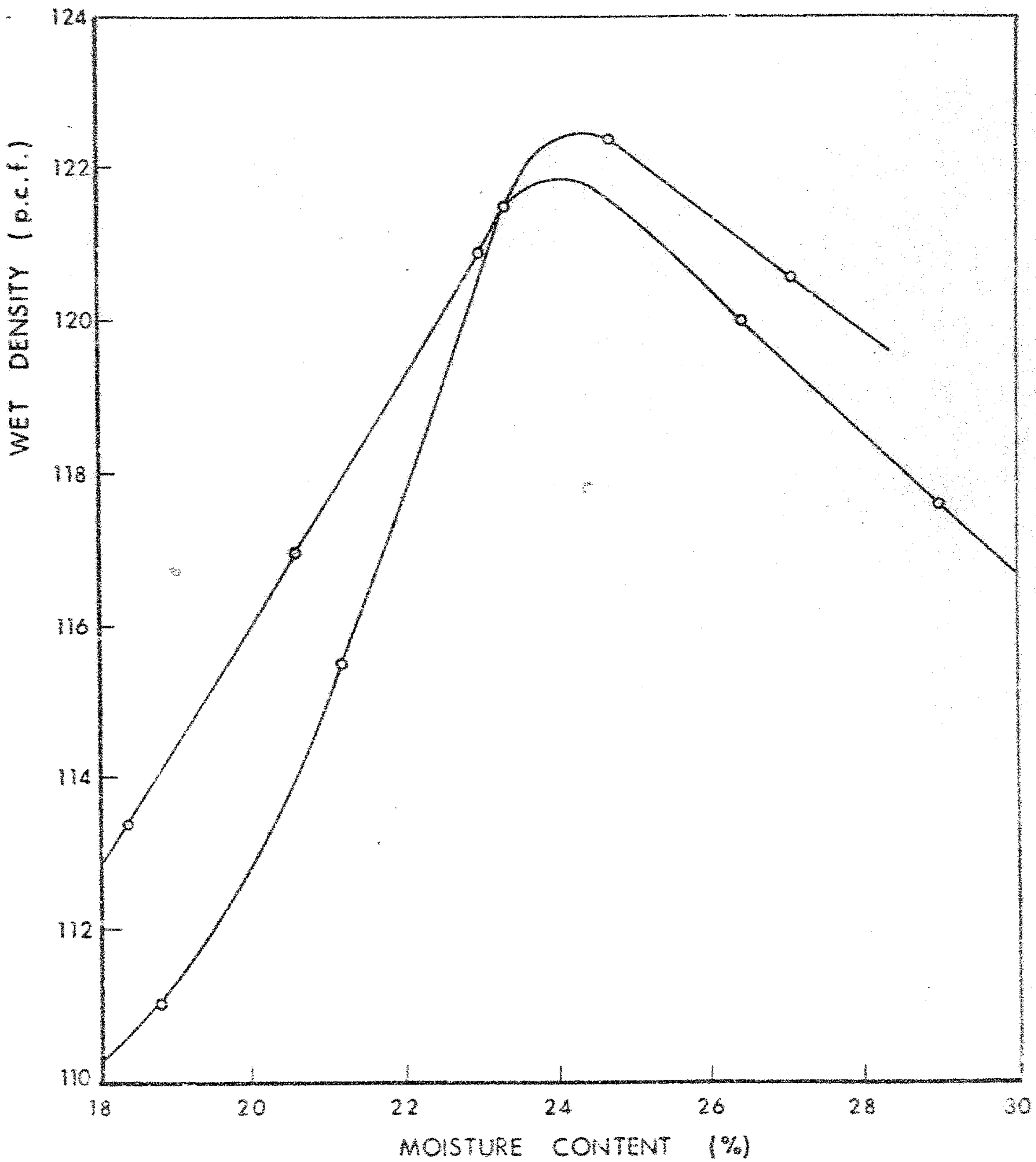
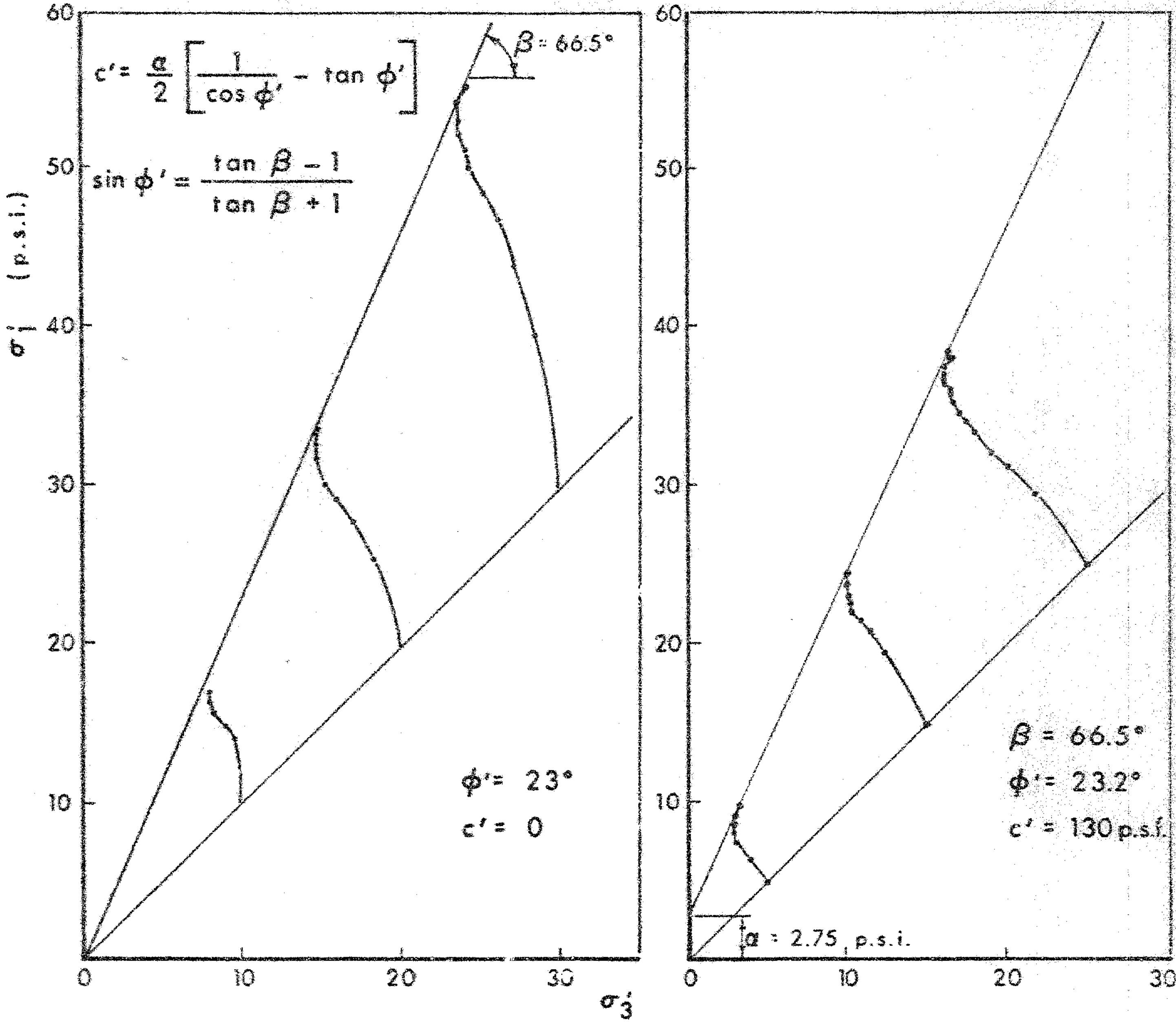


FIG. 1

EFFECTIVE STRESS LABORATORY TESTS

72-11025

FILL



— Isotropically Consolidated Undrained Tests with Pore Pressure Measurements —

PARENT MATERIAL

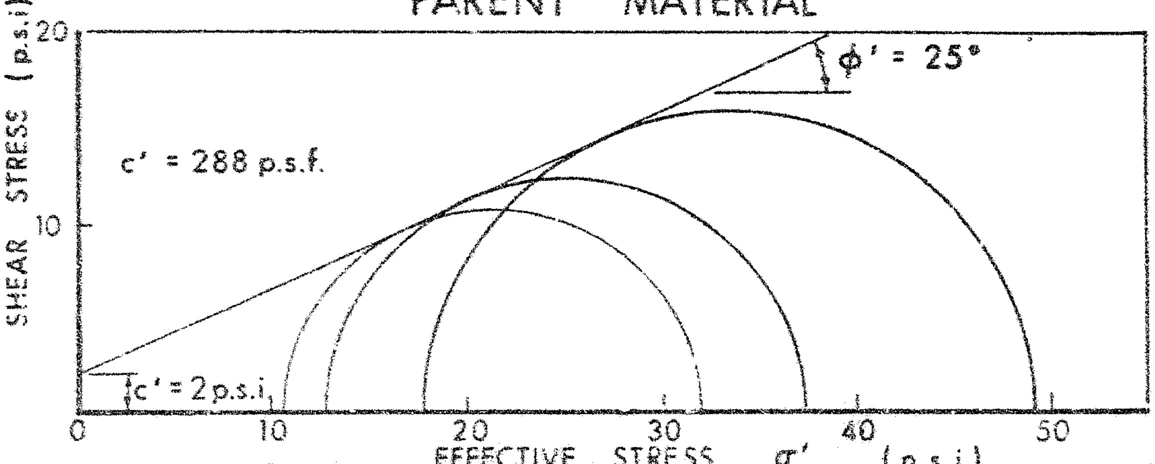


FIG. 2

MOISTURE CONTENT OF FILL MATERIAL

72-11025

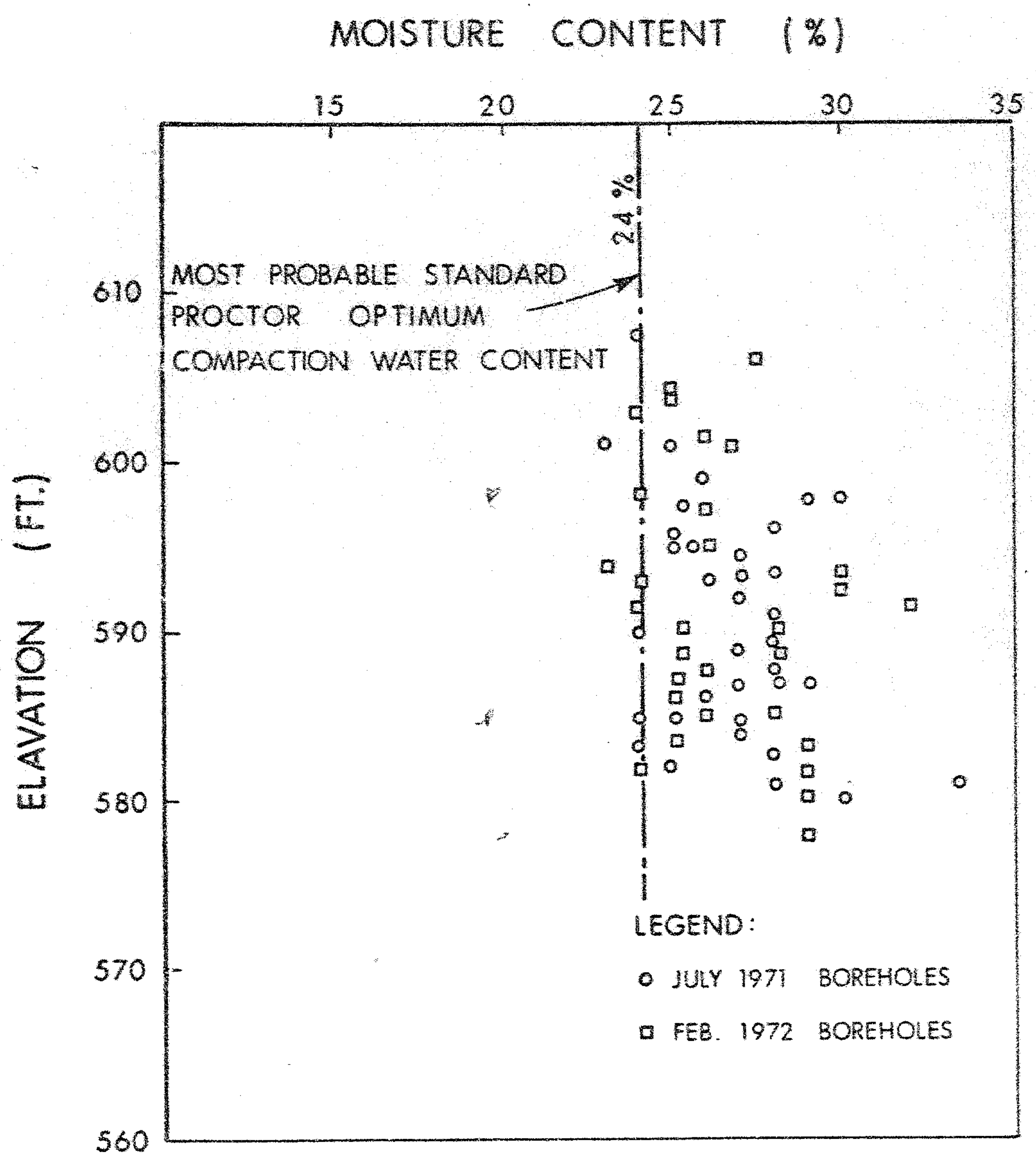
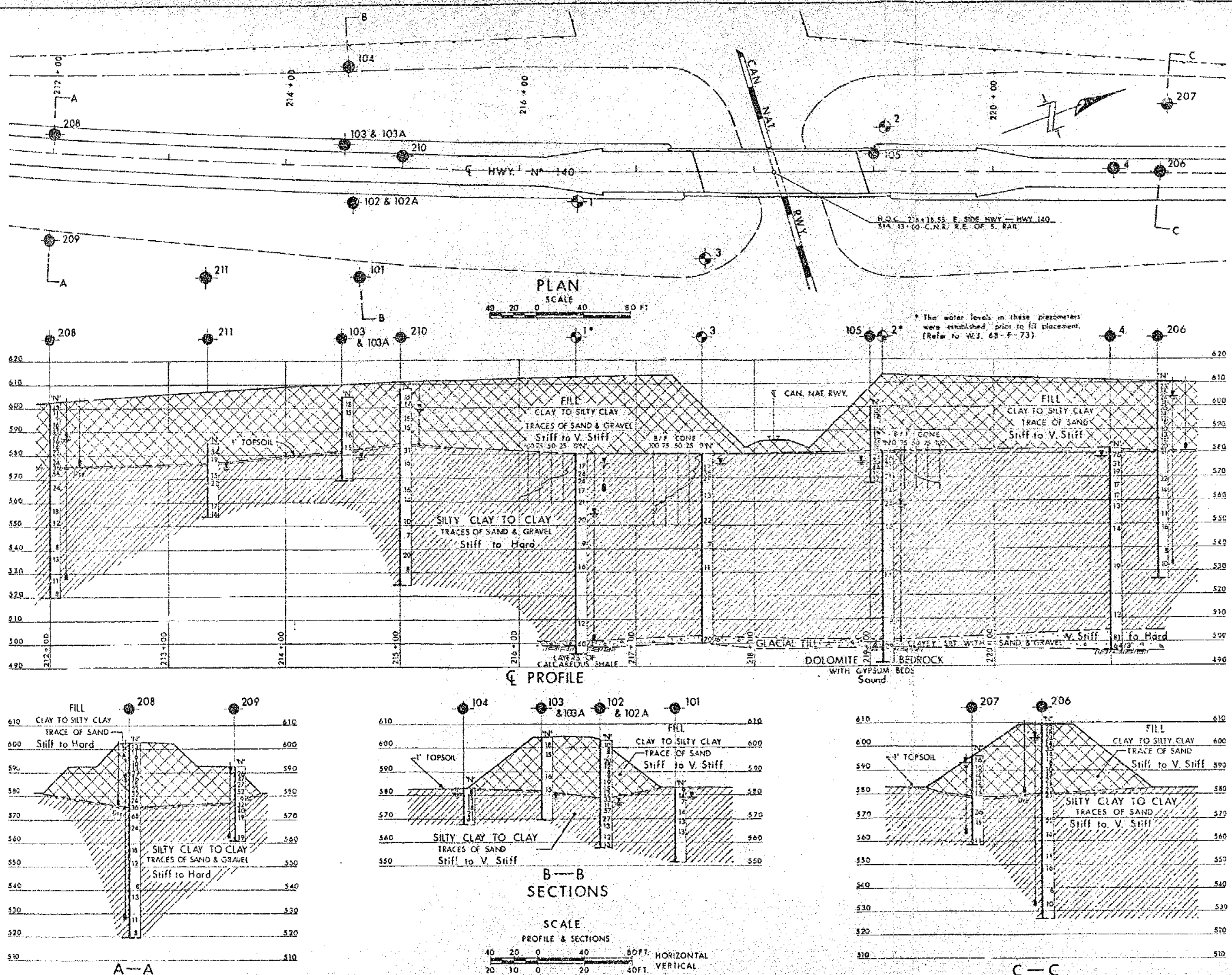
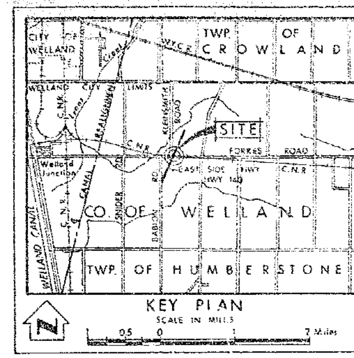


FIG. 3





LEGEND

- Bore Hole
- Cone Penetration Test
- Bore Hole & Cone Test
- Water Levels established at time of field investigation
- Piezometer

NO	ELEVATION	STATION	OFFSET	
1	579.0	216 + 50	25' RT.	OCT. 1968
2	582.0	219 + 10	38' LT	
3	580.5	217 + 53	73' RT	
4	582.0	221 + 00	5' LT.	
101	582.8	214 + 67	94' RT	JULY 1971
102&102A	600.3	214 + 60	32' RT.	
103&103A	604.6	214 + 50	18' LT.	
104	582.8	214 + 50	86' LT.	
105	600.0	219 + 50	85' LT.	
206	609.8	221 + 40	2' LT.	FEB. 1972
207	596.0	221 + 46	60' LT	
208	602.7	212 + 00	15' LT.	
209	592.5	212 + 00	75' RT	
210	607.4	215 + 00	9' LT	
211	585.2	213 + 36	100' RT	

— NOTE —

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

REVISIONS		
DATE	BY	DESCRIPTION

MINISTRY OF TRANSPORTATION & COMMUNICATIONS
DESIGN SERVICES BRANCH — FOUNDATIONS OFFICE

CANADIAN NATIONAL RAILWAYS

NEAR FORKES ROAD

HIGHWAY NO 140 (EAST SIDE HWY.) DIST. NO 4
CO WELLAND
TWP. HUMBERSTONE LOT 16 & 20 CON. V & IV

BORE HOLE LOCATIONS & SOIL STRATA

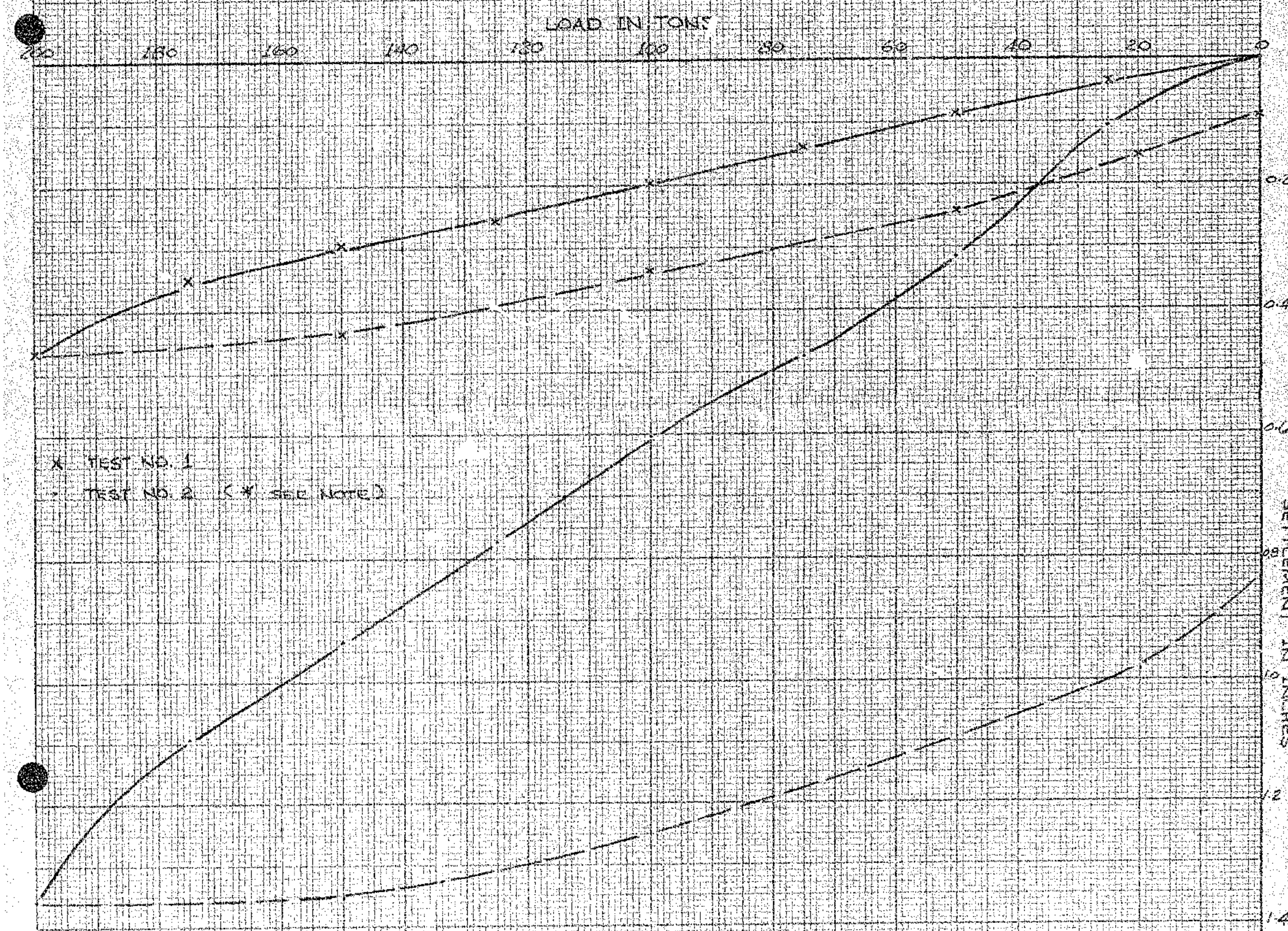
SUBM'D. B.T.D.	CHECKED	W.P. NO. 60-68-02	DRAWING NO 72-11025A
DRAWN O.E.	CHECKED	JOB NO 72-11025	BRIDGE DRAWING NO
DATE JUNE 23, 1972		SITE NO	
APPROVED		CONT. NO.	

PRINCIPAL FOUNDATION ENGINEER

72-11025

DESCRIPTION OF TEST PILES : "HERKULES" PRECAST CONCRETE PILE TYPE 800
(12" flat to flat Hexagonal)

TEST NO	1	2
BEARING STRATUM	driven to bedrock	driven to 10 feet into very dense glacial till
FINAL SET	25 blows / 1/8 inch	25 blows / 1/4 inch
DRIVING ENERGY	10,500 ft-lbs	13,200 ft-lbs
LOCATION & REFERENCE	Netherby Rd overpass (Cont. 1089 - S.L.S.A.)	Wilhelm Rd. overpass (Cont. 1114 - S.L.S.A.)



NOTE: DUE TO THE EXCESSIVE SETTLEMENT OF THE TEST PILE AT DESIGN LOAD, THE ST. LAWRENCE SEAWAY AUTHORITY PERSONNEL CONSIDERED THIS TEST UNSATISFACTORY. AFTER RE-DRIVING THE TEST PILE WITH VARIOUS DRIVING ENERGIES, THE FOLLOWING CRITERIA FOR REFUSAL WERE ESTABLISHED.

i) 42 blows per inch for a minimum of 2 inches penetration using a driving energy of 19,800 ft-lbs (3 feet drop height)

or ii) 62 blows per inch for a minimum of 2 inches penetration using a driving energy of 13,200 ft-lbs (2 feet drop height)

CONT. 70-212

DEPARTMENT OF HIGHWAYS ONTARIO

MEMORANDUM

To: Mr. B. R. Davis,
Bridge Engineer,
Bridge Division,
Admin. Bldg.

Attention: Mr. S. McCombie

From: Foundation Section,
Materials & Testing Office,
Room 107, Lab. Bldg.

Date: December 4, 1968

Our File Ref.

In reply to DEC 11 1968

Subject:

FOUNDATION INVESTIGATION REPORT
For
The Crossing of the C.N.R. Tracks
And Proposed East Side Highway
(Near Forkes Road)
Twp. of Humberstone, Co. of Welland
District No. 4 (Hamilton)
W.J. 68-F-73 -- W.P. 60-68-02

Attached, we are forwarding to you, our detailed foundation investigation report on the subsoil conditions existing at the above structure site.

We believe that the factual data and recommendations contained therein, will prove adequate for your design requirements. Should additional information be required, please do not hesitate to contact our Office.

A. G. Stermac
PRINCIPAL FOUNDATION ENGINEER

AGS/MdeF
Attach.
cc: Messrs. B. R. Davis (2)
H. A. Tregaskes
D. W. Farren
G. K. Hunter (2)
H. Greenland
W. S. Melinyshyn
T. J. Kovich
B. A. Singh
Foundations Files
Gen. Files

TABLE OF CONTENTS

FOUNDATION INVESTIGATION REPORT
For
The Crossing of the C.N.R. Tracks
And Proposed East Side Highway
(Near Forkes Road)
Twp. of Humberstone, Co. of Welland
District No. 4 (Hamilton)
W.J. 68-F-73 -- W.P. 60-68-03

1. INTRODUCTION:

The Foundation Section was requested to carry out a subsurface investigation at the site of the crossing of the C.N.R. tracks and the proposed East Side Highway in the Twp. of Humberstone, Co. of Welland. The request was contained in a memo from the Bridge Office (Mr. F. I. Hewson, Senior Bridge Liaison Engineer), dated September 23, 1968.

Subsequently, a foundation investigation was carried out at the proposed site to determine the subsoil and groundwater conditions.

This report contains the results of the investigation, together with recommendations pertaining to the foundations of the proposed structure, as well as the stability of the approach embankments.

2. DESCRIPTION OF THE SITE AND GEOLOGY:

The site is located some 200 ft. east of the intersection of Forkes Rd. and Kleinsmith Rd., approximately 2 miles east of Welland Junction. At this location the C.N.R. tracks, which run parallel to Forkes Rd., are about 100 ft. to the north. The tracks are elevated about 4 ft. above the surrounding ground level on a 25-ft. wide embankment. Forkes Rd. is a two-lane, paved County road; the profile grade of this road is about 1 to 2 ft. above the surrounding terrain. Shallow ditches run along both sides of Forkes Rd. as well as the C.N.R. embankment.

cont'd. /2 ...

2. DESCRIPTION OF THE SITE AND GEOLOGY: (cont'd.) ...

The surrounding area is generally flat-lying; the surficial drainage is very poor. The land to the north of the site is used for farming purposes, while the land to the south, at the present time, is abandoned.

Physiographically, the site is situated in the region known as the Haldimand Clay Plain. In this area the subsoil consists of extensive, mainly glacial-lacustrine deposits, laid down in glacial Lake Warren during the Wisconsin age. These deposits are composed of stratified silts and clays, and are generally underlain by a basal [illegible]cial till sheet, which in turn, is followed by dolomitic limestone or shale bedrock. The bedrock is of the Salina formation of the Silurian period.

3. FIELD AND LABORATORY WORK:

A total of four sampled boreholes, three of which were accompanied by dynamic cone penetration tests, were carried out. B.H.'s #1, 2 and 3 were advanced to bedrock using a Penn drill employing power auger techniques. In B.H.'s #1 and 2, a diamond drill rig was set up over the pre-augered hole and bedrock was proven by BX size rock core samples. B.H. #4 was put down by the diamond drill rig, which was adapted for soil sampling purposes.

Samples, of the cohesive portion of the overburden, were recovered at required depths, where possible, in 2" and 3" I.D. Shelby tubes, which were pushed either manually or hydraulically into the soil. Elsewhere, samples were obtained in a 2" O.D. split-spoon sampler, which was hammered into the soil in accordance with the specifications for the Standard Penetration Test. The same method was used to advance the dynamic cone penetration tests. Field vane tests were carried out to determine the undrained shear strength of the cohesive stratum.

The groundwater level conditions across the site were determined by installing sealed piezometers in two of the boreholes.

cont'd. /3 ...

3. <u>FIELD AND LABORATORY WORK</u>: (cont'd.) ...

This information was supplemented by recording the groundwater level in the open holes at the remaining boring locations.

The locations and elevations of all the borings were surveyed in the field by personnel from the Central Region Engineering Surveys Section. This information is shown on Dwg. 68-F-73A, together with the estimated stratigraphical profile across the site. All elevations are referred to a Geodetic datum.

All samples were visually examined and identified in the field and subsequently in the laboratory. Following this inspection, laboratory tests were carried out on selected representative samples to determine the physical properties of the subsoil, namely:

Bulk Densities
Natural Moisture Contents
Atterberg Limits
Grain-Size Distributions
Undrained Shear Strengths
Consolidation Characteristics

On completion of these tests, the various soil samples were classified as to type and consistency in accordance with the Unified Soil Classification System (Oct. 1963).

The results of the laboratory testing are plotted on the Record of Borelog sheets and summarized in Appendix I of this report.

4. <u>SUBSOIL CONDITIONS</u>:

4.1) <u>General</u>:

The predominant stratum across the site is composed of a firm to hard silty clay to clay with traces of sand and gravel; this deposit is about 73 to 82 feet thick. This stratum is underlain by a 1 to 7 ft. thick deposit of hard (or very dense) glacial

cont'd. /4 ...

4. SUBSOIL CONDITIONS: (cont'd.) ...

4.1) General: (cont'd.) ...

till, followed by dolomite bedrock.

The boundaries between the various deposits, as determined at the boring locations, are shown on the accompanying borehole sheets.

The stratigraphical profile across the site, inferred from this data, is shown on Drawing 68-F-73A.

From ground surface downwards, the various soil types encountered are described as follows:

4.2) Silty Clay to Clay with Traces of Sand and Gravel:

In general, the surficial cover across the site is composed of up to 6 inches of clayey topsoil. In B.H. #3, put down on the edge of Forkes Rd., a 4.5 ft. surficial layer of a very stiff clayey silt roadway fill was encountered.

Immediately below the fill or topsoil is the predominant overburden stratum across the site, composed of a reddish-brown (with occasional grey seams) silty clay to clay with traces of sand. Gravel sizes were scattered at random throughout the deposit. The overall thickness of the silty clay to clay stratum ranges from 73 to 82 feet. The upper 15 to 20 feet of the stratum is characteristically brown in colour; it is considered that this zone has been desiccated. In all the borings occasional grey silt partings and seams, varying from a fraction of an inch to up to 3 inches in thickness, were encountered throughout. Random pockets of gypsum crystals were observed in many of these seams above elevation 554. In B.H. #2 thin sand seams up to 1/4" thick were observed below elev. 510. Grain-size distribution curves, carried out on samples of the stratum, are appended to this report.

cont'd. /5 ...

4. SUBSOIL CONDITIONS: (cont'd.) ...

4.2) Silty Clay to Clay with Traces of Sand and Gravel: (cont'd.) ...

The physical properties of the overall stratum, as determined by field and laboratory testing, are summarized on Figure 1; a brief resumé follows:

	Desiccated Crust Range (Average)	Lower Zone Range (Average)
Liquid Limit (W_L) (%)	39 - 64 (51)	32 - 64 (49)
Plastic Limit (W_P) (%)	21 - 29 (24)	15 - 30 (23)
Natural Moisture Content (W) (%)	25 - 33 (27)	24 - 49 (37)
Bulk Density (γ) (p.c.f.)	114 - 127 (122)	108 - 127 (120)
Initial Void Ratio (e_o)	-	0.68 - 1.09
Compression Index (C_c)	-	0.28 - 0.50
Undrained Shear Strength (C_u) (p.s.f.)		
i) Field Vanes	>2,000	1,300 - >2,000
ii) Lab. Vanes	1,200 - 1,700	1,100 - 1,700
iii) Lab. Testing	1,500 - 2,650	900 - 1,600
Sensitivity	2 - 7	2 - 5
'N' Values (Blows/ft.)	13 - 76	7 - 19

The Atterberg Limit tests, summarized above, are also plotted on the Plasticity Chart, Fig. #2. These results indicate that the stratum is inorganic and of intermediate to high plasticity. The liquidity index of the upper desiccated zone is typically between 0.1 to 0.3, while the index of the remaining portion of the stratum is generally between 0.2 to 0.6.

cont'd. /6 ...

4. SUBSOIL CONDITIONS: (cont'd.) ...

4.2) Silty Clay to Clay with Traces of Sand and Gravel: (cont'd.) ...

The consistency of the overall stratum, as determined from the undrained shear strength testing, varies from hard to very stiff in the upper 20 to 25 feet (desiccated zone), decreasing to very stiff to stiff with depth.

The consolidation characteristics of the stratum were determined by carrying out three laboratory consolidation tests, the results of which are shown as Void Ratio vs. Pressure Plots, on Figure #5. The results of this testing indicate that the main body of the clay is preconsolidated by about 2 to 4 t.s.f. in excess of the existing overburden pressure. It is estimated that the upper 20 to 25 feet of the stratum (desiccated crust) is preconsolidated by something in excess of 5 t.s.f.

4.3) Clayey Silt with Sand and Gravel (Glacial Till):

This heterogeneous, but generally cohesive deposit, was encountered immediately below the silty clay to clay stratum between elevations 504 and 500. The thickness of the glacial till varies from 1 to 6 ft. In general, the deposit is composed of a brown to grey clayey silt with sand and gravel. In B.H. #1, however, the deposit is basically non-cohesive, being composed of a sand with gravel and fragments of bedrock. In B.H. #2, a layer (6") of white gypsum was encountered just above the bedrock. A grain-size distribution curve carried out on a representative sample of the deposit is shown on Figure #4 in the Appendix of this report.

The Atterberg Limit tests, carried out on representative samples of the glacial till, are shown on the Plasticity Chart, Figure #2. These results indicate that the liquid limit and plastic limit are, on the average, about 18% and 11%, respectively. The corresponding natural water content is generally at or below the plastic limit. Based on these results, it is estimated that the matrix of the glacial till is of low plasticity.

cont'd. /7 ...

4. SUBSOIL CONDITIONS: (cont'd.) ...

4.3) Clayey Silt with Sand and Gravel (Glacial Till): (cont'd.) ...

The standard penetration resistance or 'N' values vary from 29 to well over 100 blows per foot, indicating that the consistency of the cohesive deposit is very stiff to hard.

4.4) Dolomite Bedrock:

Bedrock was established in B.H.'s #1 and 2 by obtaining 5 ft. of BX size rock core. In the other borings bedrock was inferred to exist at the location where the split-spoon sampler met practical refusal. The depth at which bedrock was encountered ranged from elevation 497 to 500 - i.e., some 79 to 85 ft. below existing ground surface.

The bedrock is generally composed of a grey dolomite with numerous gypsum lenses throughout. In B.H. #1 the bedrock was interbedded with dark grey calcareous shale, while in B.H. #2 a white gypsum bed, some 2 ft. thick, was encountered just below the bedrock surface. The bedrock is generally sound; occasional horizontal fractures are present, however, in the upper few feet.

5. GROUNDWATER CONDITIONS:

Groundwater level observations have been carried out during the period of the investigation in i) sealed piezometers installed in boreholes #1 and 2, and ii) the open holes at the remaining boring locations. These observations are recorded on the Borelog sheets and summarized on Drawings 68-F-73A. The results of the measurements indicate that, at the time of the investigation, the piezometric groundwater level within the glacial till deposit ranged from elevation 554 to 558 - i.e., some 25 feet below ground level. The groundwater level within the overlying silty clay to clay stratum ranged from elevation 576 to 579 - i.e., some 3 to 5 feet below ground level.

Previous subsurface investigations in the immediate vicinity indicated that an artesian pressure existed in the lower

cont'd. /8 ...

5. GROUNDWATER CONDITIONS: (cont'd.) ...

glacial till deposit and the upper fractured zone of the bedrock. As discussed in the foregoing paragraph, this is not now the case at this site, since the piezometric groundwater level in the basal till deposit is at a much lower level than that in the overlying cohesive stratum. This change in condition is probably caused by the excavation for the realigned Welland Canal presently underway at a location some 2 miles to the west. At this excavation site, a dewatering scheme is being employed to lower the piezometric groundwater level within the confined aquifer composed of the glacial till and upper zone of the bedrock. The effects of the dewatering in the vicinity were observed at three farm wells, located adjacent to the site; these wells extend into the glacial till. Prior to dewatering, the water level in the wells was at or slightly below ground surface; however, once the dewatering was put into effect it was lowered below the intake elevation of the pump.

6. DISCUSSION AND RECOMMENDATIONS:

6.1) General:

It is proposed to construct an overhead structure to carry the proposed East Side Hwy. over the Canadian National Railway's track. Tentative proposals call for a three-span (40'-40'-40') structure with approach fills having a maximum height of about 30 ft. above surrounding ground level.

Subsoil at the site consists generally of an extensive stratum of silty clay to clay with traces of sand and gravel, followed by a relatively thin glacial till deposit, composed primarily of clayey silt with sand and gravel. The overburden is underlain, at a depth of 79 to 85 feet below existing ground surface, by sound dolomite bedrock.

cont'd. /9 ...

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

6.2) Structure Foundations:

6.2.1) Pier Foundations:

The upper very stiff to hard zone of the cohesive stratum (desiccated crust) is a competent foundation subsoil. Based on this, it is recommended that the piers be founded on spread footings founded in this upper zone. A minimum soil cover of 4 feet should be provided to satisfy the frost protection requirements in the area. Spread footings so founded, may be designed for a safe bearing pressure of 2.5 t.s.f. Settlement of the foundation subsoil at and below the footing level will occur due to the induced footing pressure and to a small extent, by the adjacent embankment loading; the magnitude of these settlements is discussed in Sub-section 6.3.

No major dewatering problems are anticipated during construction of the footings, in view of the relatively impermeable nature of the subsoil. Care should be taken to prevent softening of the subsoil at the footing levels due to minor groundwater seepage or surface run-off. In this regard it is recommended that the foundation base be protected by pouring a mat of lean concrete as soon as subgrade level is reached.

6.2.2) Abutment Foundations:

The abutments of the proposed structure could be constructed, within the approach fills, on spread footings. The fill material, below the spread footings, should consist of well compacted G.B.C. 'A' material and should extend for a horizontal distance of at least 10 feet from the footing edges in the plane of the footing tops. This portion of the fill should be constructed with side slopes of 2:1. The remainder of the fill should be completed to about profile grade for at least a distance of 50 feet behind the abutments before re-excavating for the abutment footings. A design load of 2.0 t.s.f. may be used for footing design.

cont'd. /10 ...

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

6.2) Structure Foundations: (cont'd.) ...

6.2.2) Abutment Foundations: (cont'd.) ...

As an alternative, the proposed abutments may be constructed within the approach fills and supported on 12-3/4 inch O.D. x 1/4 inch wall thickness closed-end steel tube piles driven about 10 feet into the upper very stiff to hard portion of the cohesive deposit; in no case should the final pile tip elevation be lower than 565. The piles can be designed for a safe capacity of 20 tons/pile.

Care should be taken to ensure that rock or bouldery fill is not placed within the areas in which piles have to be driven.

The differential settlement between the various structure elements is the governing factor in determining the type of structure to be employed at this location. The sub-section to follow will discuss this aspect in detail.

6.3) Settlement Considerations:

Computations, based on Schmertmann's* method, have been carried out to determine the consolidation settlement of the foundation subsoil due to the pier footing and the embankment loading.

Results of the analysis are summarized in tabular form below:

1) Ultimate settlements at the pier locations -

Induced by the footing pressure of 2.5 t.s.f. (footing size 50' x 8')	≈ 2-1/2"
Induced by embankment loading (30-ft. height)	≈ 1/2"
Total ---	≈ 3"

*Schmertmann, J. H. -
"The Undisturbed Consolidation Behaviour of Clay" -
American Society of Civil Engineers" - 1955.

cont'd. /11 ...

6. DISCUSSION AND RECOMMENDATIONS: (cont'd.) ...

6.3) Settlement Considerations: (cont'd.) ...

2) Ultimate settlement at the abutment locations ≈ 10"
(Induced by embankment)

These values represent the total consolidation settlements, the majority of which should occur within 1 to 1-1/2 years with about 50% occurring within 4 to 6 months.

Because of the predicted relatively rapid rate of settlement, it would be extremely advantageous to construct the approach fills prior to construction of the structure footings, in order to reduce the differential settlements between the various elements. In this regard it is recommended that, if scheduling allows, the fills be constructed and remain in place for a period of between 4 and 6 months. For example, it is estimated that if a staging period of 6 months is allowed, the differential settlement between the abutments and piers would be of the order of 2 to 2-1/2 inches.

6.4) Approach Embankments:

The approach embankments will have a maximum height of about 30 feet. For fills of this magnitude, no stability problems are anticipated, provided standard 2:1 slopes are adopted. The maximum computed consolidation settlement of the embankment will be of the order of 10 inches. The time rate of settlement will be similar to that discussed previously.

7. SUMMARY:

A foundation investigation for the proposed overhead structure at the crossing of the C.N.R. tracks and East Side Highway is reported.

Subsoil at the site consists of a deposit of hard to stiff silty clay to clay some [illegible] to 82 ft. thick, followed by a

cont'd. /12 ...

7. SUMMARY: (cont'd.) ...

thin, competent glacial till deposit. The glacial till is, in turn, underlain by dolomite bedrock, the surface of which is some 79 to 84 ft. below ground surface.

Pier foundations for the structure should be supported on spread footings located at least 4 ft. below the ground surface; a safe bearing pressure of 2.5 t.s.f. can be applied.

The abutments can be founded within the approach fill either: i) within a zone composed of properly compacted granular fill using an allowable bearing pressure of 2.0 t.s.f., or ii) on 12-3/4" O.D. closed-end pipe piles driven about 10 feet into the hard to very stiff silty clay; the allowable load per pile will be about 20 tons.

The anticipated settlement of the structure foundations and approach fills are discussed in the Section, "Discussion and Recommendations". In order to reduce the magnitude of the differential settlement between the abutments and piers, it would be advantageous to construct the approach fills prior to construction of the structure foundations, as discussed in the report

No major dewatering problems are anticipated for the pier footing excavations.

No stability problems are anticipated for the approach fills, provided 2:1 slopes are employed.

8. MISCELLANEOUS:

The field work, performed during October 17 to November 1, 1968, was supervised by Mr. W. Hutton, Project Foundation Engineer, who also prepared this report.

The investigation was carried out under the supervision of Mr. M. Devata, Supervising Foundation Engineer, who reviewed the report.

Equipment used was owned and operated by F. E. Johnston Drilling Co. Ltd.

December 1968.

APPENDIX I.

FORM 00 MT-126
66-4546

OFFICE REPORT ON SOIL EXPLORATION

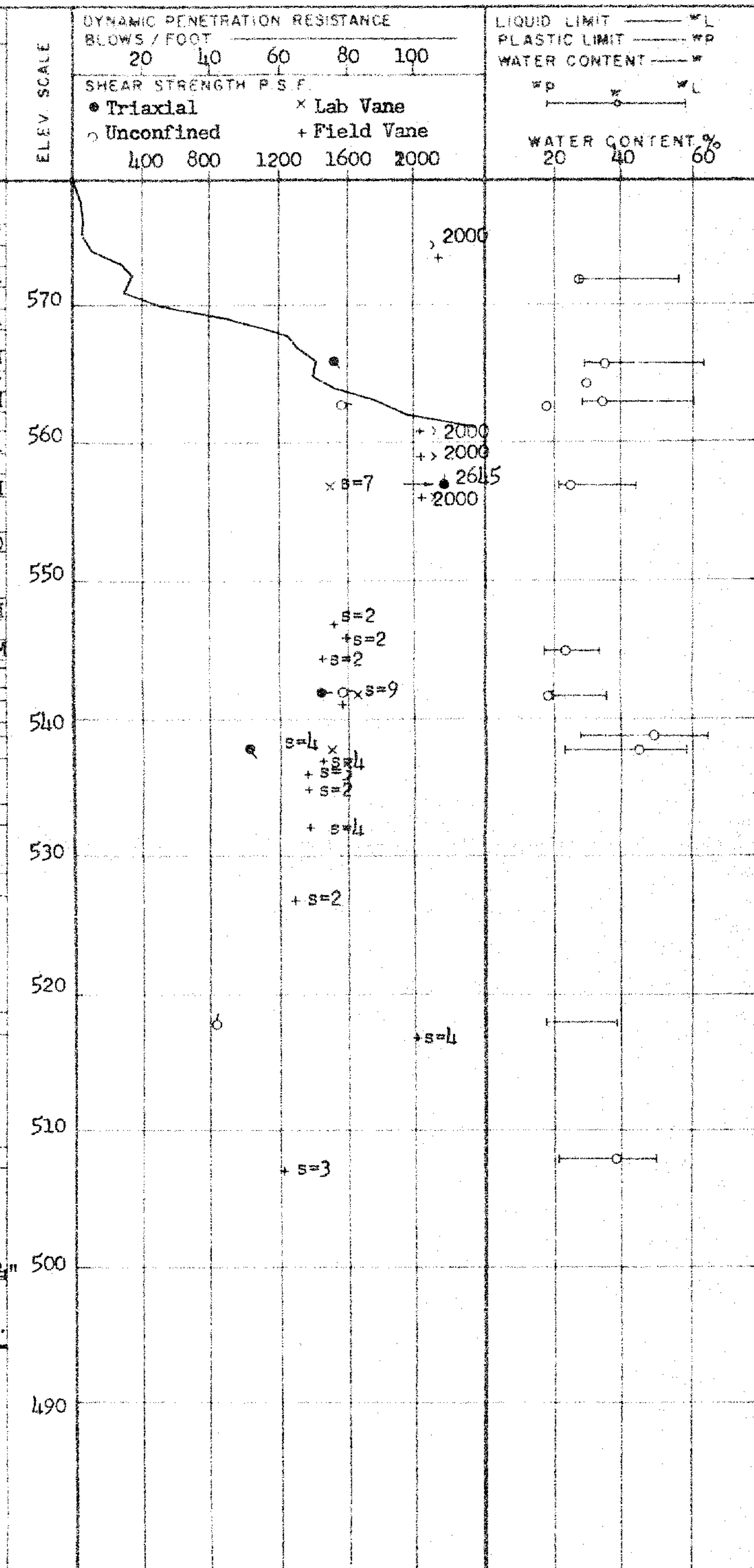
DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

JOB 68-F-73 LOCATION Sta. 216+50 ℄ East Side Hwy. o/s 25' Rt. ORIGINATED BY WH
W.P. 60-68-03 BORING DATE Oct. 17 - Nov. 1, 1968 COMPILED BY WH
DATUM Geodetic BOREHOLE TYPE Cont. Flight auger & diamond drill CHECKED BY

ELEV. DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	BLOWS / FOOT	BULK DENSITY γ P.C.F.	REMARKS Gr.Sa.Si.Cl
579.0	Ground Level						
0.0	Silty clay to clay with trace of sand & gravel with occ. silt layers up to 3" thick throughout. Pocket of silty sand with gravel 3" thick at elev. 539		1A	SS	17		576.
			2	SS	24		0 1 29 70
			3	SS	24		
			4	TW	PH	116	
			5	SS	17		0 1 33 66
			6	3"TW	PH	114	Piez.#2 Tip.elev.564
			7	SS	21		
			8	TW	PH	127.5	
			9	SS	20		554.
			10	TW	PM		
546.0 / 33.0	Clayey silt zone		10A	TW	PM		
			11	SS	9		
			11A	TW	PM	127	
540.0 / 39.0	Brown to reddish brown. Stiff to very stiff.		12A	TW	PM		
			12	TW	PM	111	0 1 22 77
			13	SS	16		
			14	TW	PM		
			15	TW	PH	121	
			16	SS	12		
500.5 / 78.5	(Glacial Till) Very dense Sand with gravel & fragments of bedrock.		17	SS	40/4"		Piez.#1 Tip Elev.499.
79.3	Dolomite bedrock with gypsum lenses. Occ. layers of calcareous shale. Sound.		18	BXL RC	80% Rec.		
494.6 / 84.4	End of Borehole						



DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT

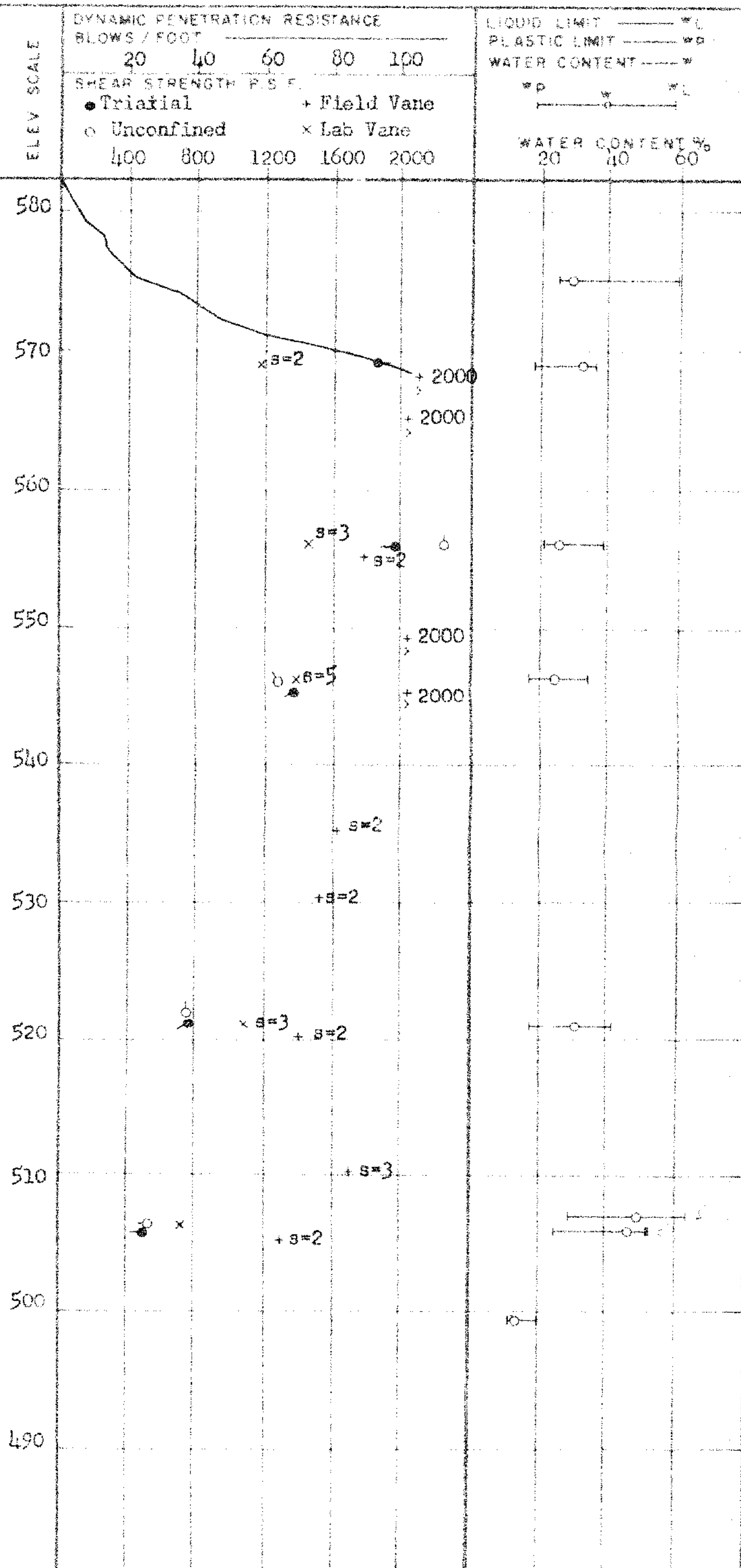
DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 2

FOUNDATION SECTION

JOB 68-F-73 LOCATION Sta. 219+10 ℄ East Side Hwy. o/s 38' Lt. ORIGINATED BY WH
W.P. 60-68-03 BORING DATE Oct. 23-29, 1968 COMPILED BY WH
DATUM Geodetic BOREHOLE TYPE Cont. flight auger & diamond drill CHECKED BY

ELEV. / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FOOT	BULK DENSITY P.C.F.	REMARKS Gr.Sa.Si.Cl.
582.0	Ground Level					
0.0						
		1	SS	26		
	Clayey silt to clay with trace of sand & gravel	2	SS	21		0 1 (99)
		3	SS	21		
	Occ. very thin grey silt seams containing clear gypsum crystals above elev. 559.	4	TW	PH	116	
		5	SS	14		
		6	TW	PH		
		7	SS	24		
		8	TW	PH	128	558
		9A	SS	10		
		10	TW	PH	125	
		11	TW	PH		
		12	SS	12		
	Brown to reddish brown					
		13	TW	PH	121	
	Firm to very stiff					
510.0		14	SS	2		
72.0	Occ. thin sand seams up to ½" thick	15	TW	PH	112 108	
500.0		16	TW	PH		Piez. #3 Tip. Elev. 500'
82.0	Glac. Till-clayey silt with sand, gravel and gypsum. Very stiff.					
497.5		17	SS	29		12 34 39 15
84.5	Dolomite bedrock with a bed of gypsum 2' thick Sound. Grey.	18	BXL RC	76% Rec		
493.0						
89.0	End of Borehole					



DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT

FORM ... OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 3

FOUNDATION SECTION

JOB 68-F-73 LOCATION Sta. 217+53 Ø East Side Hwy. o/s 73' Rt. ORIGINATED BY WH

W P 60-68-03 BORING DATE Oct. 28-29, 1968 COMPILED BY WH

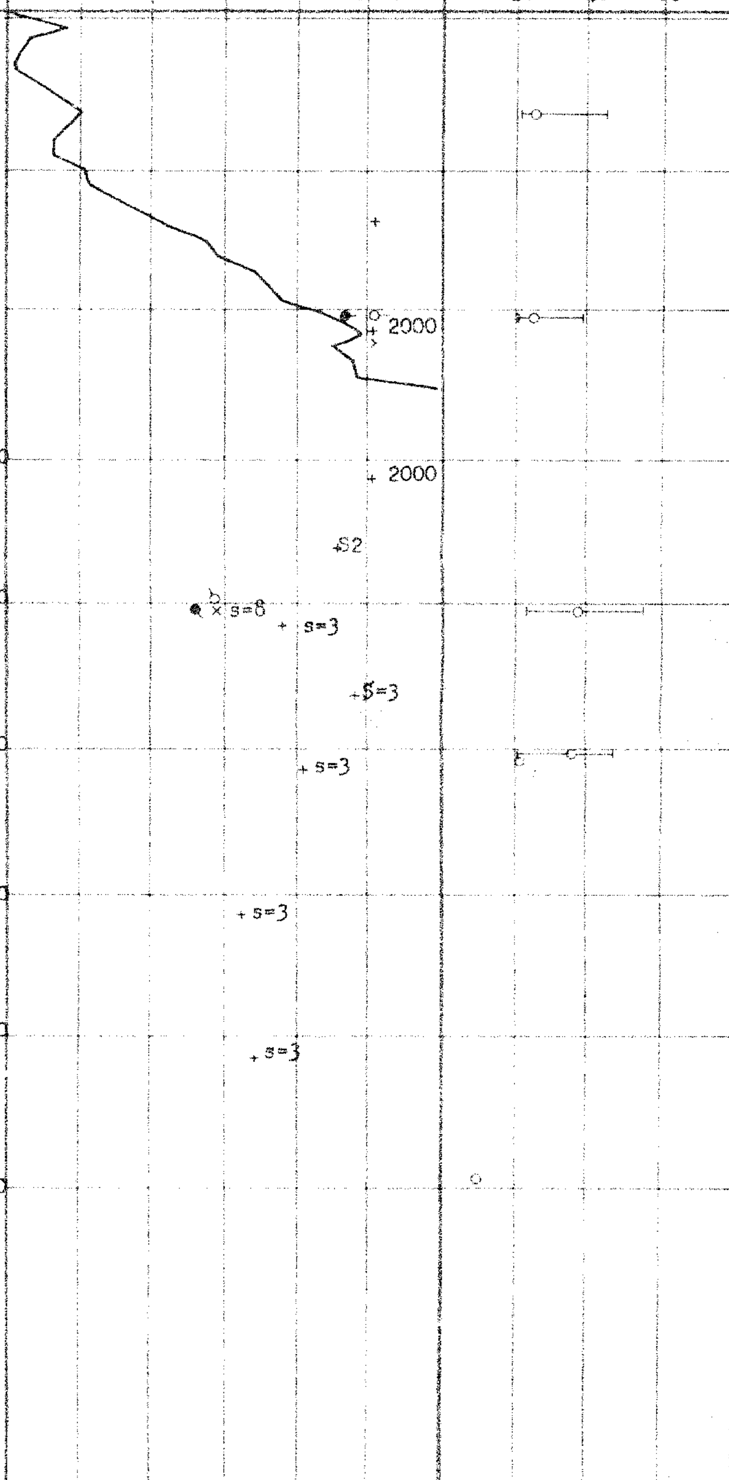
DATUM Geodetic BOREHOLE TYPE Cont. Flight Auger CHECKED BY

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS/FOOT	ELEV SCALE	BULK DENSITY PCF	REMARKS Gr.Sa.Si.Cl
580.5	Ground Level				580		
0.0	Roadway Fill Clayey silt with some sand & gravel. V.Stiff. dark grey.						
576.0		1	SS	17			▽ 575.5
4.5	Silty clay to clay with trace of sand occ. very thin grey silt seams containing clear gypsum crystals above elev. 554.	2	SS	36			
		3	SS	27	570		
		4	TW	PH			
		5	SS	13			
		6	TW	PH	560	127	0 1 54 45
		7	SS	22			
		8	3" TW	PH	550		
		9	SS	7			
		10	3"TW	PH	540	125 111	
535.0 45.5	Occasional silt layers up to 3" thick.	11	SS	11			
		12	3"TW	PH	530		0 0 90 10 (silt seam)
525.0 55.5	Brown to reddich brown	13	SS	-	520		
	Firm to very stiff	14	TW	PH	510		
502.5 78.0	Glacial till - clayey silt with sand & grav. Hard. Brown.						
500.0		15	SS	70/6"	500		
80.5	End of borehole Probable ~~Mxx~~ Bedrock						

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT: 20 40 60 80 100

SHEAR STRENGTH P.S.F.: ● Triaxial, ○ Unconfined, + Field Vane, × Lab Vane: 400 800 1200 1600 2000

LIQUID LIMIT — w_L; PLASTIC LIMIT — w_P; WATER CONTENT — w; WATER CONTENT %: 20 40 60



DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT

FORM 06-MT-126
66-4545

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 4

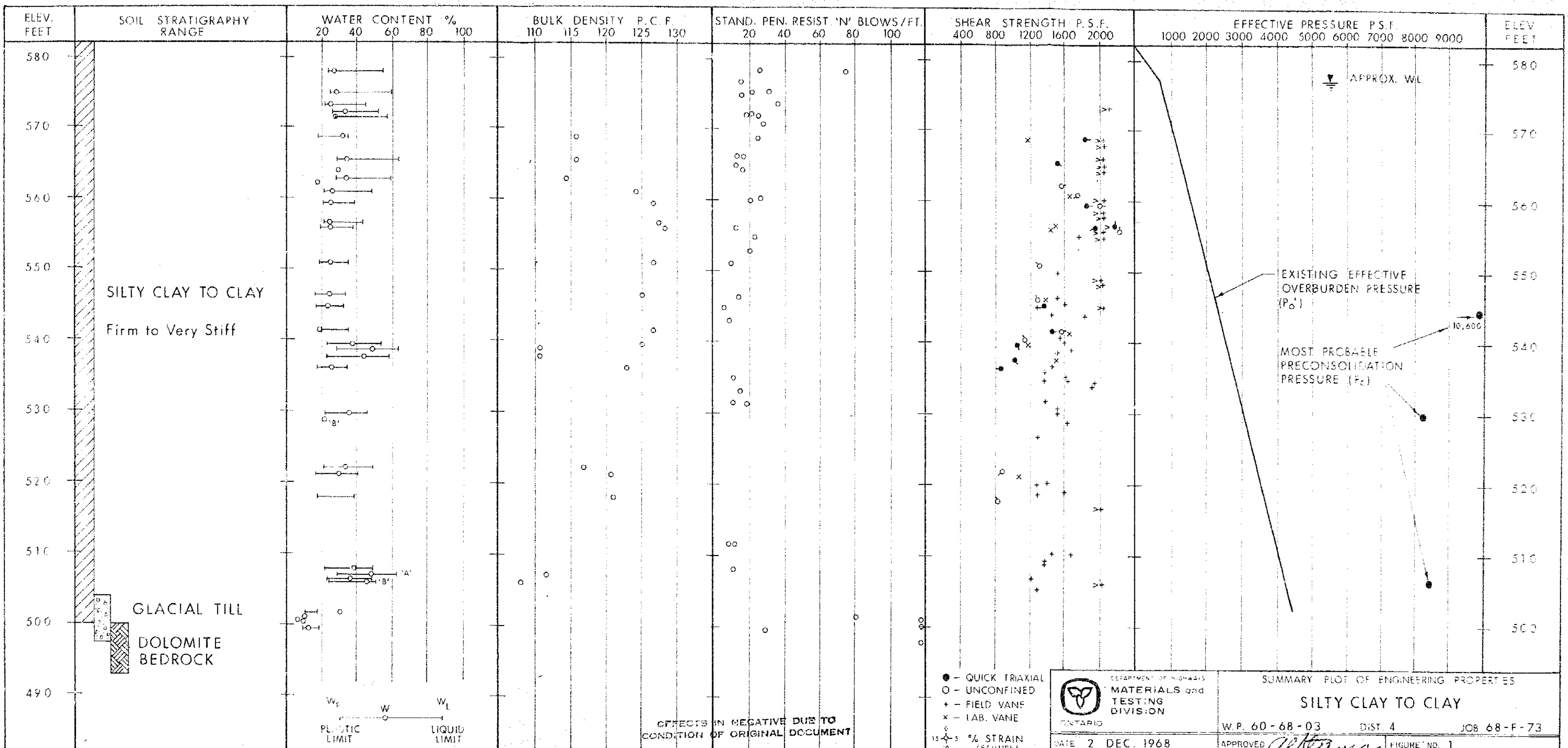
FOUNDATION SECTION

JOB 68-F-73 LOCATION Sta. 221+00 ℄ East Side Hwy. o/s 5' Lt. ORIGINATED BY WH
W.P. 60-68-03 BORING DATE Oct. 30-31, 1968 COMPILED BY WH
DATUM Geodetic BOREHOLE TYPE Diamond Drill - NX Casing CHECKED BY

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT
SHEAR STRENGTH P.S.F. ● Triaxial + Field Vane ○ Unconfined Comp. × Lab Vane — 400 800 1200 1600 2000
LIQUID LIMIT w_L, PLASTIC LIMIT w_P, WATER CONTENT w — WATER CONTENT % 20 40 60
BULK DENSITY γ P.C.F.
REMARKS Gr.Sa.Si.Cl

ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS/FOOT	ELEV. SCALE	SHEAR STRENGTH	γ P.C.F.	REMARKS Gr.Sa.Si.Cl
582.0	Ground Level							
0.0	Silty clay to clay with trace of sand & gravel	1	SS	76	580			▼579; 0 1 30 69
		2	SS	31				
	occ. very thin grey silt seams containing clear gypsum crystals above elev. 560	3	SS	19				
		4	TW	PM	570	+2000		
		5	SS	17		+2000		
		6	SS	17	560	s=4	124	
	Brown to reddish brown	6A	SS	13		+2000		
	Stiff to hard	7	TW	PM	550	s=2	127	
		8	SS	14		s=2		
		9	TW	PM	540	s=2, s=2		
		10	TW	PM		s=5, s=3, s=2	123	0 2 61 37
534.0 / 48.0	Occ. layers of silt up to 3" thick	11	SS	19	530	s=3		
		12	TW	PM	520	s=3, s=3	117	
520.0 / 62.0								
		13	SS	12	510	s=3, s=2		
		14	TW	PM		+2000		
504.0 / 78.0	Glacial till-clayey silt with sand & grav. Hard. Brown to grey.	15	SS	81	500			
		16	SS	64/3"				
497.3 / 84.7	End of Borehole. Probable Bedrock				490			

DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT



DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION
ONTARIO

SUMMARY PLOT OF ENGINEERING PROPERTIES

SILTY CLAY TO CLAY

W.P. 60-68-03 DIST. 4 JOB 68-F-73

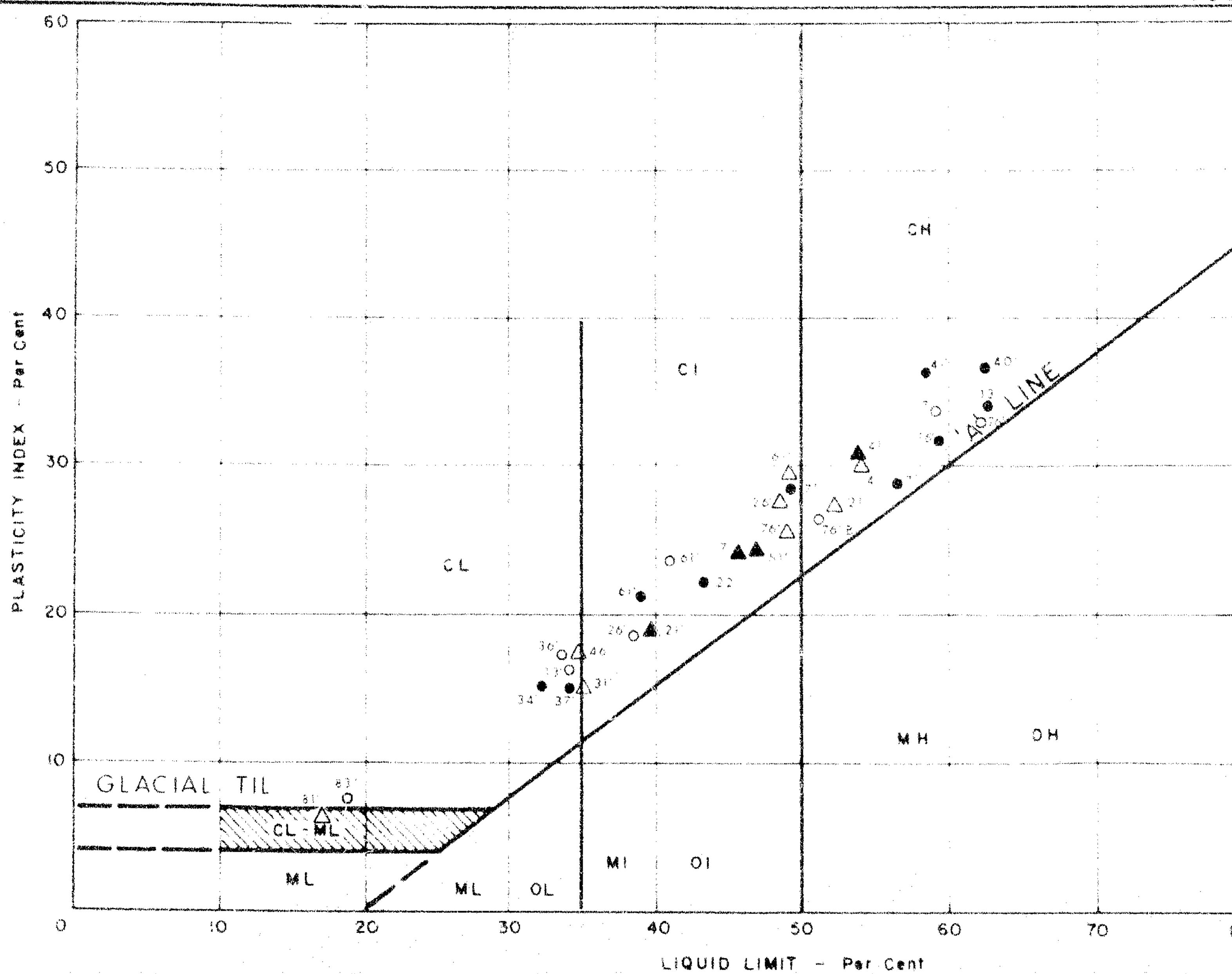
DATE 2 DEC. 1968

APPROVED

FIGURE NO. 1

FORM OB-MT-252
REVISED JULY 1966

DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT



LEGEND		
BH No.	SAMPLE No.	SYMBOL
1	2 – 16	●
2	2 – 17	o
3	2 – 12	▲
4	1 – 15	△

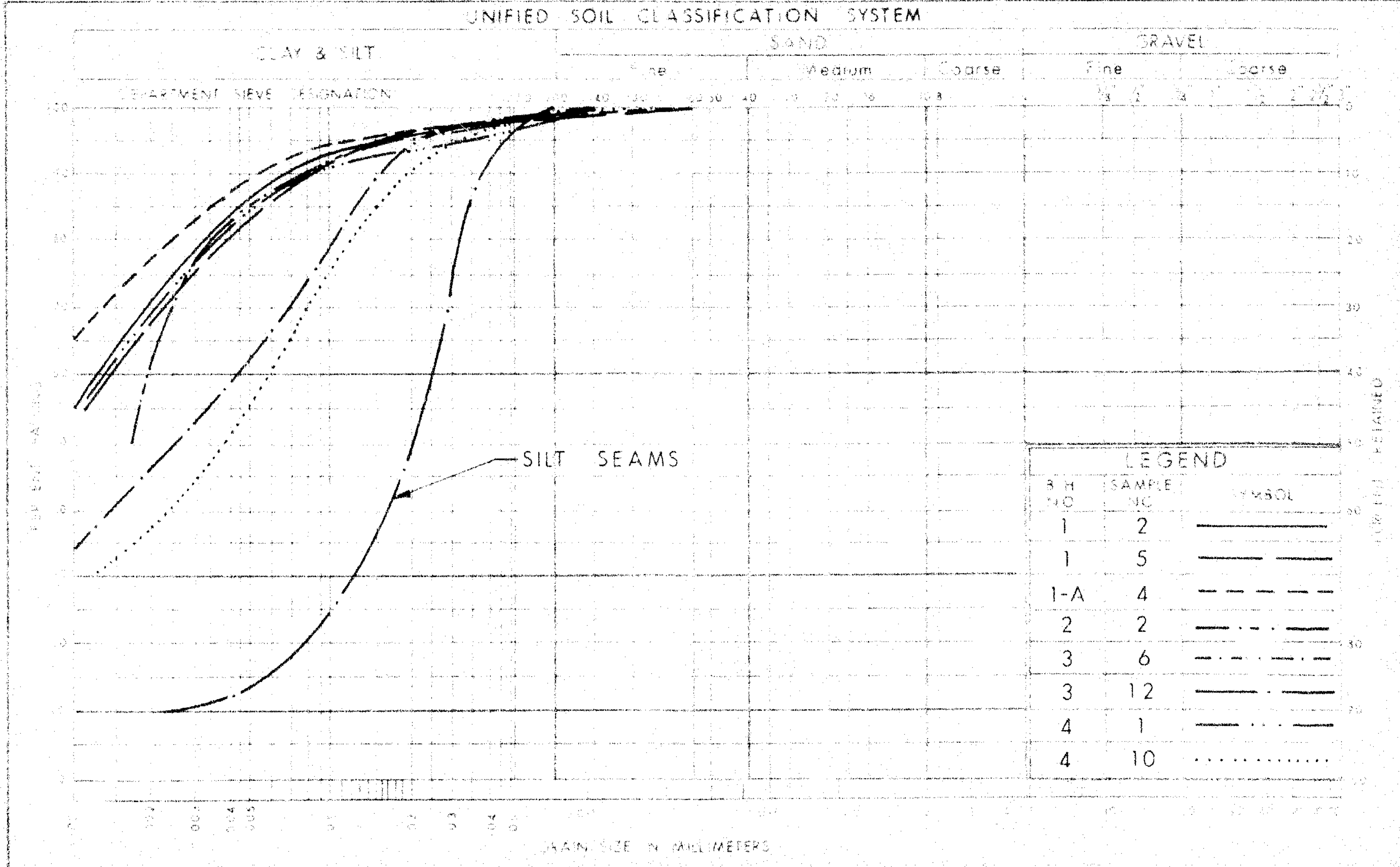
DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION

PLASTICITY CHART
SILTY CLAY TO CLAY
OCC. ZONES OF CLAYEY SILT

W.P. No. 60 - 68 - 03
JOB No. 68-F - 73
FIG. NO. 2

FORM DB MT-17
REV APR 1965

DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT



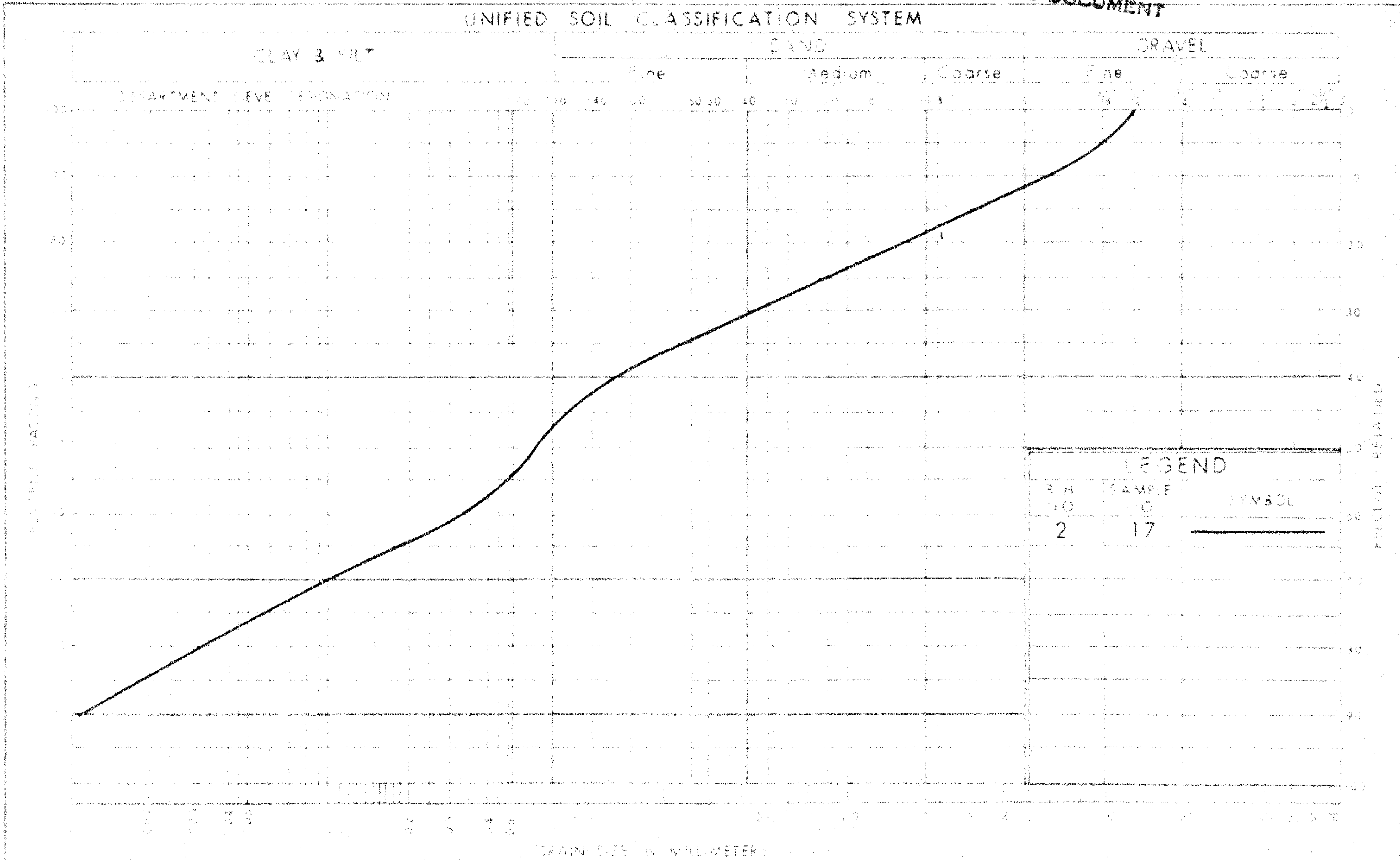
DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION

GRAIN SIZE DISTRIBUTION
SILTY CLAY TO CLAY
WITH TRACE OF SAND AND GRAVEL

W.P. No. 60 – 68 – 03
JOB No. 68 – F – 73
FIG. NO. 3

FORM OB MT-17
REV APR 1965

DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT



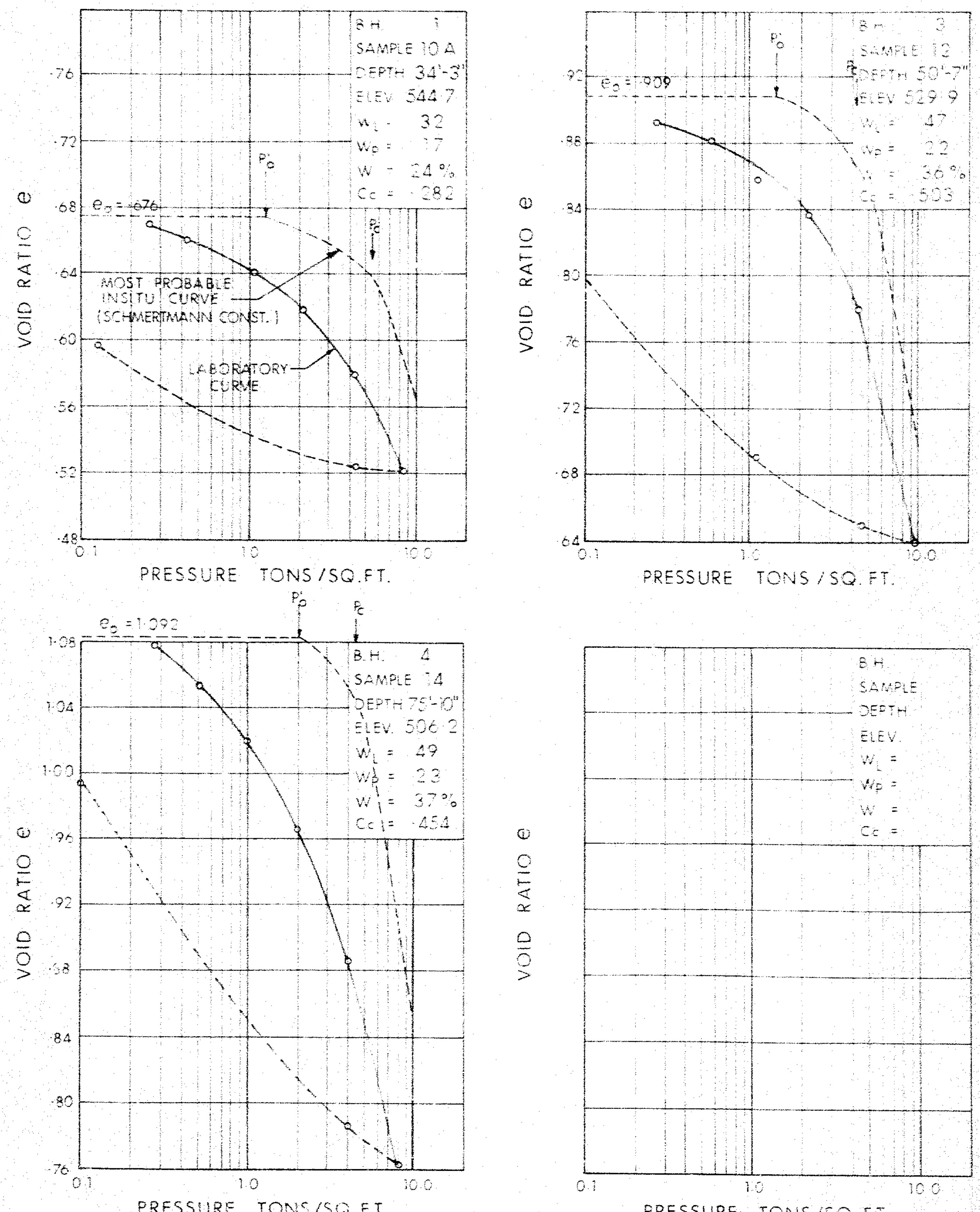
DEPARTMENT OF HIGHWAYS
MATERIALS and TESTING DIVISION

GRAIN SIZE DISTRIBUTION
GLACIAL TILL
CLAYEY SILT WITH SAND & GRAVEL

W.P. No. 60 – 68 – 03
JOB No. 68 – F – 73
FIG. NO. 4

VOID RATIO – PRESSURE CURVES

JOB NO. 68-F-73

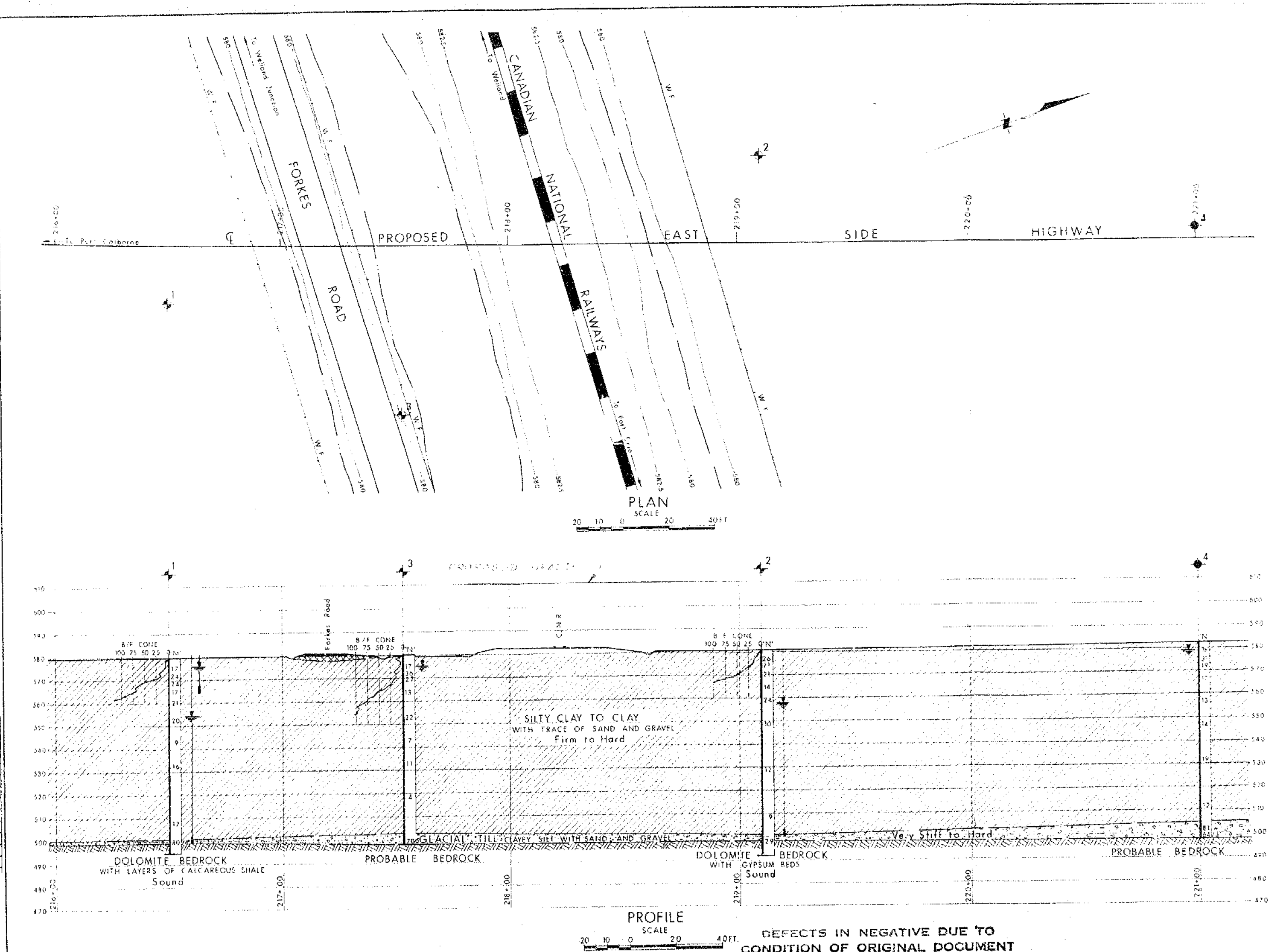


DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT

FIG. 5

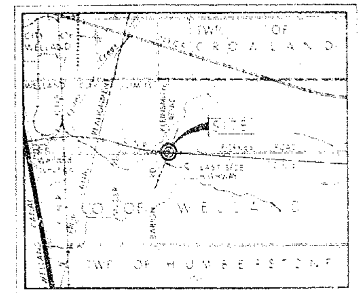
P_o' - EFFECTIVE OVERBURDEN PRESSURE

P_c - MOST PROBABLE PRECONSOLIDATION PRESSURE



DEFECTS IN NEGATIVE DUE TO
CONDITION OF ORIGINAL DOCUMENT

REF. No. S.L.S.A. DRG. No. 26696 SHEET No. 6



KEY PLAN
SCALE IN MILES

LEGEND

- Bore Hole
- Cone Penetration Hole
- Bore & Cone Penetration Hole
- Water Levels established at time of field investigation Oct. 1968
- Piezometer

NO.	ELEVATION	STATION	OFFSET
1	579.0	216+50	25' RT
2	582.0	219+10	38' LT
3	580.5	217+53	75' RT
4	582.0	221+00	8' LT

- NOTE -

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence and may be subject to considerable error.

REVISIONS		
DATE	BY	DESCRIPTION

DEPARTMENT OF HIGHWAYS – ONTARIO
MATERIALS & TESTING DIVISION – FOUNDATION SECTION

CANADIAN NATIONAL RAILWAYS OVERHEAD
(NEAR FORKES ROAD)

KING'S HIGHWAY NO. EAST SIDE HIGHWAY DIST. NO. 4
CO. WELLAND
TWP. HUMBERSTONE LOT 16 & 20 CON. V & IV

BORE HOLE LOCATIONS & SOIL STRATA

SUBM'D W. H.	CHECKED	W.P. NO. 60-68-03	M.&T. DRAWING NO. 68–F–73A
DRAWN G. P.	CHECKED	JOB NO. 68-F-73	
DATE Nov. 27, 1968		SITE NO.	BRIDGE DRAWING NO.
APPROVED (signature) PRINCIPAL FOUNDATION ENGINEER		CONT. NO.	

PRINT RECORD

NO.	FOR	DATE

APPENDIX E

Site Photographs – September 2008

Photographs of Distressed Areas
Golder Site Visit September 2008
Highway 140 / CNR Overpass North Embankment and Approach

FIGURE E1

Photo #1: Sloughing of granular at crest of slope, west side of north embankment facing north.

Photo #2: Sloughing of granular inside guide rail, west side of north embankment facing north.

DATE: June 2009
PROJECT: 08-1111-0031

Golder Associates

DWG: MWK
CHK: JPD

Photographs of Distressed Areas
Golder Site Visit September 2008
Highway 140 / CNR Overpass North Embankment and Approach

FIGURE E2

Photo #4: Tilting of guide rail along north embankment, facing south.

DATE: June 2009

PROJECT: 08-1111-0031

Golder Associates

DWG: MWK

CHK: JPD

Photographs of Distressed Areas
Golder Site Visit September 2008
Highway 140 / CNR Overpass North Embankment and Approach

FIGURE E3

Photo #5: Tension crack along crest of east slope of north embankment, facing south.

DATE: June 2009

PROJECT: 08-1111-0031

Golder Associates

DWG: MWK

CHK: JPD

Photographs of Distressed Areas
Golder Site Visit September 2008
Highway 140 / CNR Overpass North Embankment and Approach

FIGURE E4

Photo #3: Sloughing of granular at crest of slope, west side of north embankment facing north.

DATE: June 2009

PROJECT: 08-1111-0031

Golder Associates

DWG: MWK

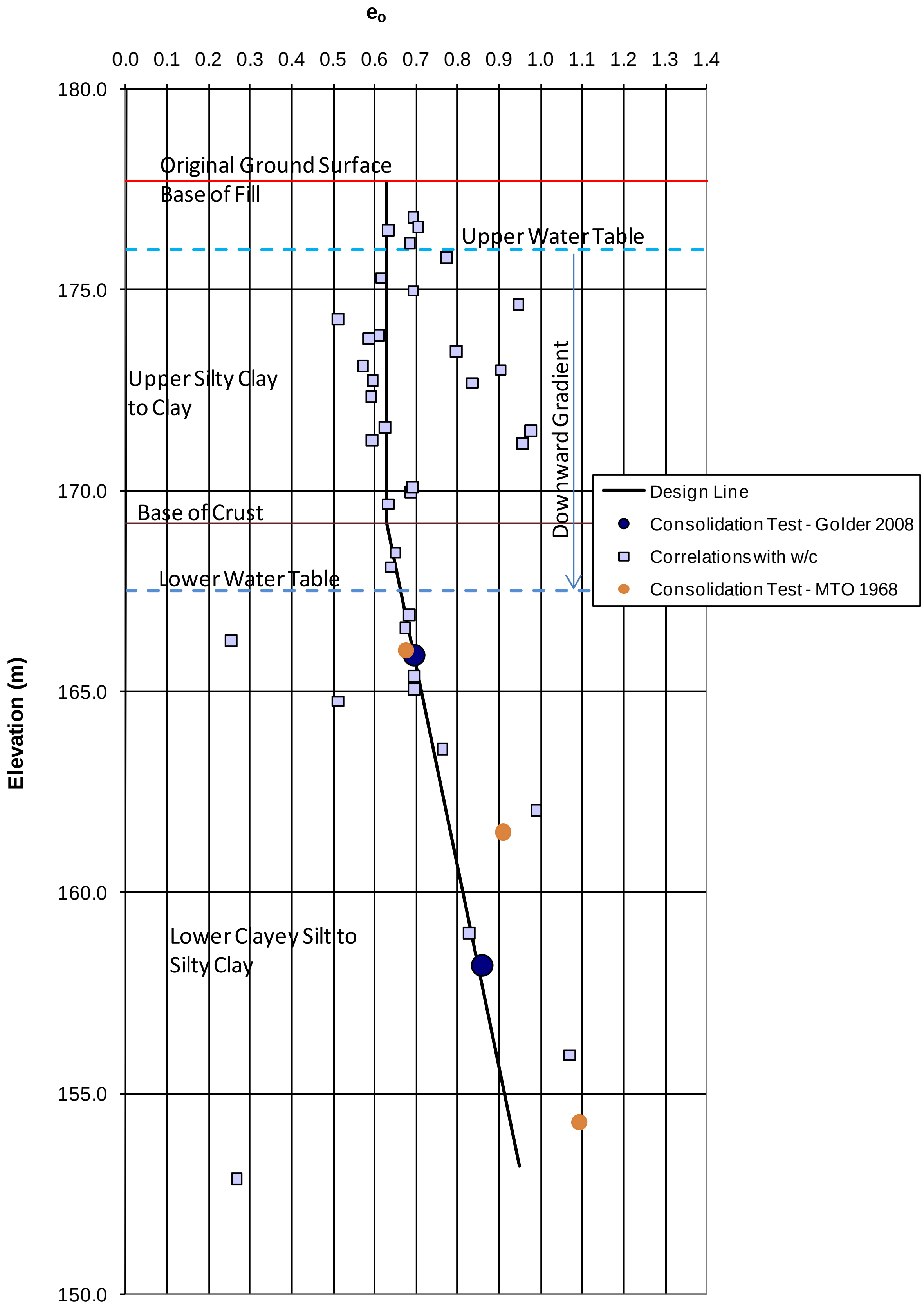
CHK: JPD

APPENDIX F

Data Interpretation and Analysis

Void Ratio vs. Elevation
Highway 140 / CNR Overpass – North Embankment and Approach

FIGURE F1



DATE: JUNE 2009

PROJECT: 08-1111-0031

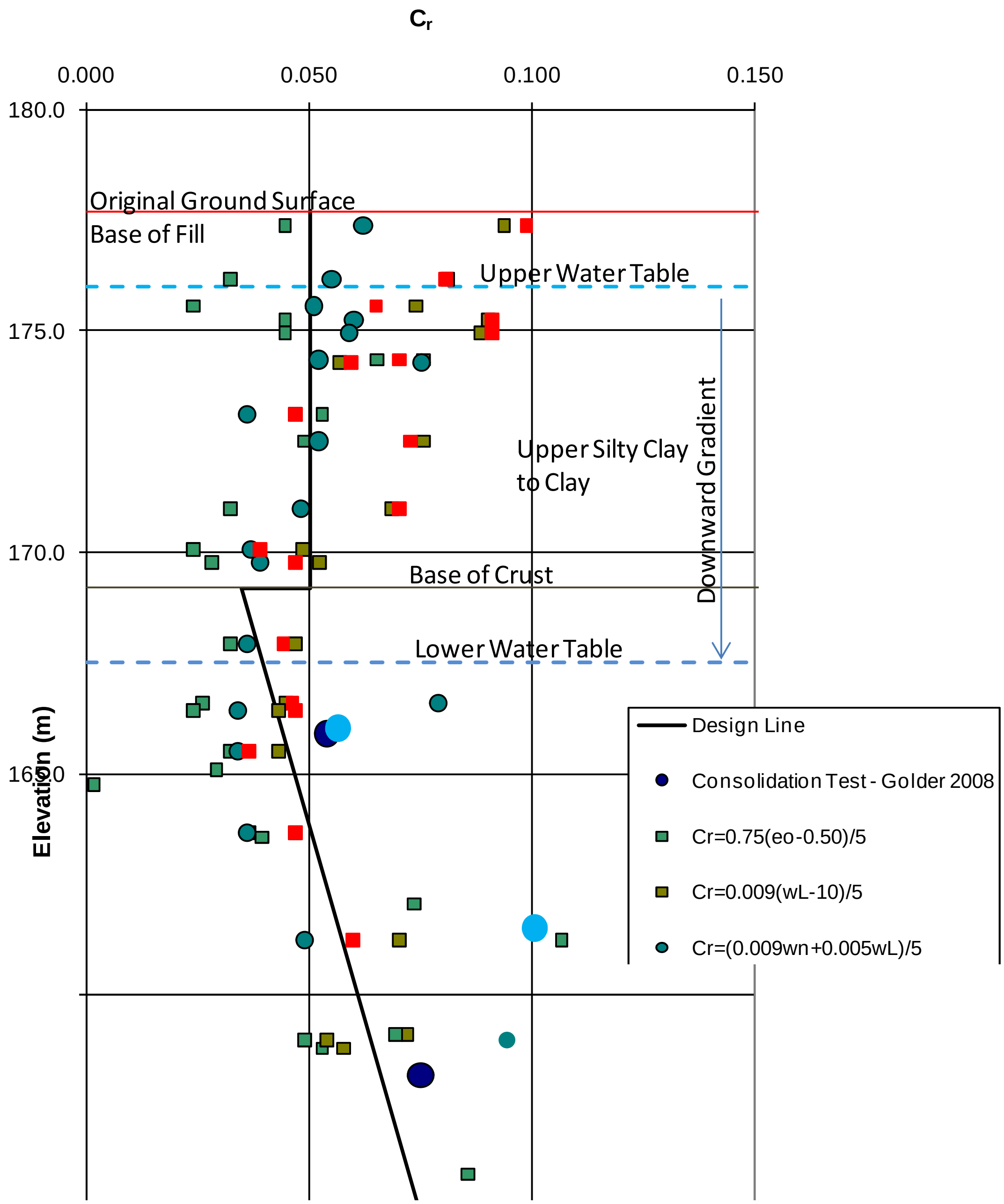
Golder Associates

DWG: MWK

CHK: JPD

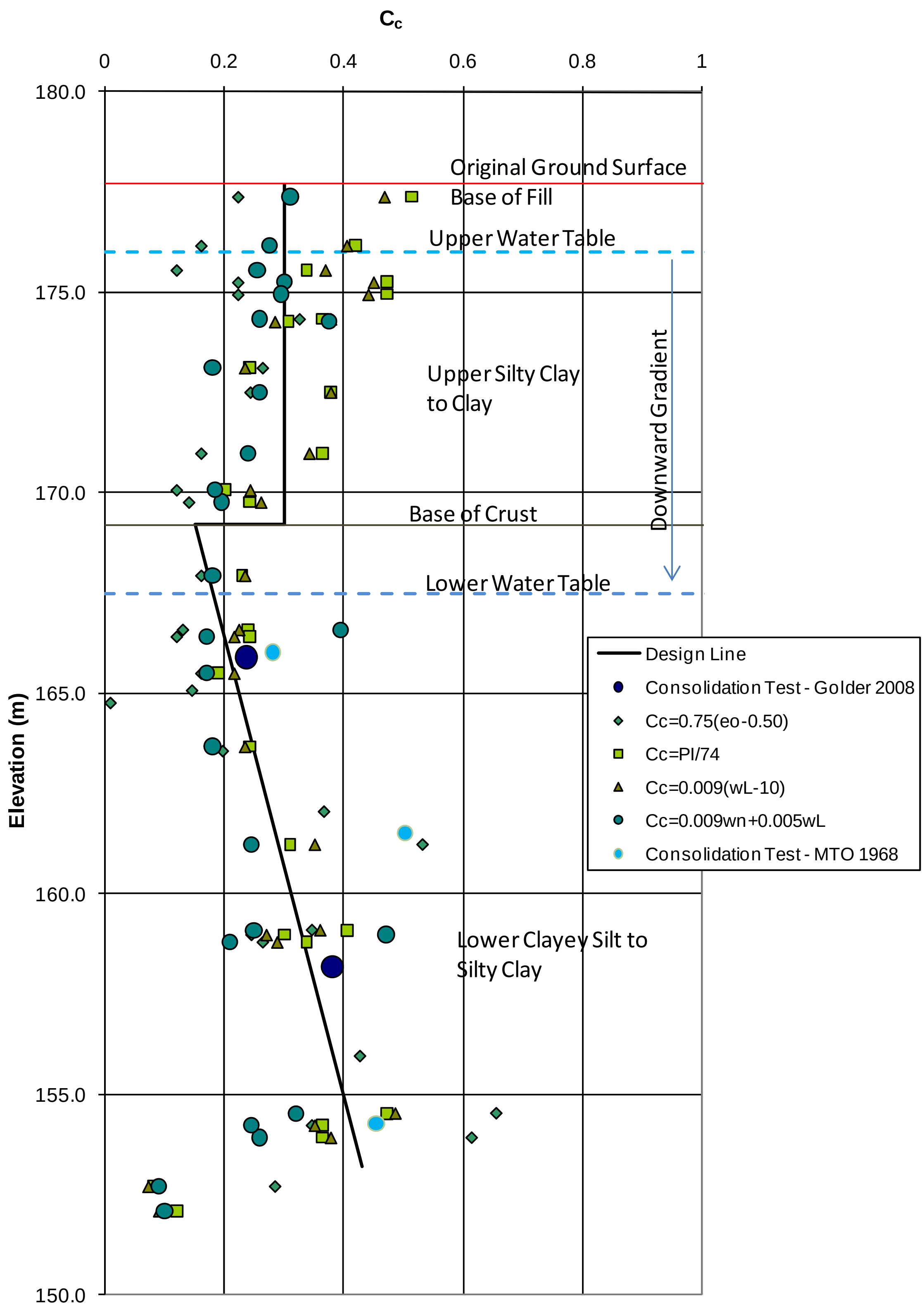
Recompression Index vs. Elevation
Highway 140 / CNR Overpass – North Embankment and Approach

FIGURE F2



Compression Index vs. Elevation
Highway 140 / CNR Overpass – North Embankment and Approach

FIGURE F3



DATE: JUNE 2009

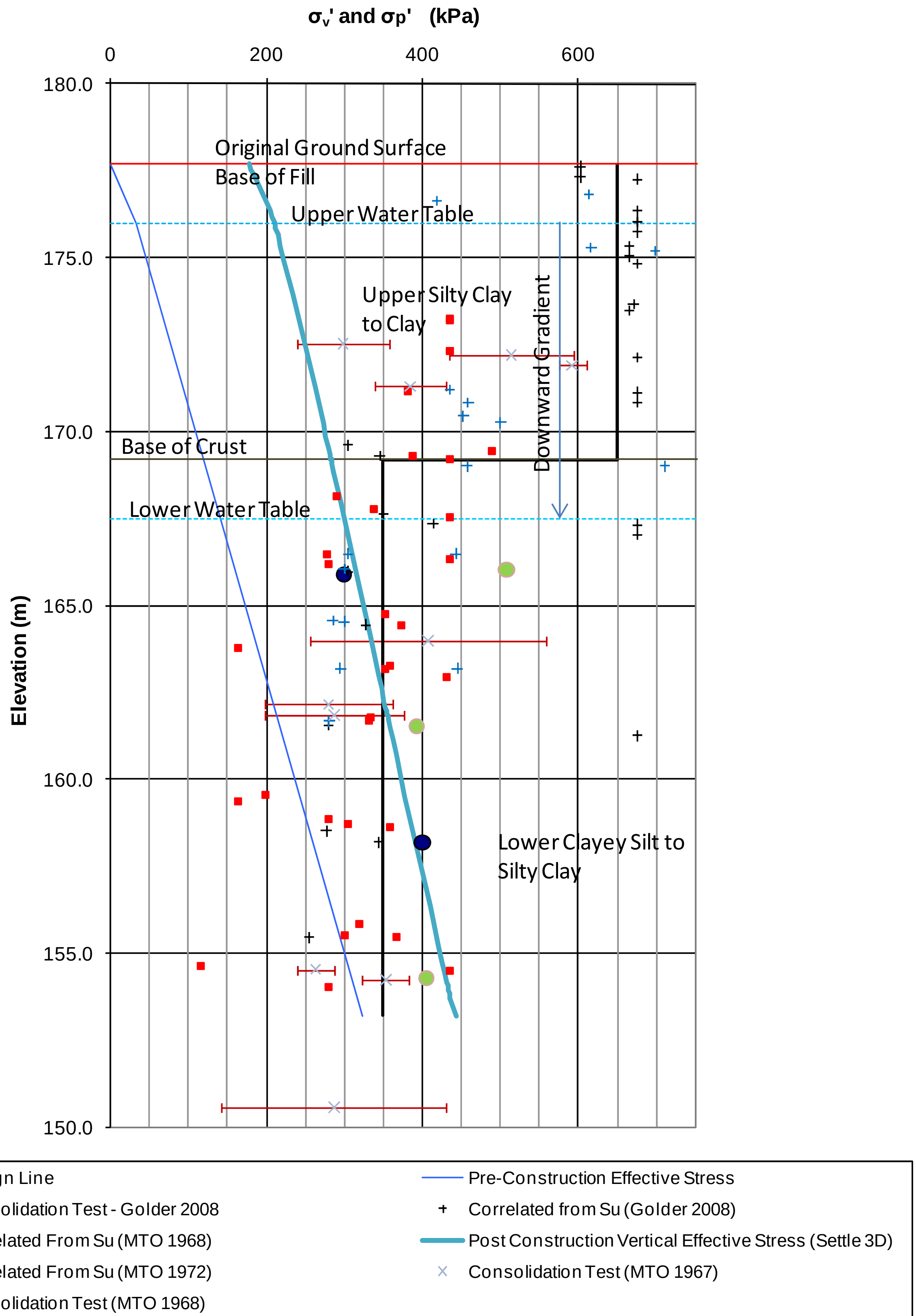
PROJECT: 08-1111-0031

DWG: MWK

CHK: JPD

Effective Vertical and Preconsolidation Stress vs. Elevation
Highway 140 / CNR Overpass – North Embankment and Approach

FIGURE F4



DATE: JUNE 2009

PROJECT: 08-1111-0031

Golder Associates

DWG: MWK

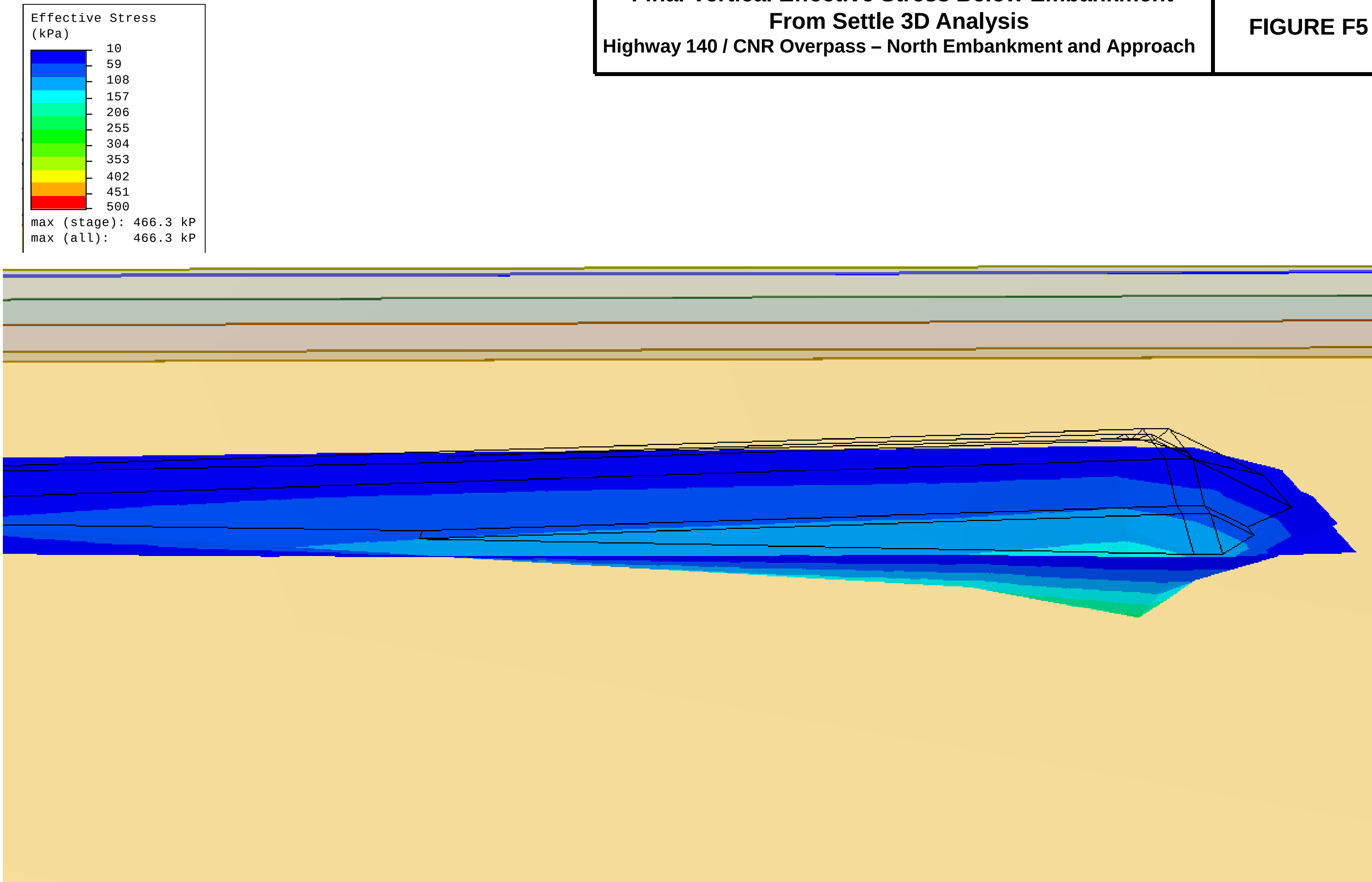
CHK: JPD

Project: 08-1111-0031 Drawn: MWK Reviewed: JPD Rev.: May 2009

Final Vertical Effective Stress Below Embankment From Settle 3D Analysis

Highway 140 / CNR Overpass – North Embankment and Approach

FIGURE F5



DATE: JUNE 2009

PROJECT: 08-1111-0031

Golder Associates

DWG: MWK

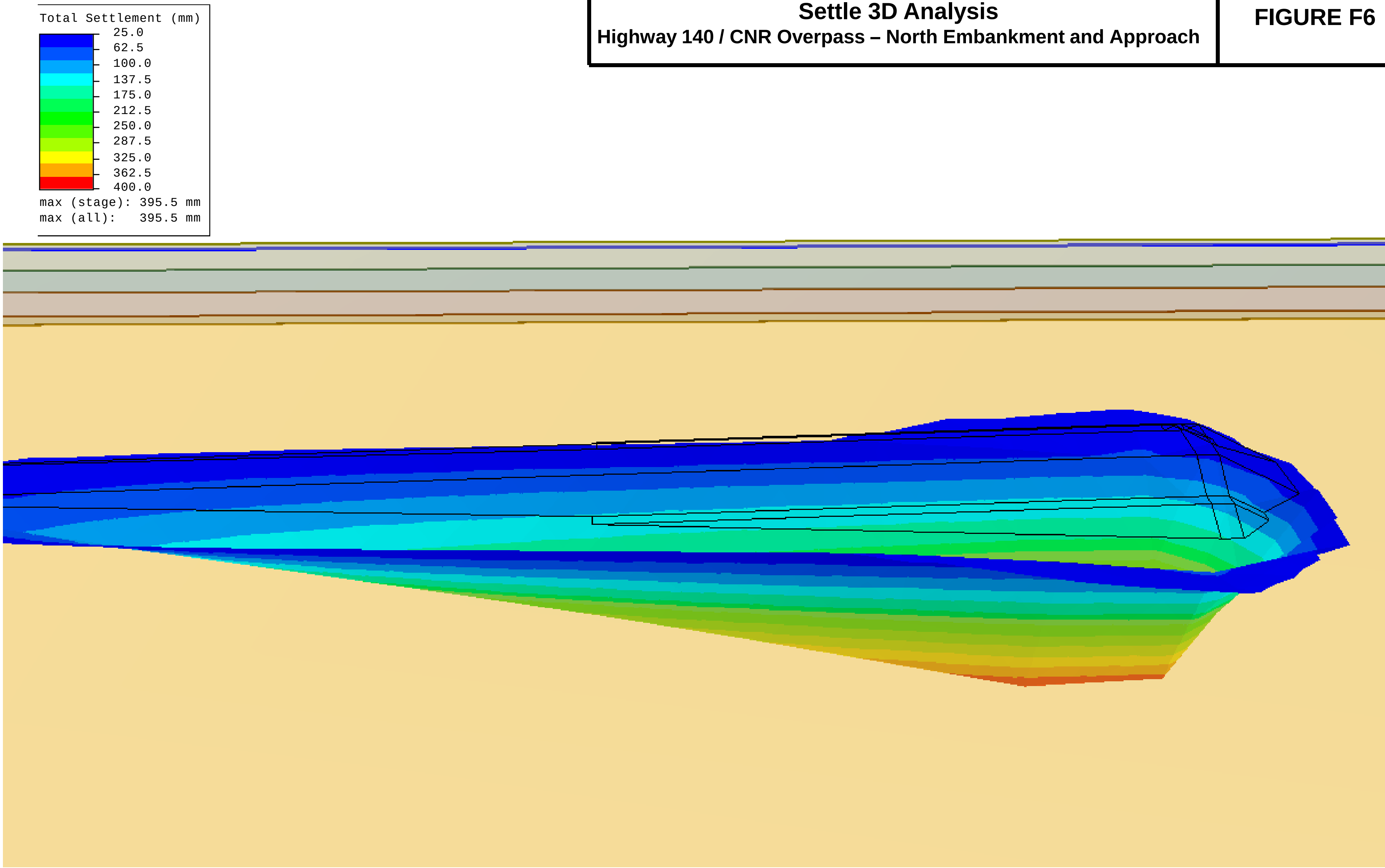
CHK: JPD

Project: 08-1111-0031 Drawn: MWK Reviewed: JPD Rev.: May 2009

Consolidation Settlement in Foundation Strata From Settle 3D Analysis

Highway 140 / CNR Overpass – North Embankment and Approach

FIGURE F6



DATE: JUNE 2009

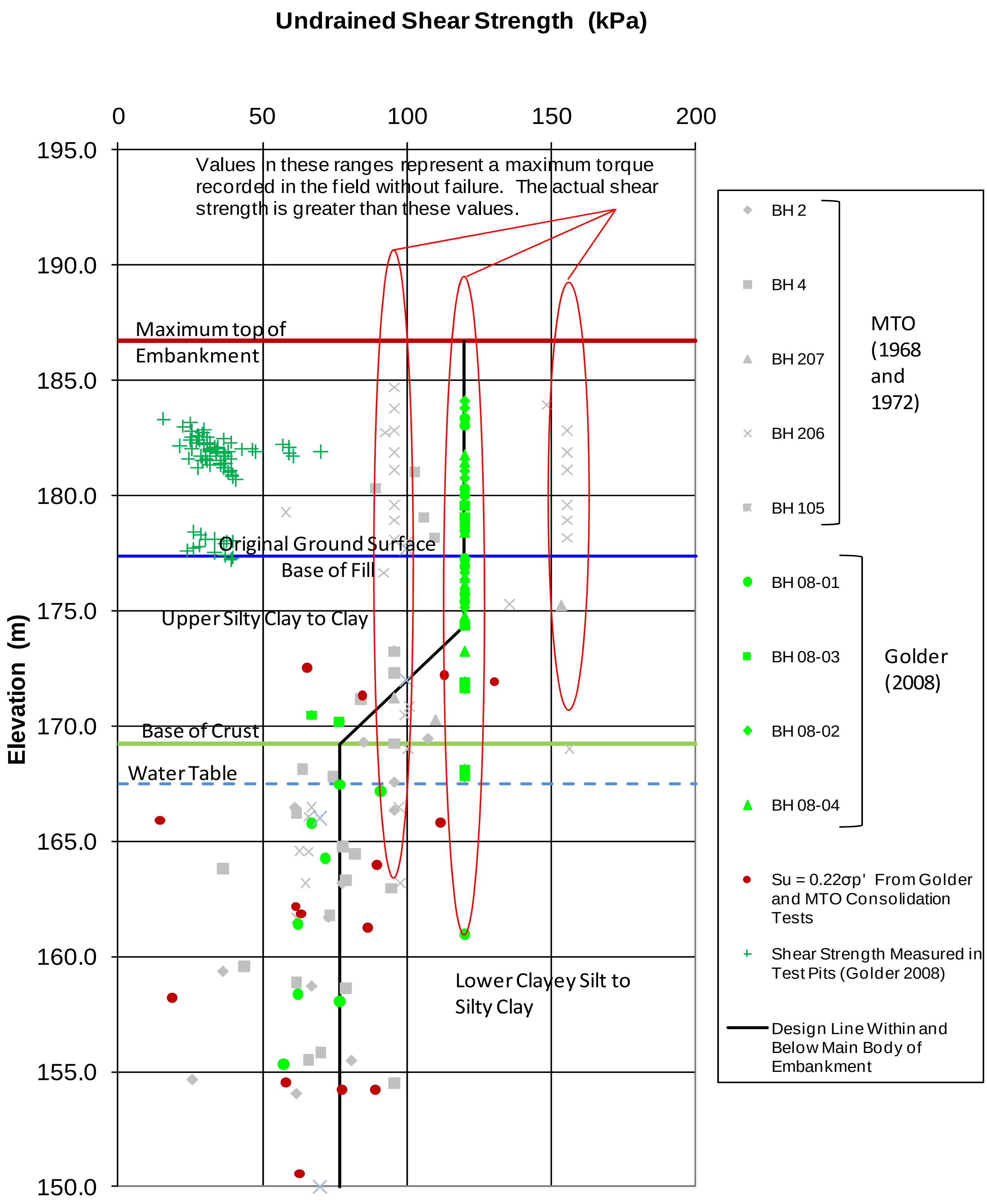
PROJECT: 08-1111-0031

DWG: MWK

CHK: JPD

In Situ Field Vane Test Results (from Boreholes and Test Pits)
(Golder 2008 and MTO 1968 and 1972 Investigations)

FIGURE F7



DATE: JUNE 2009

PROJECT: 08-1111-0031

Golder Associates

DWG: MWK

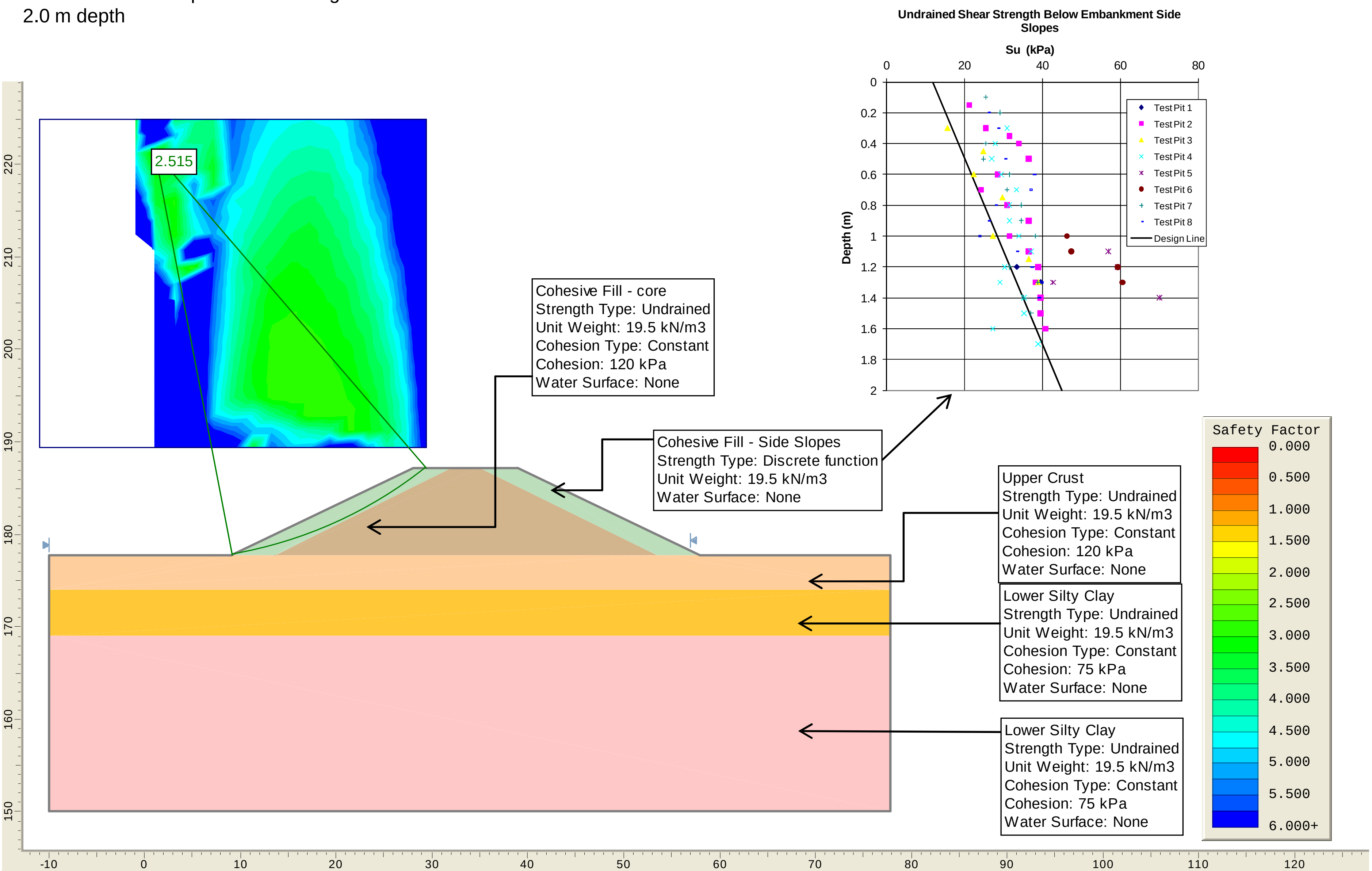
CHK: JPD

Project: 08-1111-0031 Drawn: MWK Reviewed: JPD Rev.: May 2009

Total Stress
Stability Analysis – Original Embankment Geometry
Before Construction of Berms

FIGURE F8

Stability Analysis – 9.5 m High Embankment
08-1111-031 MTO/Hwy 140/Embankment
Cohesive fill in side slopes based on shear strength measured in side walls of test pits – 12 kPa at ground surface to 45 kPa at 2.0 m depth



DATE: JUNE 2009
PROJECT: 08-1111-0031

Golder Associates

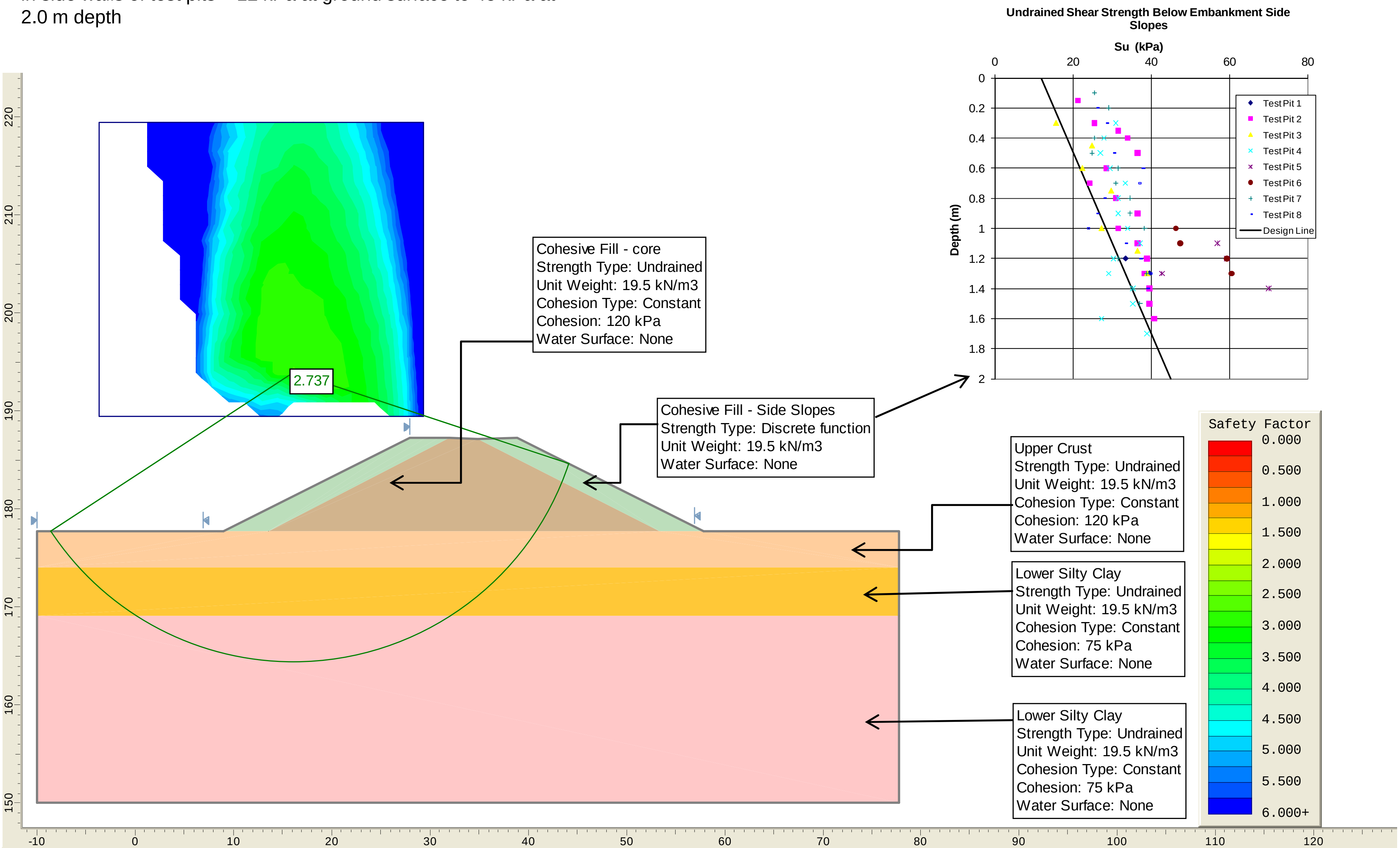
DWG: MWK
CHK: JPD

Project: 08-1111-0031 Drawn: MWK Reviewed: JPD Rev.: May 2009

Total Stress
Stability Analysis – Original Embankment Geometry
Before Construction of Berms

FIGURE F9

Stability Analysis – 9.5 m High Embankment
08-1111-031 MTO/Hwy 140/Embankment
Cohesive fill in side slopes based on shear strength measured in side walls of test pits – 12 kPa at ground surface to 45 kPa at 2.0 m depth



DATE: JUNE 2009

PROJECT: 08-1111-0031

Golder Associates

DWG: MWK

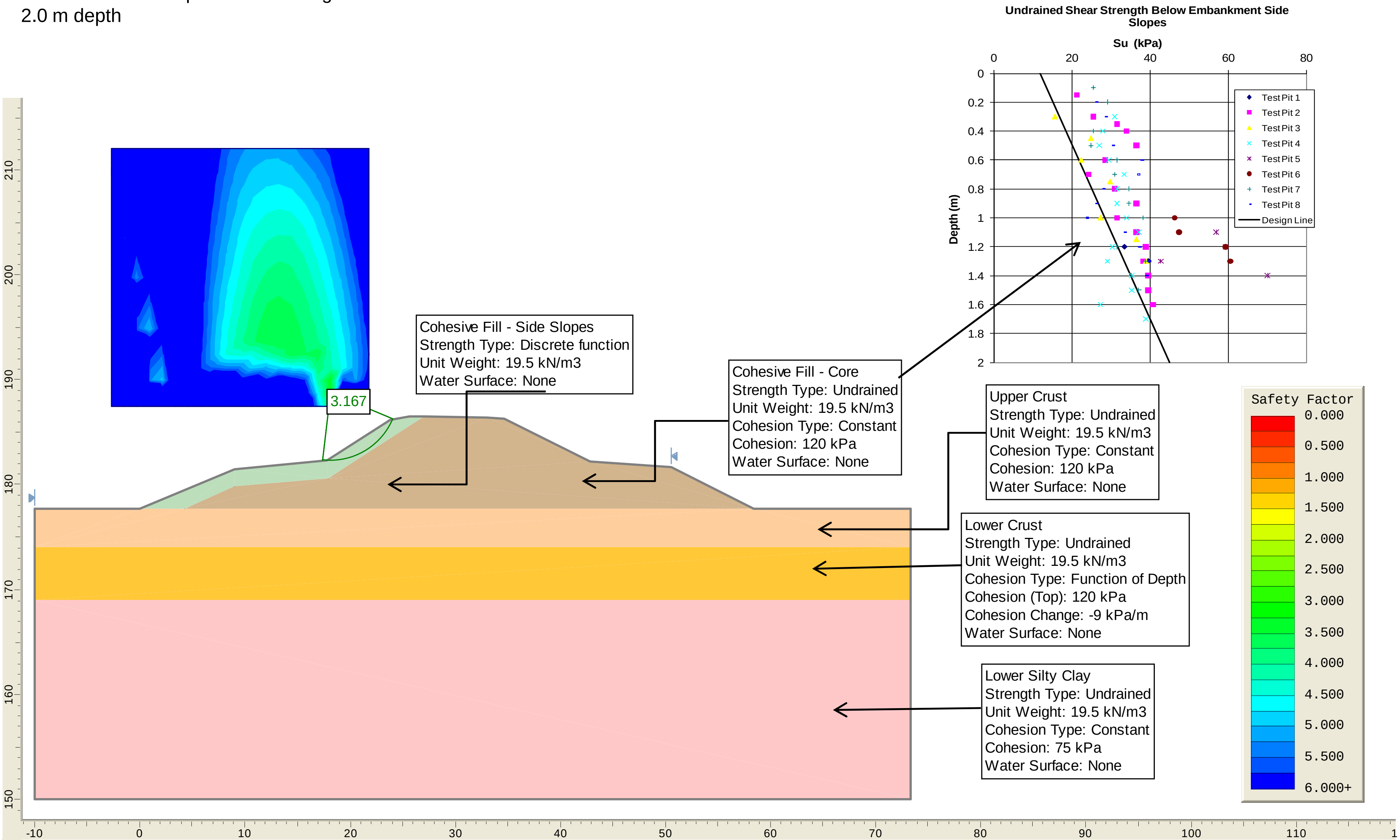
CHK: JPD

Project: 08-1111-0031 Drawn: MWK Reviewed: JPD Rev.: May 2009

Total Stress
Stability Analysis – Original Embankment Geometry
After Construction of Berms

FIGURE F10

Stability Analysis – 9.5 m High Embankment
08-1111-031 MTO/Hwy 140/Embankment
Cohesive fill in side slopes based on shear strength measured in side walls of test pits – 12 kPa at ground surface to 45 kPa at 2.0 m depth



DATE: JUNE 2009

PROJECT: 08-1111-0031

Golder Associates

DWG: MWK

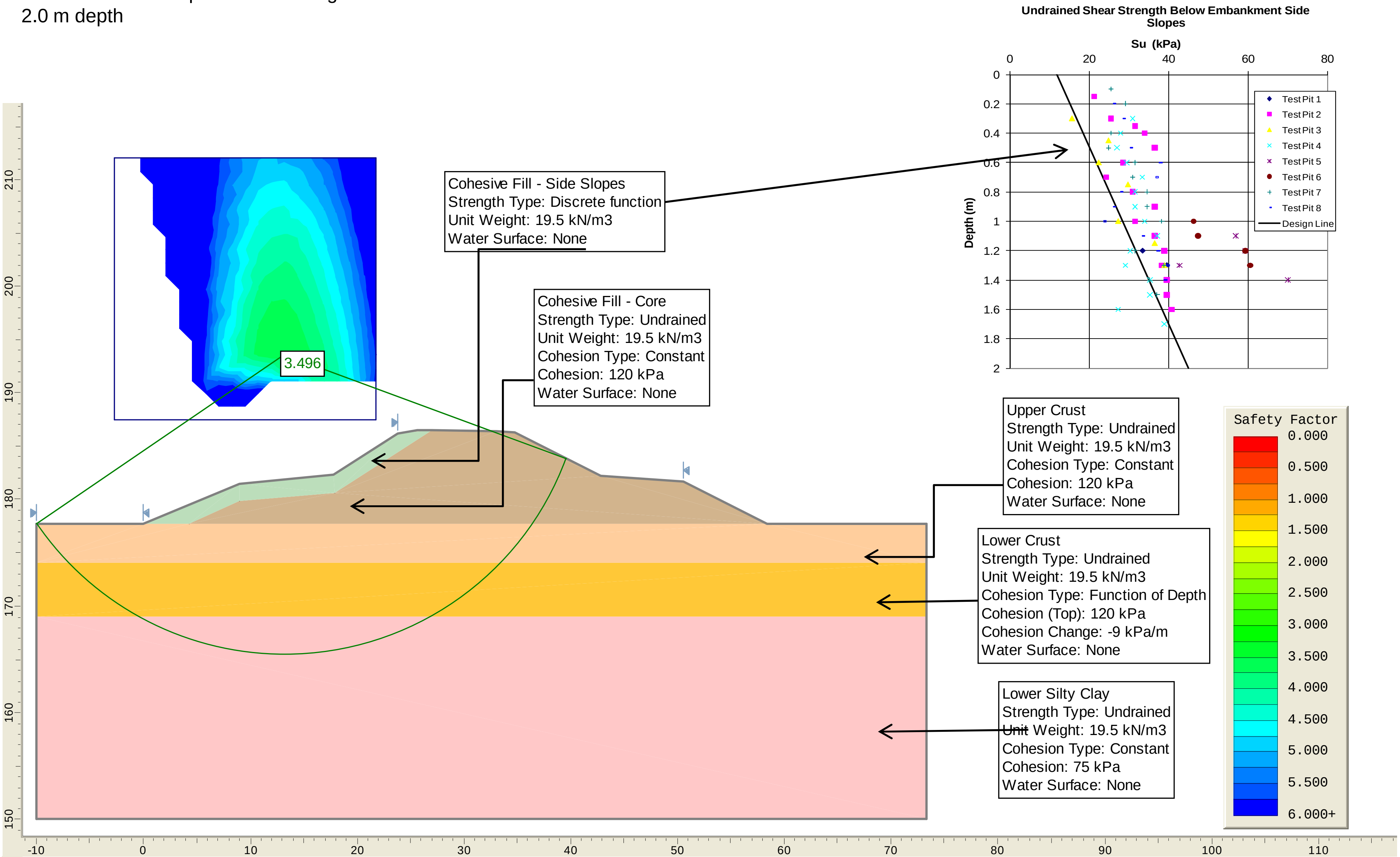
CHK: JPD

Project: 08-1111-0031 Drawn: MWK Reviewed: JPD Rev.: May 2009

Stability Analysis – 9.5 m High Embankment
08-1111-031 MTO/Hwy 140/Embankment
Cohesive fill in side slopes based on shear strength measured in side walls of test pits – 12 kPa at ground surface to 45 kPa at 2.0 m depth

Total Stress
Stability Analysis – Original Embankment Geometry After Construction of Berms

FIGURE F11



DATE: JUNE 2009
PROJECT: 08-1111-0031

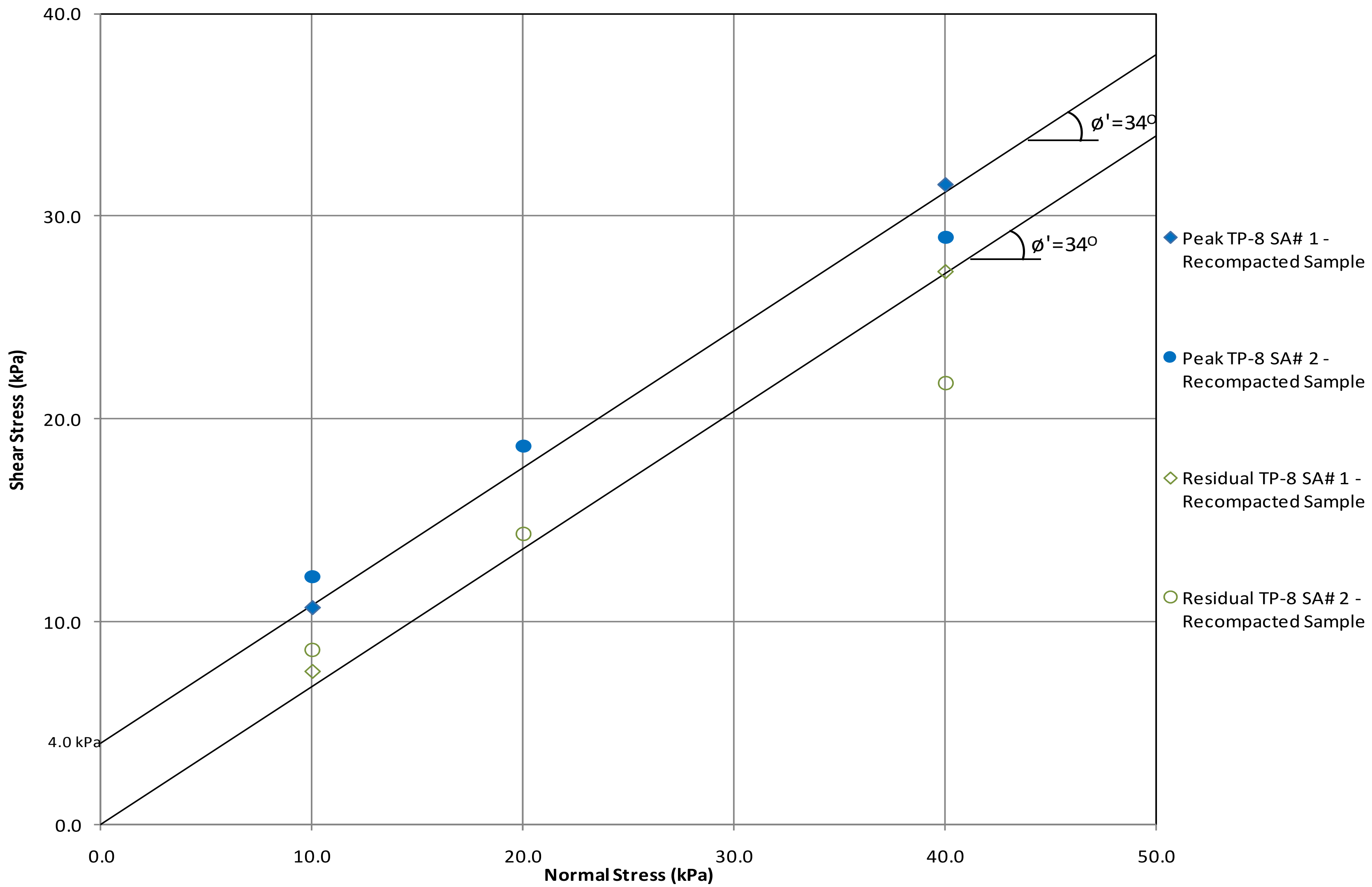
Golder Associates

DWG: MWK
CHK: JPD

Project: 08-1111-0031 Drawn: MWK Reviewed: JPD Rev.: May 2009

Direct Shear Testing Results
Mohr – Coulomb Failure Envelope(s)
Clay Fill

FIGURE F12



DATE: JUNE 2009

PROJECT: 08-1111-0031

Golder Associates

DWG: MWK

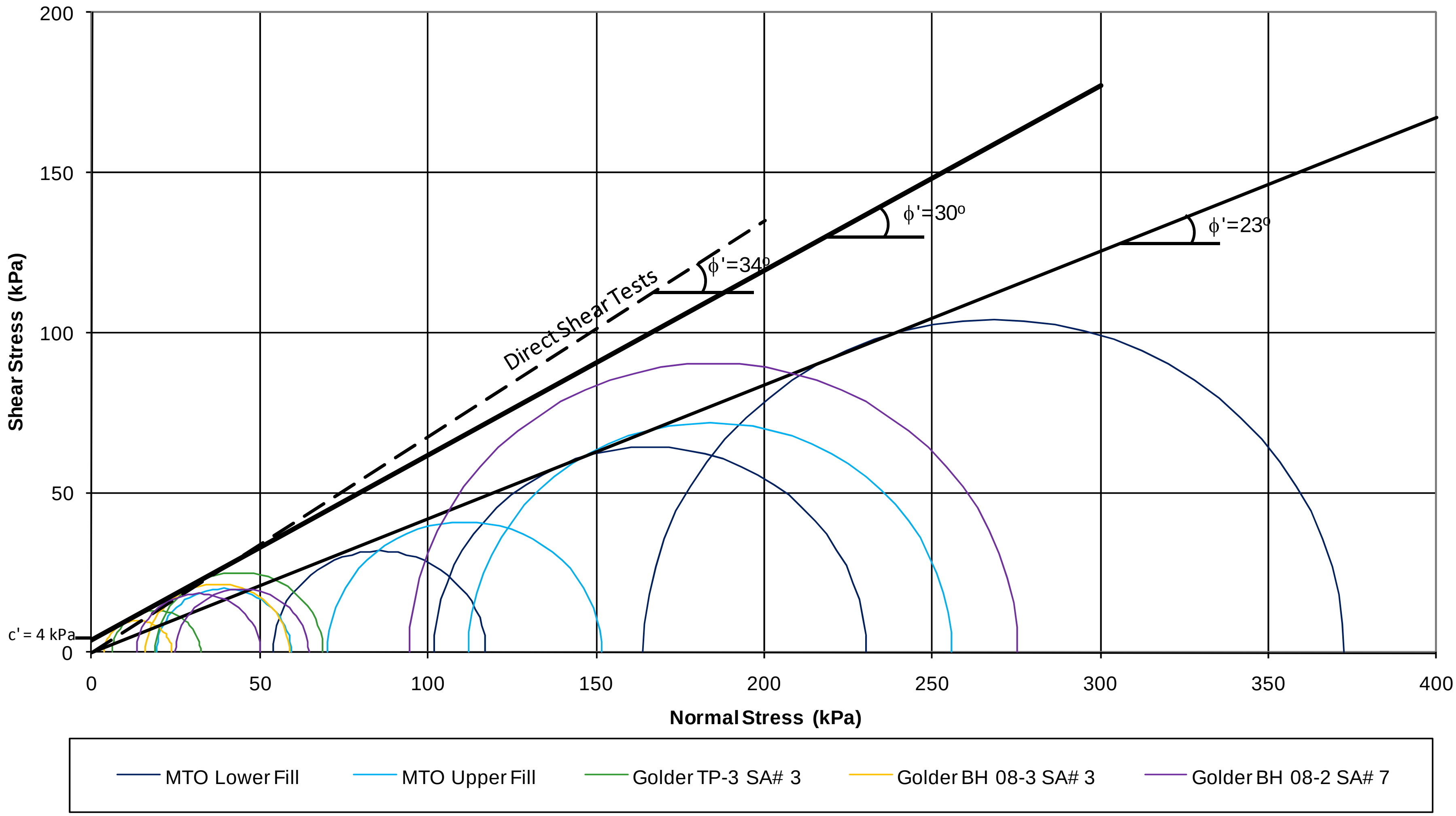
CHK: JPD

Project: 08-1111-0031 Drawn: MWK Reviewed: JPD Rev.: May 2009

Triaxial Testing Results
Mohr – Coulomb Failure Envelope(s)
Clay Fill

FIGURE F13

Golder and MTO Triaxial Test Results



DATE: JUNE 2009

PROJECT: 08-1111-0031

DWG: MWK

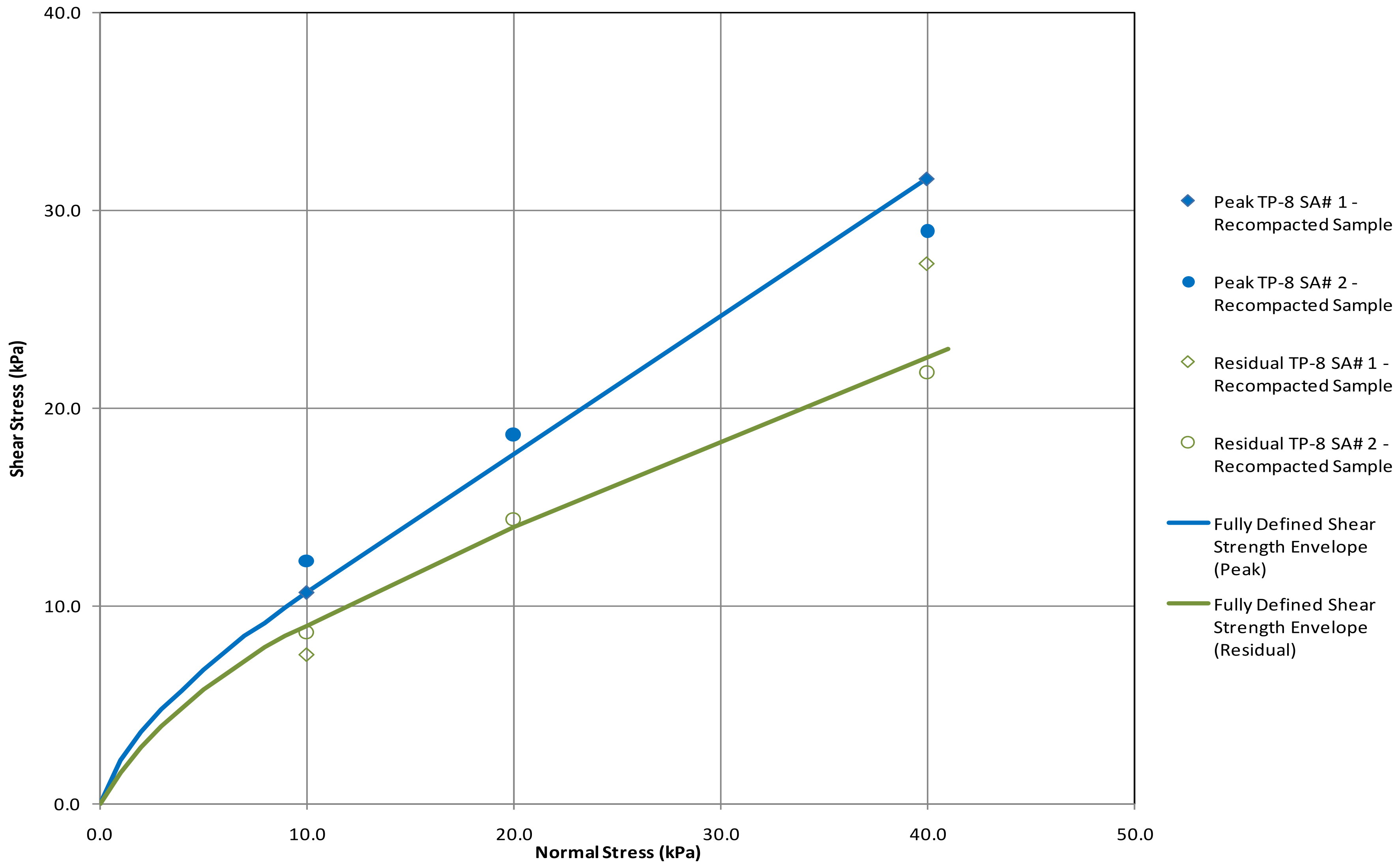
CHK: JPD

Golder Associates

Project: 08-1111-0031 Drawn: MWK Reviewed: JPD Rev.: May 2009

Direct Shear Testing Results
Fully Defined Failure Envelopes
Clay Fill

FIGURE F14



DATE: JUNE 2009

PROJECT: 08-1111-0031

Golder Associates

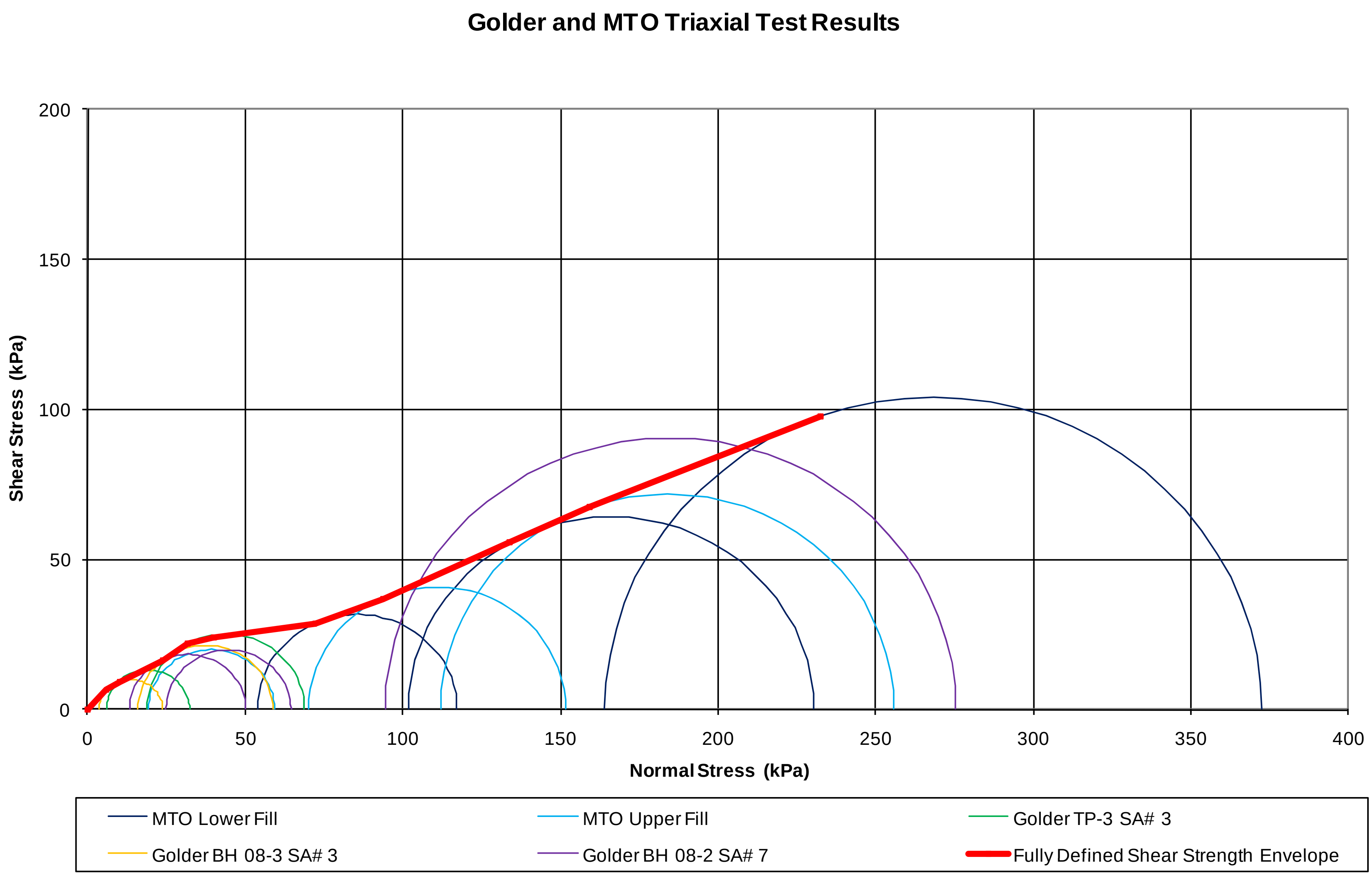
DWG: MWK

CHK: JPD

Project: 08-1111-0031 Drawn: MWK Reviewed: JPD Rev.: May 2009

Triaxial Testing Results
Fully Defined Failure Envelope
Clay Fill

FIGURE F15



DATE: JUNE 2009

PROJECT: 08-1111-0031

DWG: MWK

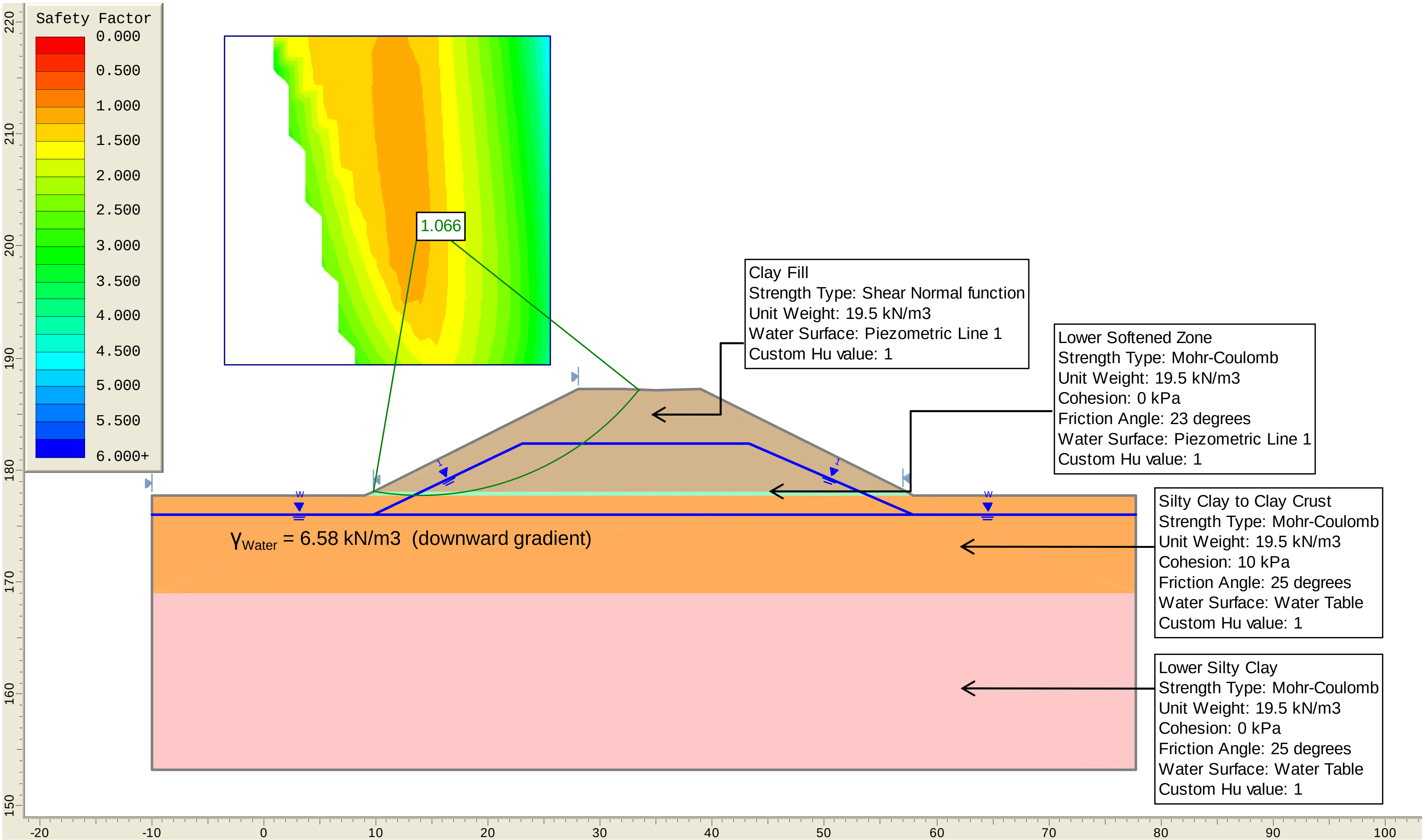
CHK: JPD

Project: 08-1111-0031 Drawn: MWK Reviewed: JPD Rev.: May 2009

Effective Stress
Stability Analysis – Original Embankment Geometry
Before Construction of Berms

FIGURE F16

Stability Analysis – 9.5 m High Embankment
08-1111-031 MTO/Hwy 140/Embankment
Strength of embankment fill given by fully defined strength envelope based on Golder and MTO triaxial test results



DATE: JUNE 2009

PROJECT: 08-1111-0031

Golder Associates

DWG: MWK

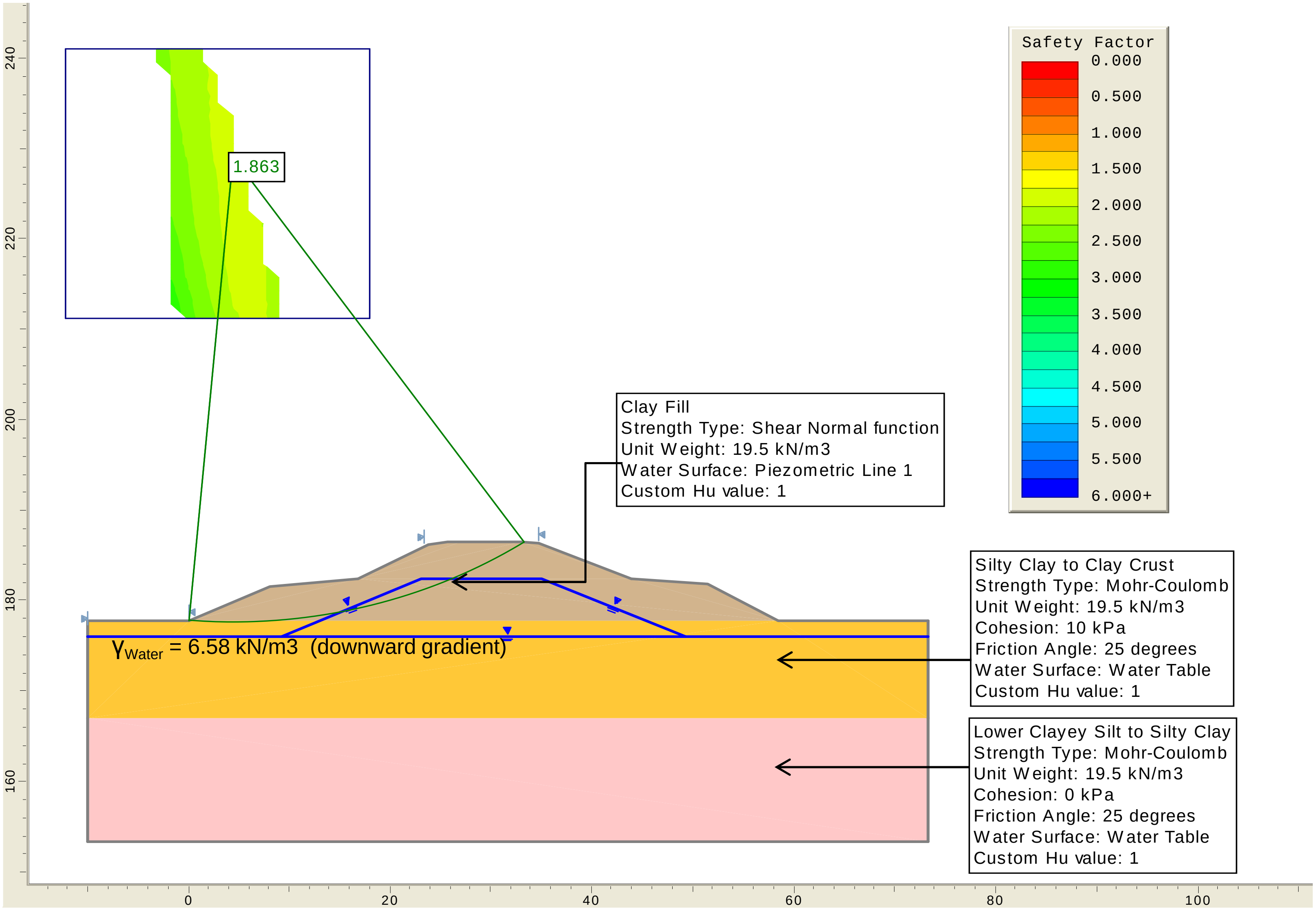
CHK: JPD

Project: 08-1111-0031 Drawn: MWK Reviewed: JPD Rev.: May 2009

Stability Analysis – 9.5 m High Embankment
08-1111-031 MTO/Hwy 140/Embankment
Strength of embankment fill given by fully defined strength envelope based on Golder and MTO triaxial test results

Effective Stress
Global Stability Analysis
Shortly After Construction of Berms (1971)

FIGURE F17



DATE: JUNE 2009
PROJECT: 08-1111-0031

Golder Associates

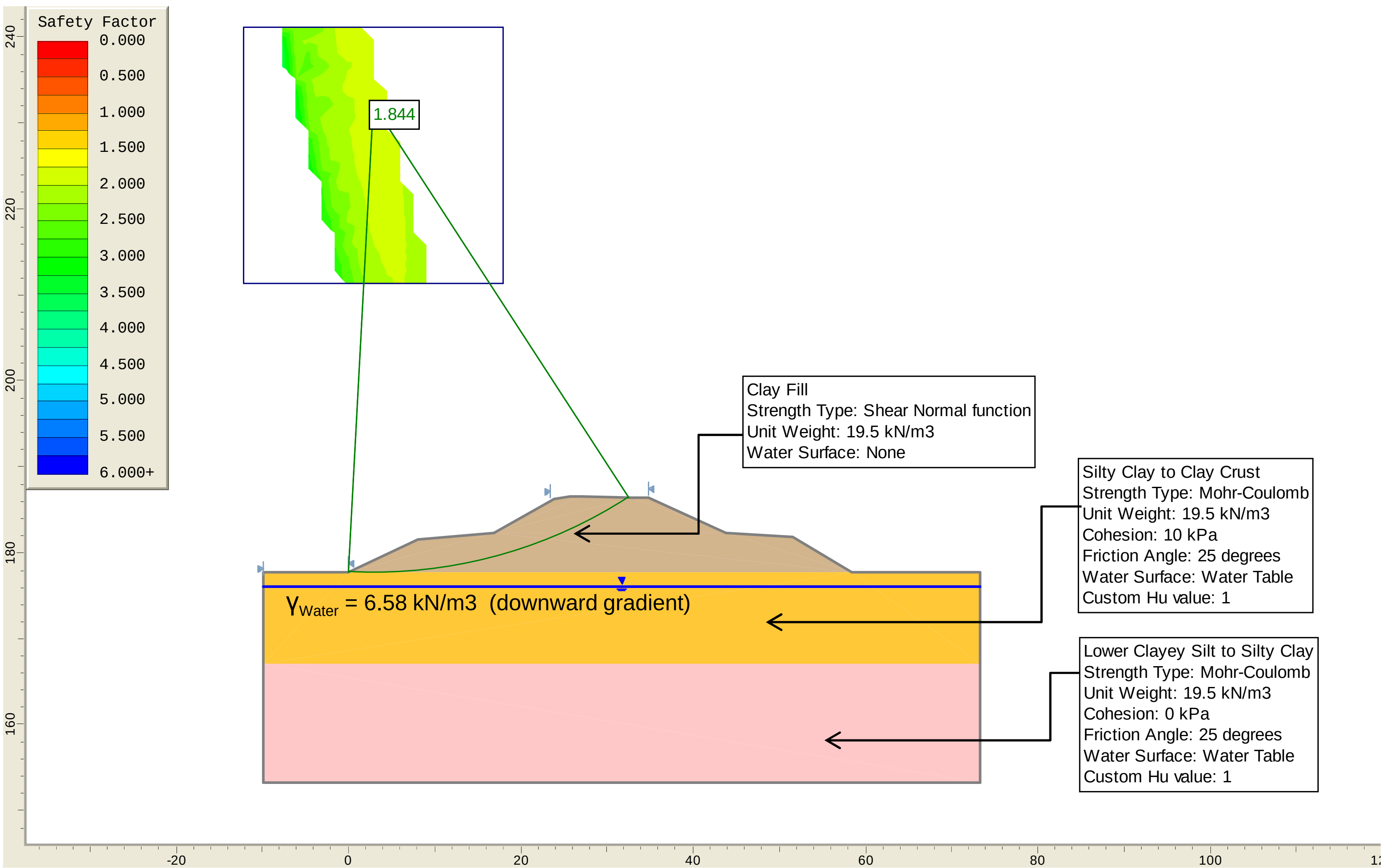
DWG: MWK
CHK: JPD

Project: 08-1111-0031 Drawn: MWK Reviewed: JPD Rev.: May 2009

Stability Analysis – 9.5 m High Embankment with Berms
08-1111-031 MTO/Hwy 140/Embankment
Strength of embankment fill given by fully defined strength envelope based on Golder and MTO Triaxial Test Results

Effective Stress
Long-Term Global Stability Analysis
After Construction of Berms (2008)

FIGURE F18



DATE: JUNE 2009

PROJECT: 08-1111-0031

Golder Associates

DWG: MWK

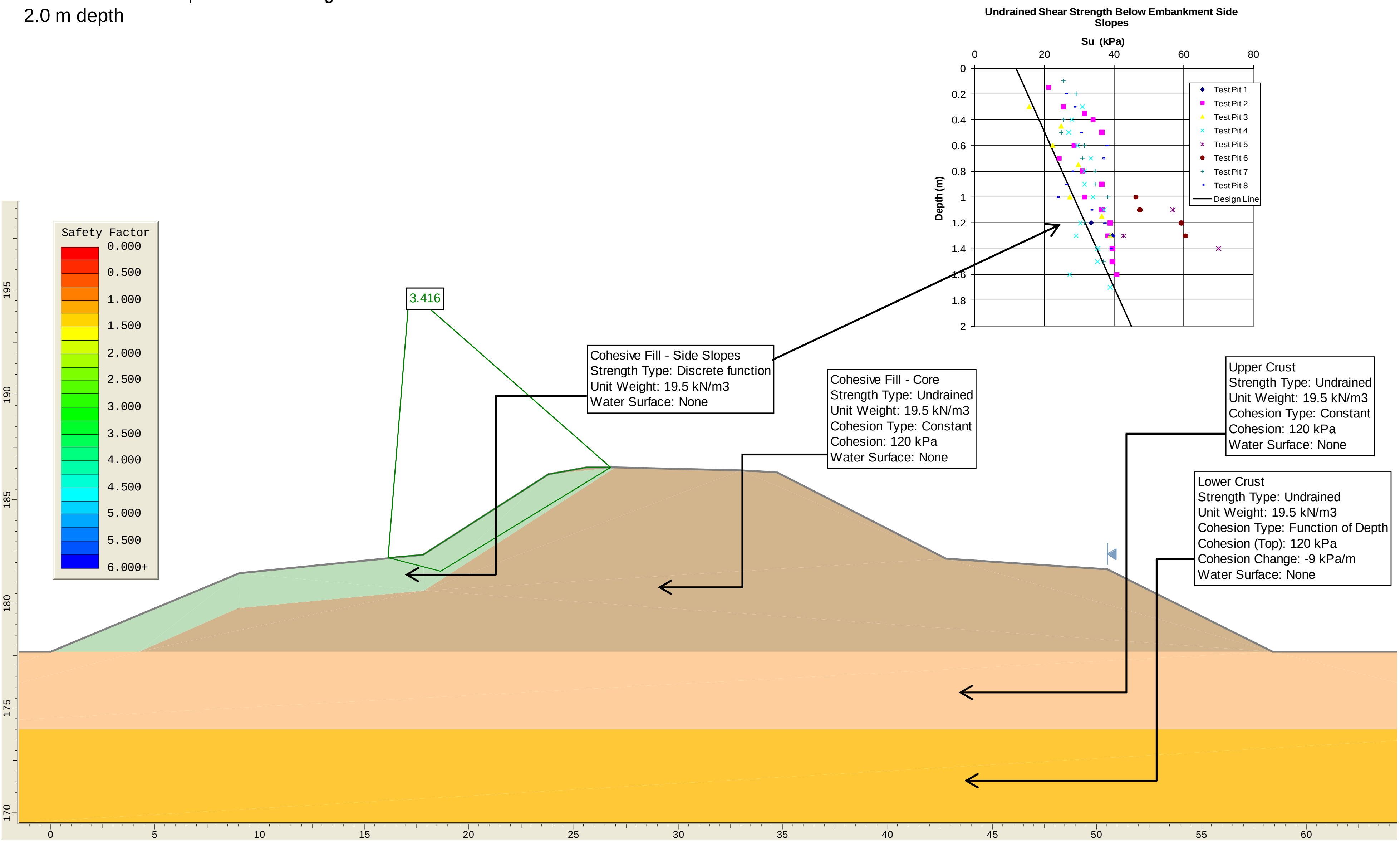
CHK: JPD

Project: 08-1111-0031 Drawn: MWK Reviewed: JPD Rev.: May 2009

Stability Analysis – 9.5 m High Embankment
08-1111-031 MTO/Hwy 140/Embankment
Cohesive fill in side slopes based on shear strength measured in side walls of test pits – 12 kPa at ground surface to 45 kPa at 2.0 m depth

Total Stress Surficial Stability Analysis After Construction of Berms

FIGURE F19



DATE: JUNE 2009

PROJECT: 08-1111-0031

Golder Associates

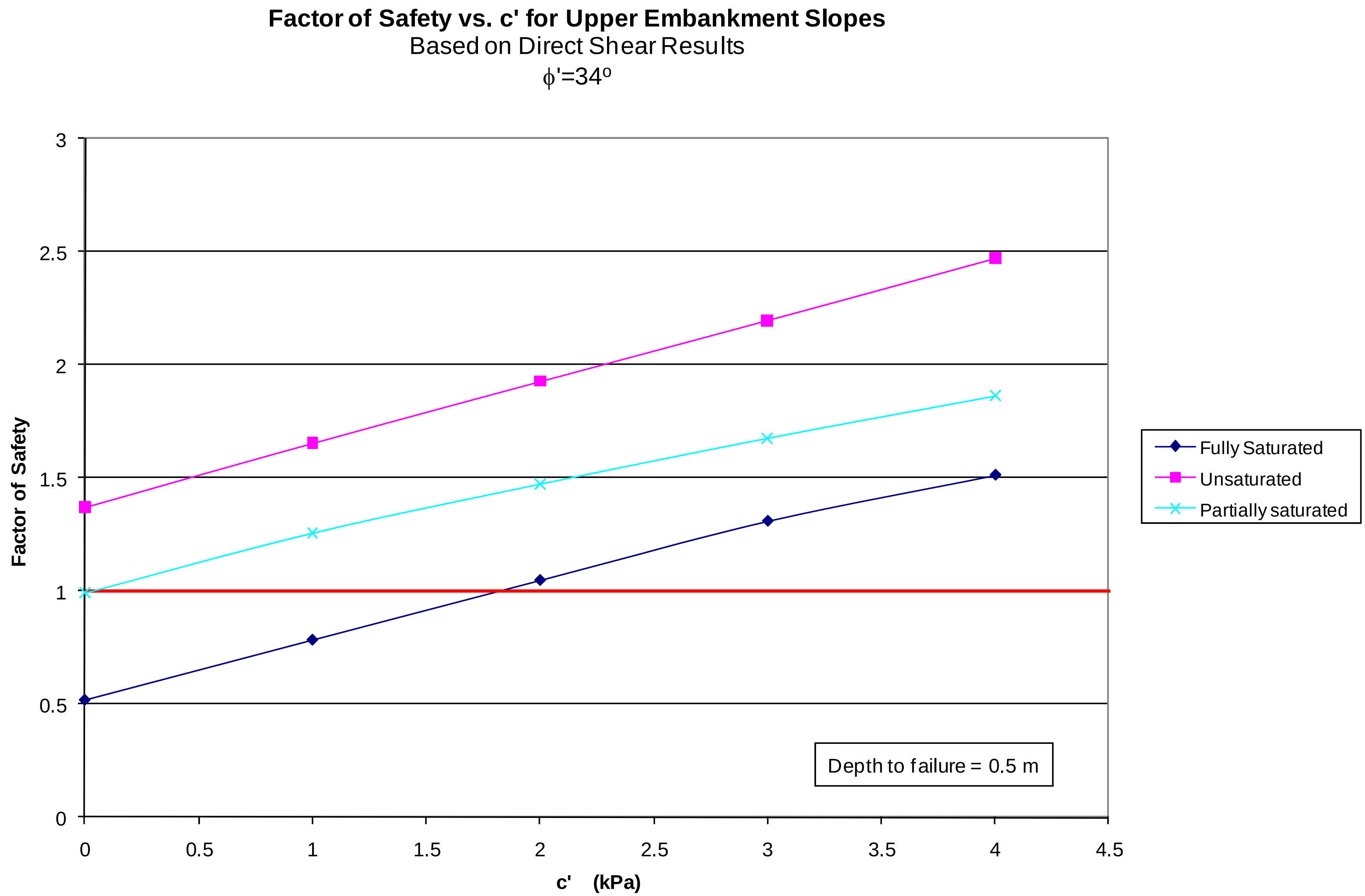
DWG: MWK

CHK: JPD

Project: 08-1111-0031 Drawn: MWK Reviewed: JPD Rev.: May 2009

Effective Stress
Surficial Stability Analysis Results

FIGURE F20



DATE: JUNE 2009

PROJECT: 08-1111-0031

Golder Associates

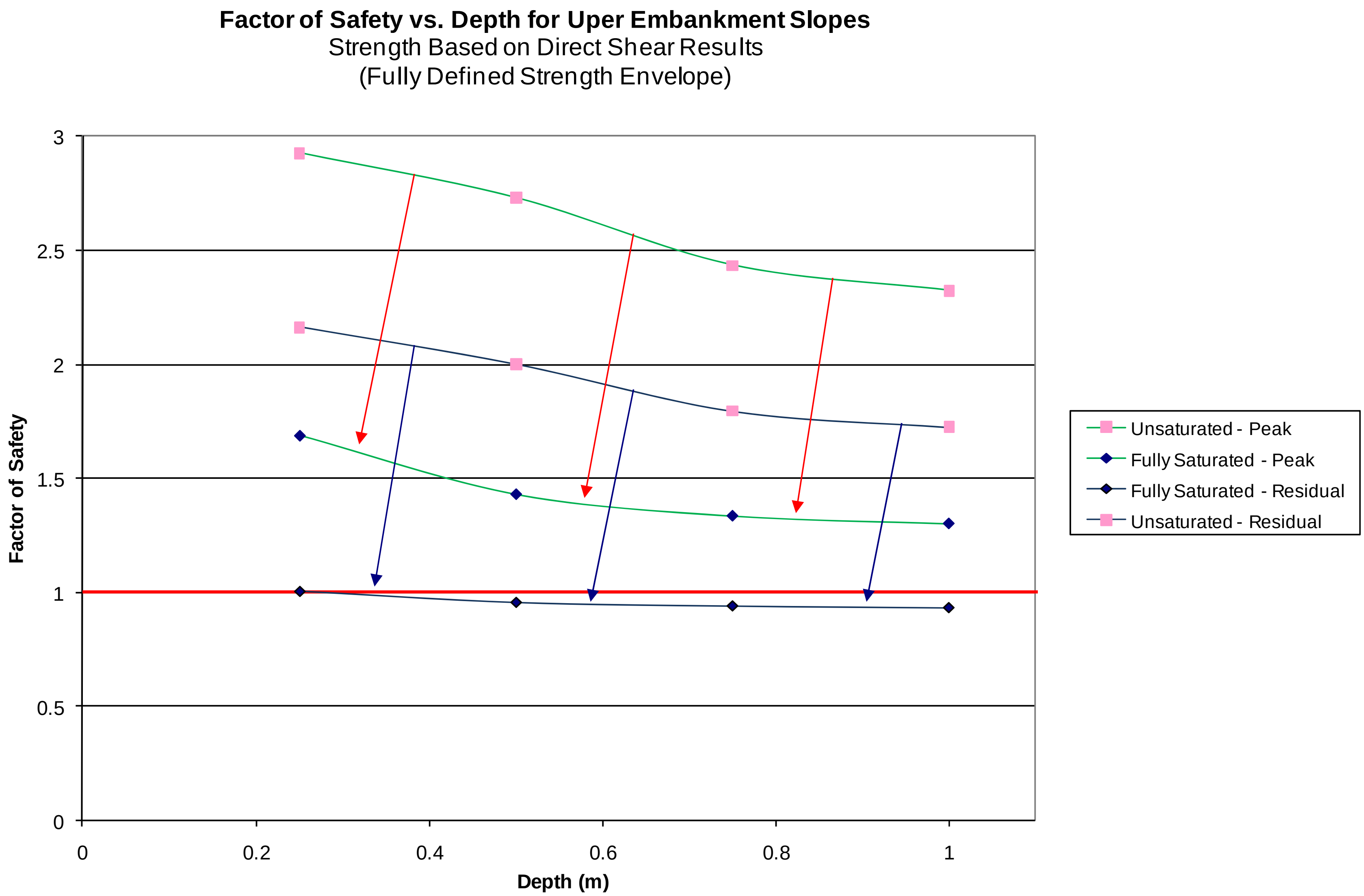
DWG: MWK

CHK: JPD

Project: 08-1111-0031 Drawn: MWK Reviewed: JPD Rev.: May 2009

Effective Stress
Surficial Stability Analysis

FIGURE F21



DATE: JUNE 2009

PROJECT: 08-1111-0031

Golder Associates

DWG: MWK

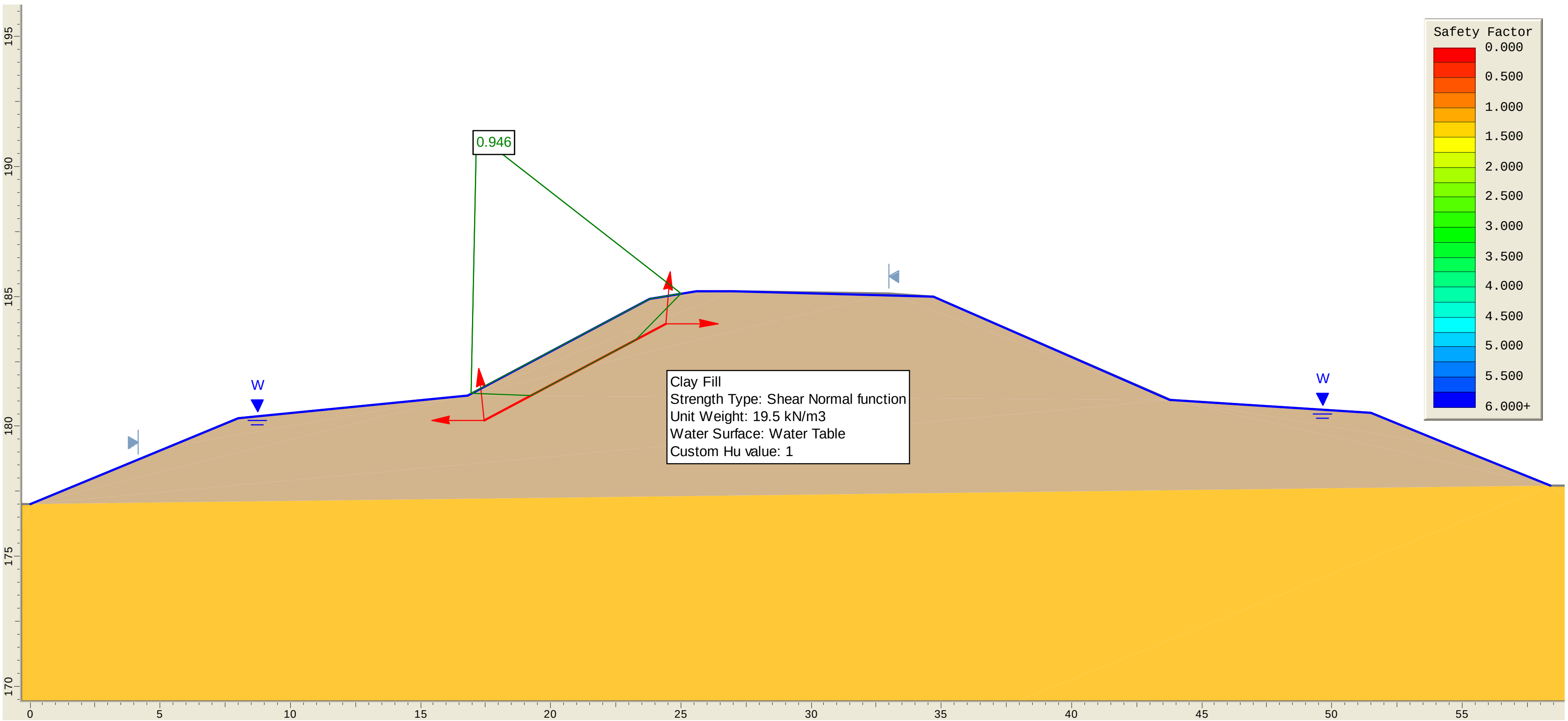
CHK: JPD

Project: 08-1111-0031 Drawn: MWK Reviewed: JPD Rev.: May 2009

Stability Analysis – 9.5 m High Embankment
08-1111-031 MTO/Hwy 140/Embankment
Strength of embankment fill given by fully defined strength envelope based on Golder direct shear test results.
1.0 m deep failure surface.

Effective Stress
Surficial Stability Analysis
Upper Slopes After Construction of Berms

FIGURE F22



DATE: JUNE 2009

PROJECT: 08-1111-0031

Golder Associates

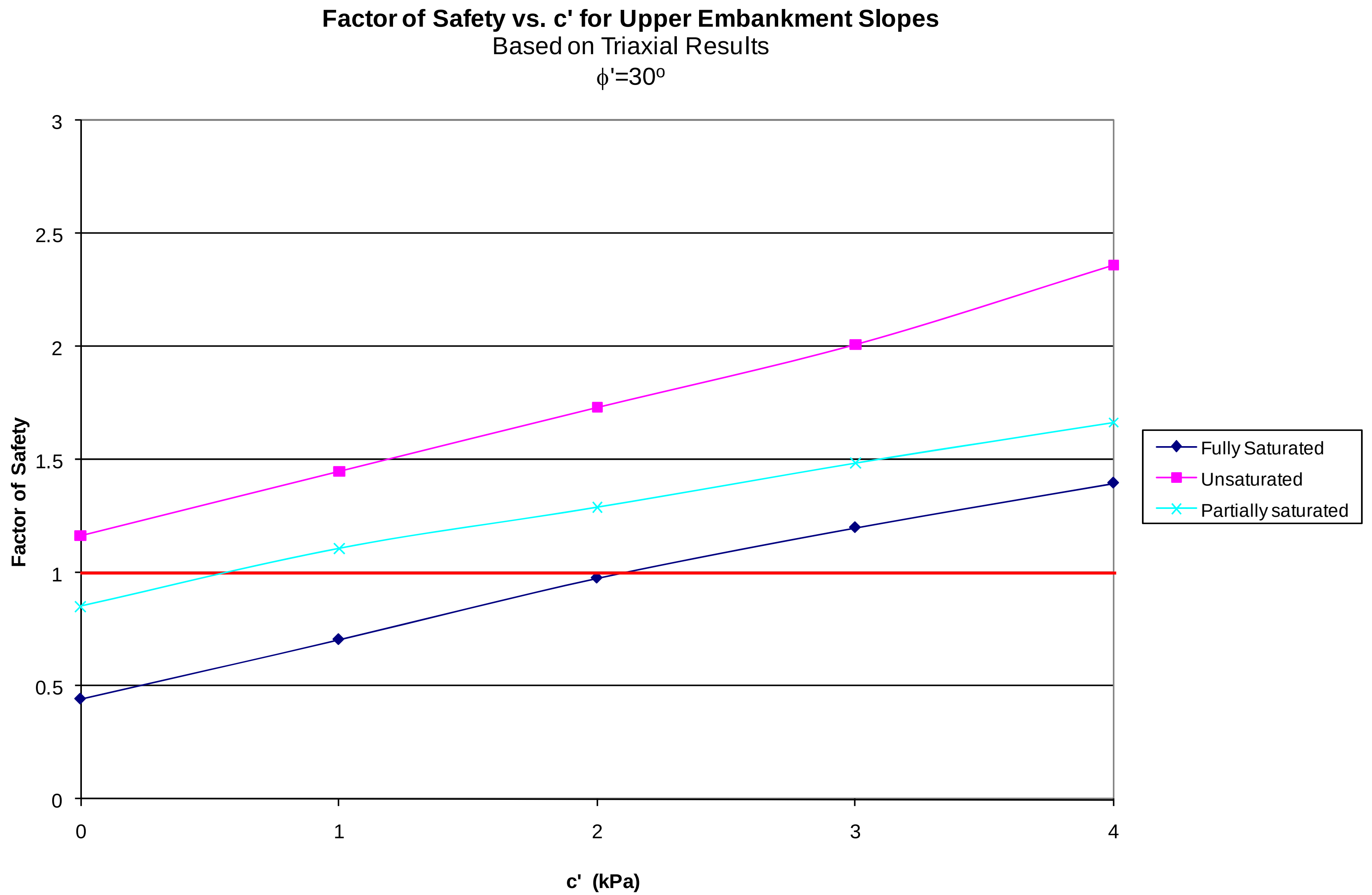
DWG: MWK

CHK: JPD

Project: 08-1111-0031 Drawn: MWK Reviewed: JPD Rev.: May 2009

Effective Stress
Surficial Stability Analysis

FIGURE F23



DATE: JUNE 2009

PROJECT: 08-1111-0031

Golder Associates

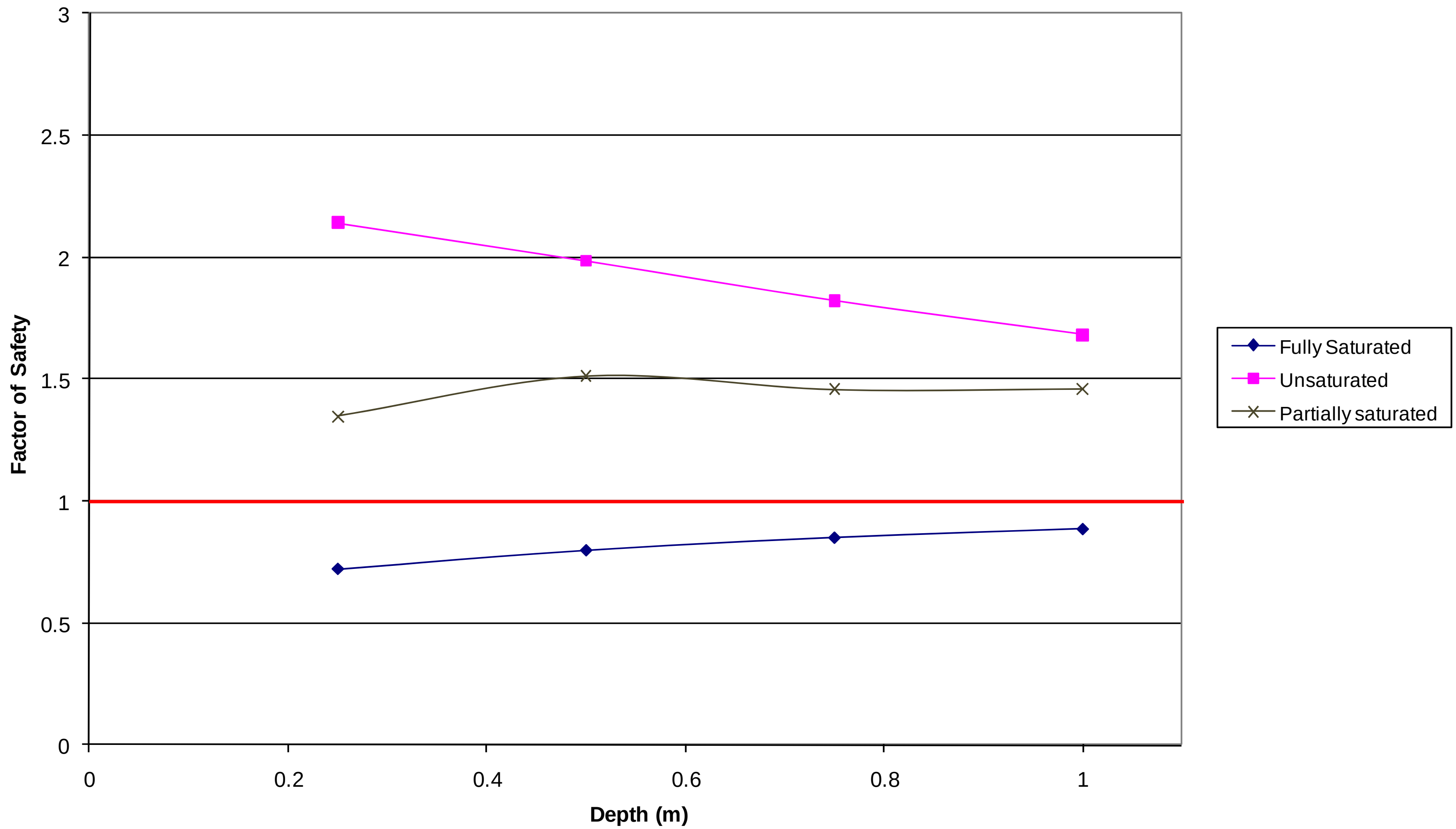
DWG: MWK

CHK: JPD

Effective Stress
Surficial Stability Analysis

FIGURE F24

Factor of Safety vs. Depth for Upper Embankment Slopes
Strength Based on Triaxial Results
(Fully Defined Strength Envelope)



DATE: JUNE 2009
PROJECT: 08-1111-0031

Golder Associates

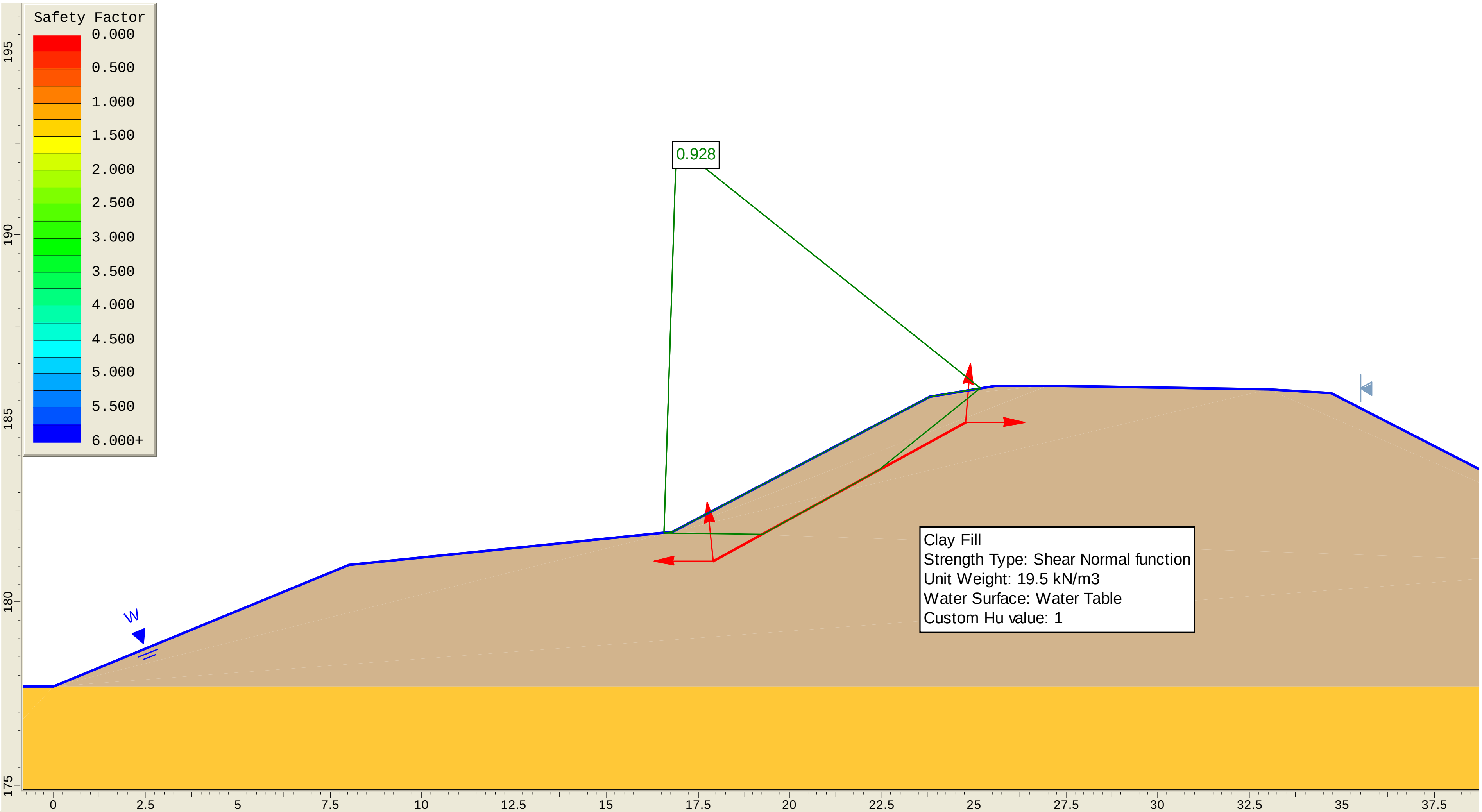
DWG: MWK
CHK: JPD

Project: 08-1111-0031 Drawn: MWK Reviewed: JPD Rev.: May 2009

Stability Analysis – 9.5 m High Embankment
08-1111-031 MTO/Hwy 140/Embankment
Strength of embankment fill given by fully defined strength envelope based on Golder and MTO Triaxial Test Results.
1.0 m deep failure surface

Effective Stress
Surficial Stability Analysis
After Construction of Berms

FIGURE F25



DATE: JUNE 2009

PROJECT: 08-1111-0031

Golder Associates

DWG: MWK

CHK: JPD

APPENDIX G

Remediation Option Drawings

PLOT DATE: August 13, 2009
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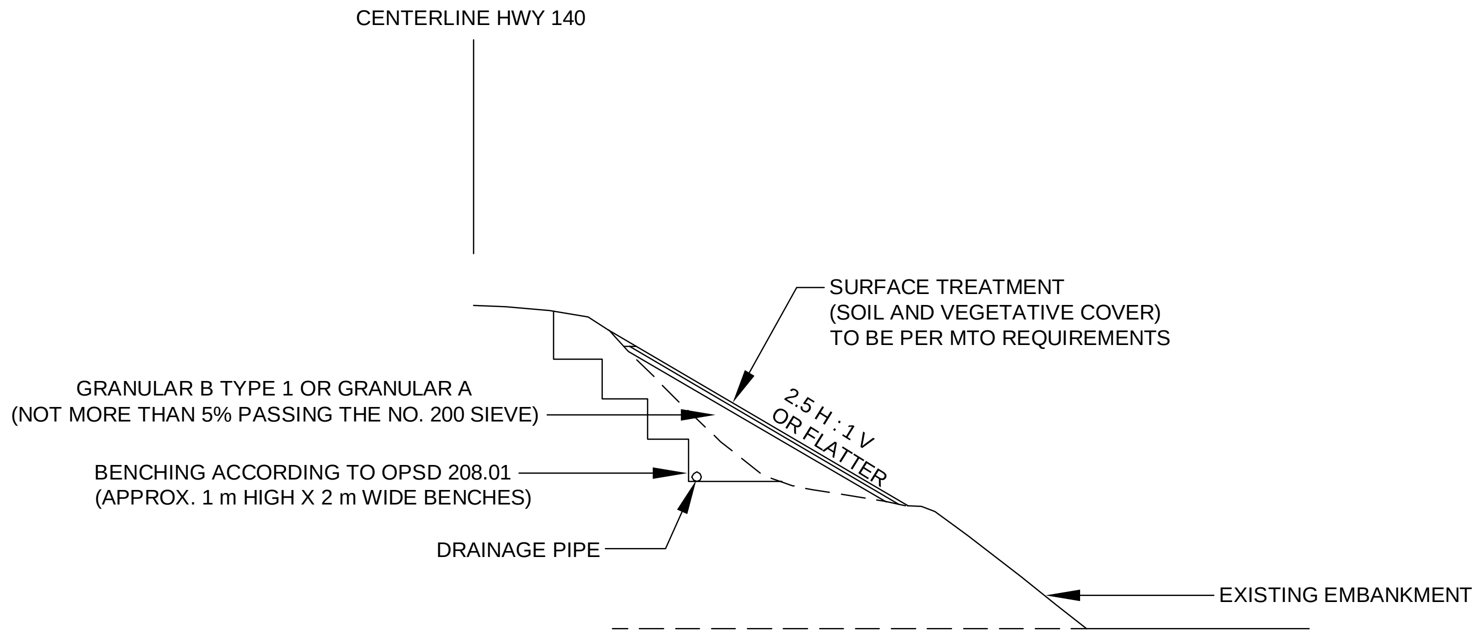

NOT TO SCALE

PROJECT
MINISTRY OF TRANSPORTATION ONTARIO
HIGHWAY 140/ CNR
NORTH EMBANKMENT AND APPROACH
PORT COLBORNE, ONTARIO

TITLE
SLOPE REMEDIATION DEEP BENCHING AND GRANULAR SLOPE FLATTENING

Golder Associates
Mississauga, Ontario, Canada

PROJECT No.		08-1111-0031	FILE No. 0811110031BA0G1.dwg
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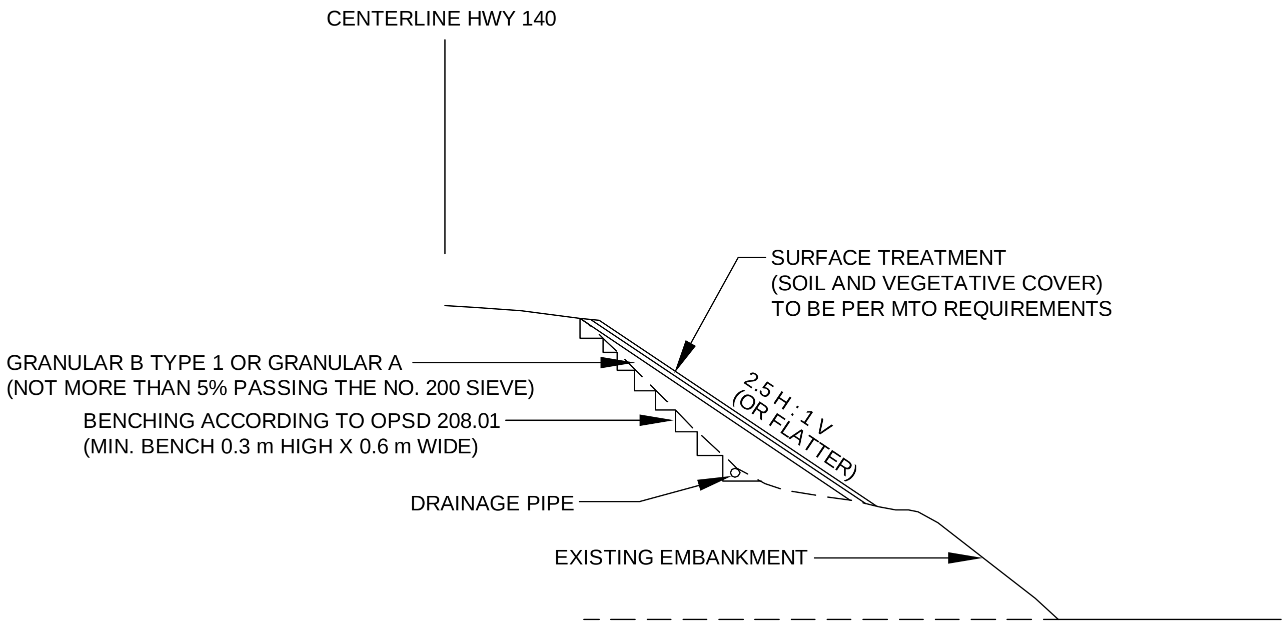
CHECK	MWK	Aug 13, 2009	G1
REVIEW	JPD	Aug 13, 2009	



NOT TO SCALE

PROJECT

MINISTRY OF TRANSPORTATION ONTARIO
HIGHWAY 140/ CNR NORTH
EMBANKMENT AND APPROACH
PORT COLBOURNE, ONTARIO

TITLE

SLOPE REMEDIATION STANDARD BENCHING AND AND GRANULAR SLOPE FLATTENING

Golder Associates
Mississauga, Ontario, Canada

PROJECT No.		08-1111-0031	FILE No. 0811110031BA0G2.dwg
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REVIEW	JPD	Aug 13, 2009	G2

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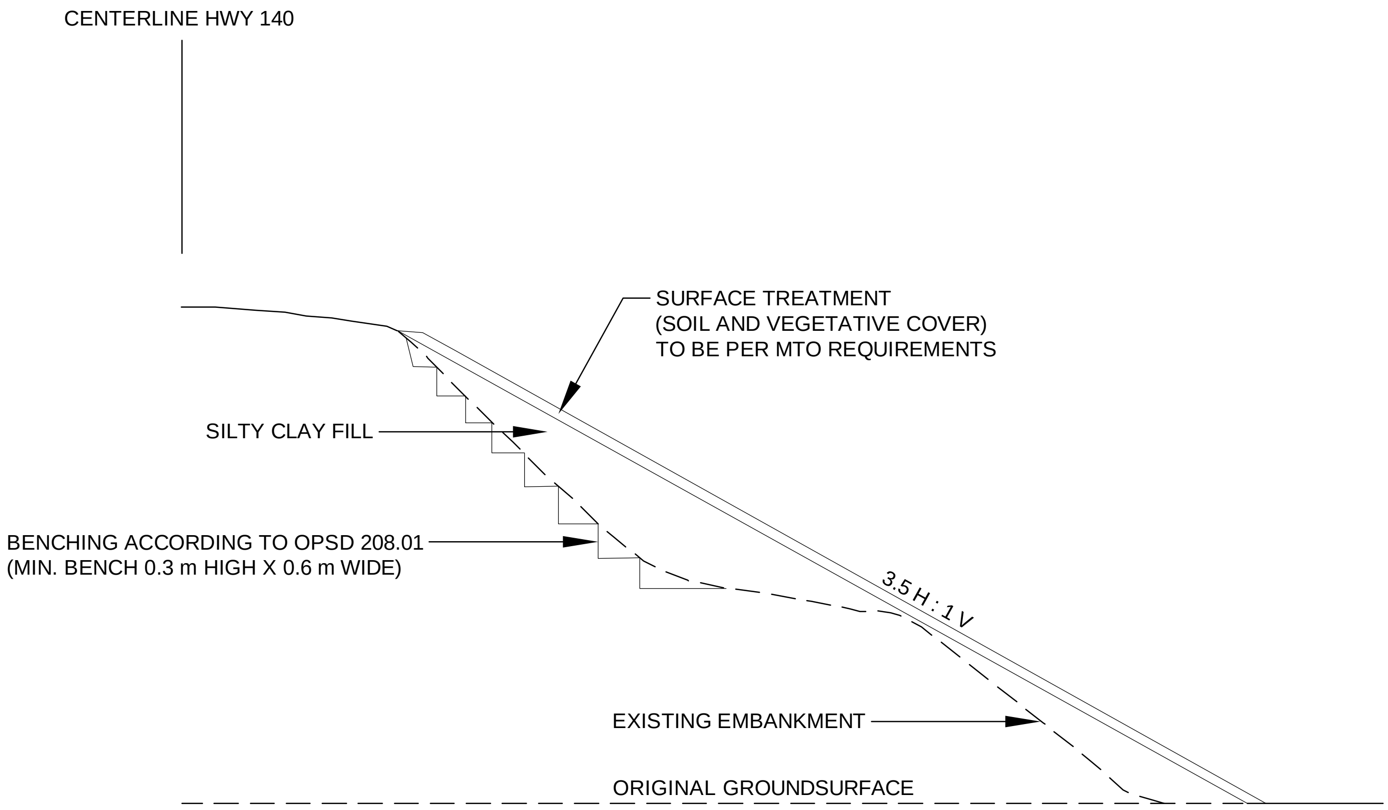

NOT TO SCALE

PROJECT
MINISTRY OF TRANSPORTATION ONTARIO
HIGHWAY 140/ CNR NORTH
EMBANKMENT AND APPROACH
PORT COLBOURNE, ONTARIO

TITLE
SLOPE REMEDIATION
SLOPE FLATTENING
WITH SILTY CLAY

Golder Associates
Mississauga, Ontario, Canada

PROJECT No.		08-1111-0031	FILE No. 0811110031BA0G3.dwg
DESIGN			SCALE AS SHOWN REV. A
CAD	DD	Aug 13, 2009	DRAWING No.
CHECK	MWK	Aug 13, 2009	**G3**
REVIEW	JPD	Aug 13, 2009	

PLOT DATE: August 14, 2009
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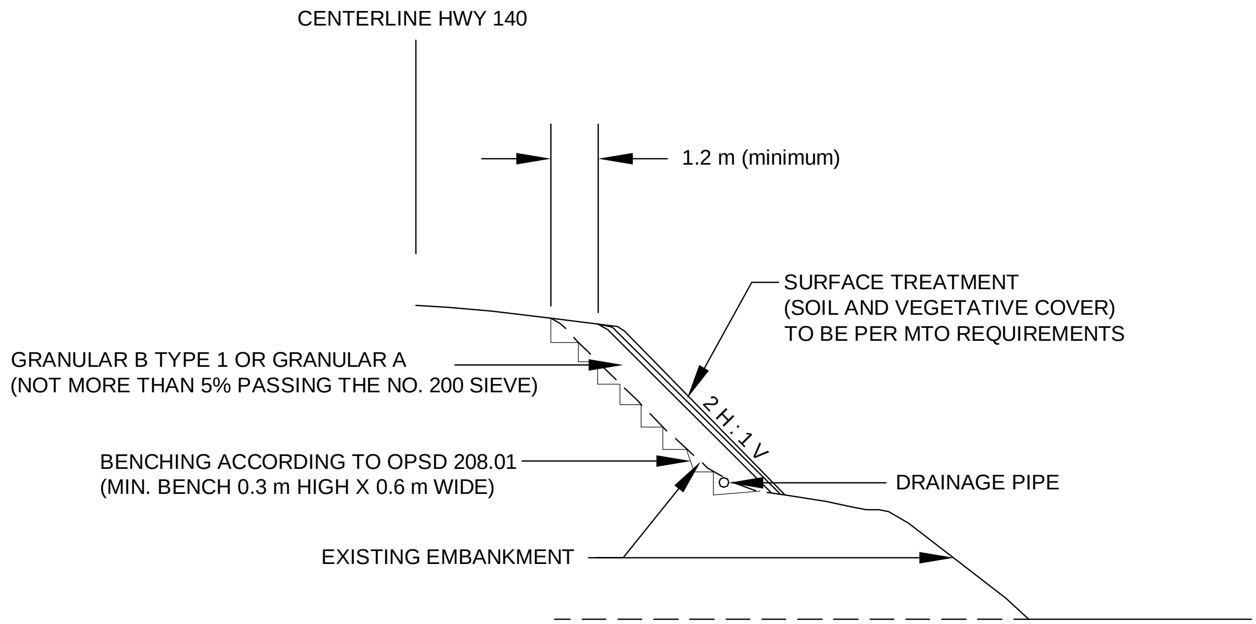

NOT TO SCALE

PROJECT

MINISTRY OF TRANSPORTATION ONTARIO
HIGHWAY 140/ CNR NORTH
EMBANKMENT AND APPROACH
PORT COLBOURNE, ONTARIO

TITLE

SLOPE REMEDIATION GRANULAR BLANKET AT 2H:1V (WITHOUT SLOPE FLATTENING)

Golder Associates
Mississauga, Ontario, Canada

PROJECT No.		08-1111-0031	FILE No. 0811110031BA0G4.dwg
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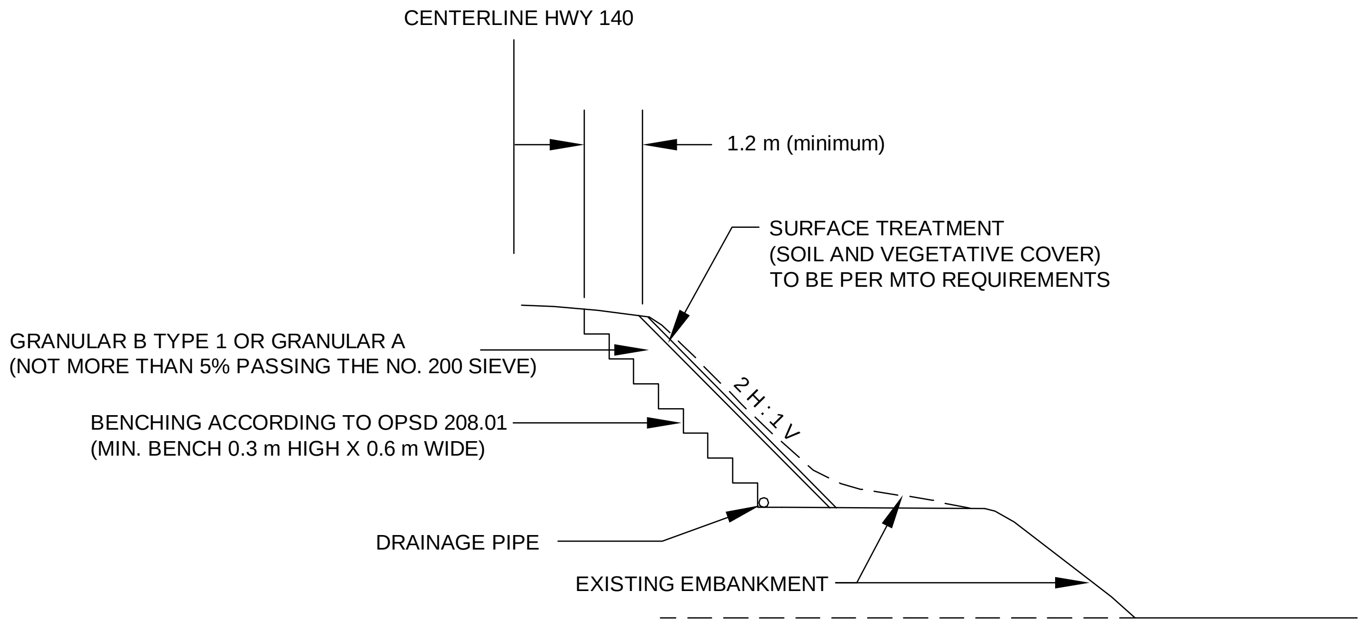
REVIEW	JPD	Aug 14, 2009	



NOTE:

PARTIAL REMOVAL TO BE CARRIED OUT VIA STAGED
EXCAVATION IN STRIPS OF LIMITED WIDTH

NOT TO SCALE

PROJECT

MINISTRY OF TRANSPORTATION ONTARIO
HIGHWAY 140/ CNR NORTH
EMBANKMENT AND APPROACH
PORT COLBOURNE, ONTARIO

TITLE

SLOPE REMEDIATION
PARTIAL REMOVAL AND REPLACEMENT WITH GRANULAR FILL AT 2H:1V (WITHOUT SLOPE FLATTENING)

Golder Associates
Mississauga, Ontario, Canada

PROJECT No.		08-1111-0031	FILE No. 0811110031BA0G5.dwg
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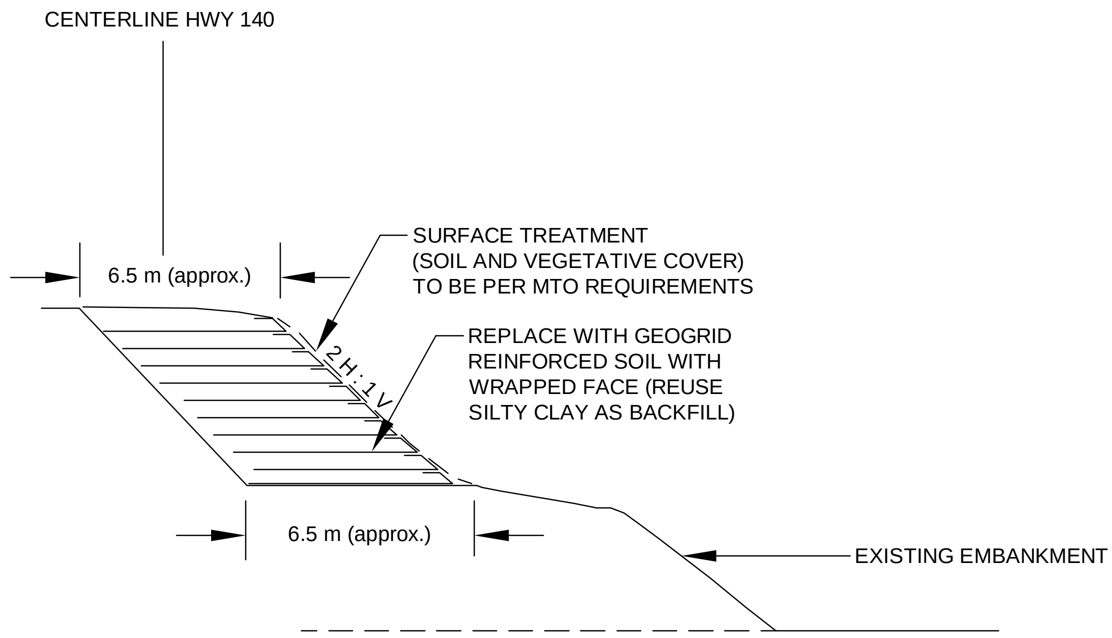

NOTE:

PARTIAL REMOVAL TO BE CARRIED OUT VIA STAGED EXCAVATION IN STRIPS OF LIMITED WIDTH

NOT TO SCALE

PROJECT
MINISTRY OF TRANSPORTATION ONTARIO
HIGHWAY 140/ CNR NORTH
EMBANKMENT AND APPROACH
PORT COLBOURNE, ONTARIO

TITLE
SLOPE REMEDIATION
PARTIAL SUB-EXCAVATION AND RECONSTRUCTION WITH GEOGRID

Golder Associates
Mississauga, Ontario, Canada

PROJECT No.		08-1111-0031	FILE No. 0811110031BA0G6.dwg
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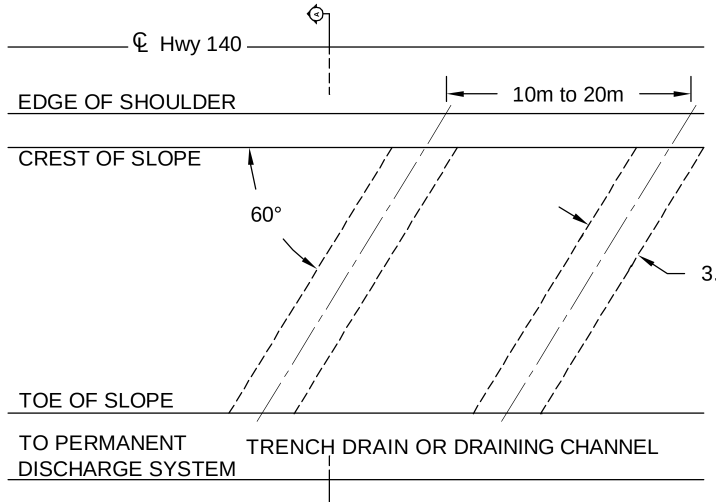
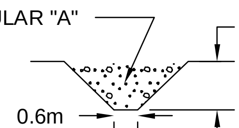
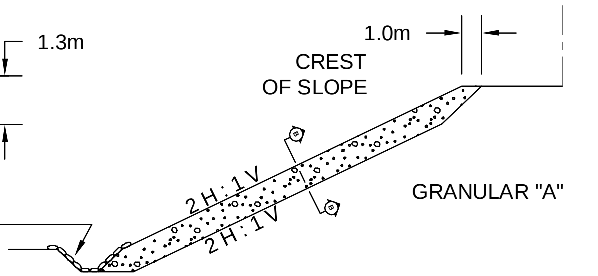


PLAN



SECTION A-A



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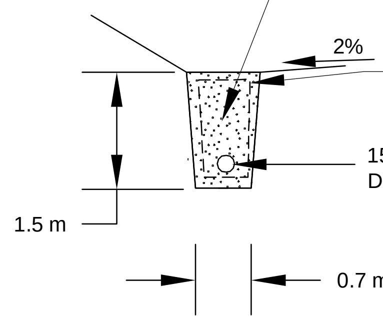

NOT TO SCALE

PROJECT

MINISTRY OF TRANSPORTATION ONTARIO
HIGHWAY 140/ CNR NORTH
EMBANKMENT AND APPROACH
PORT COLBOURNE, ONTARIO

TITLE

SLOPE REMEDIATION
COUNTERFORT DRAINS

Golder Associates
Mississauga, Ontario, Canada

PROJECT No.		08-1111-0031	FILE No. 0811110031BA0G7.dwg
DESIGN			SCALE AS SHOWN / REV. A
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PLOT DATE: August 13, 2009
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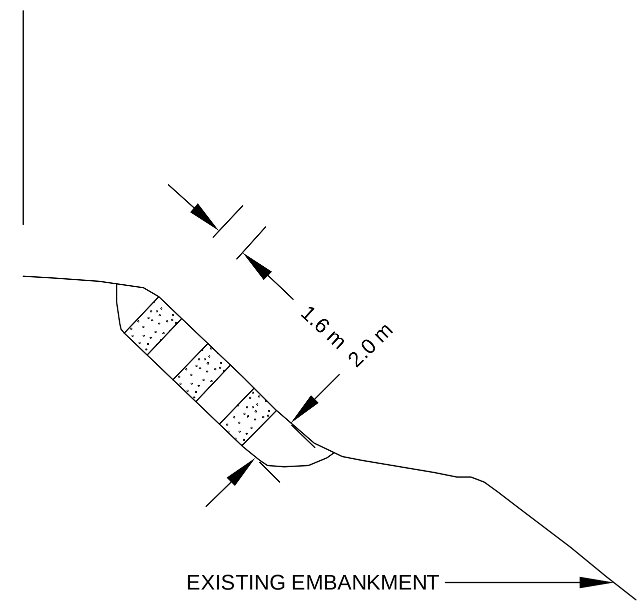

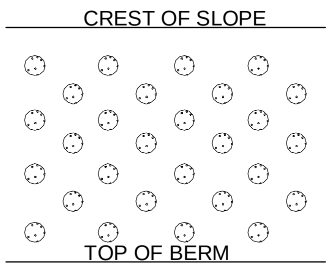

SOIL MIXING COLUMNS LAID
OUT ON REGULAR GRID

NOT TO SCALE

PROJECT

MINISTRY OF TRANSPORTATION ONTARIO
HIGHWAY 140/ CNR NORTH
EMBANKMENT AND APPROACH
PORT COLBOURNE, ONTARIO

TITLE

SLOPE REMEDIATION
CEMENT-SOIL MIXING
(DISCRETE COLUMNS ON GRID)

Golder Associates
Mississauga, Ontario, Canada

PROJECT No.		08-1111-0031	FILE No. 0811110031BA0G8.dwg
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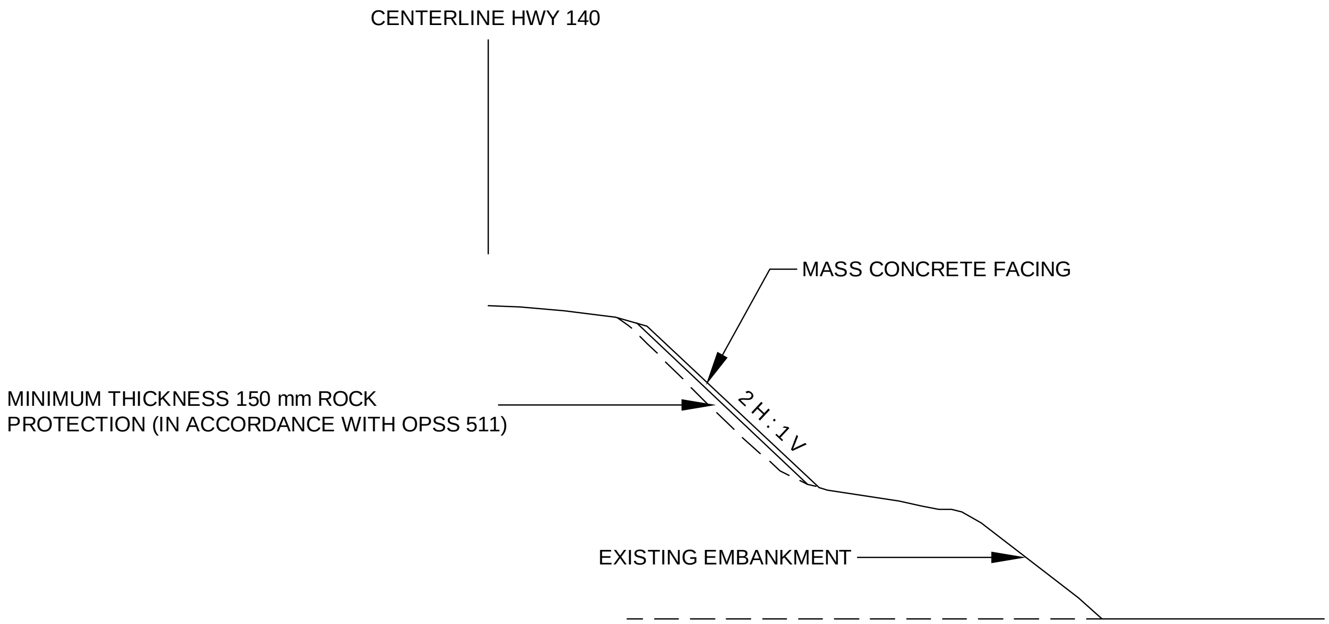

NOT TO SCALE

PROJECT

MINISTRY OF TRANSPORTATION ONTARIO
HIGHWAY 140/ CNR NORTH
EMBANKMENT AND APPROACH
PORT COLBOURNE, ONTARIO

TITLE

SLOPE REMEDIATION
SLOPE COVER WITH ROCK PROTECTION AND MASS CONCRETE

Golder Associates
Mississauga, Ontario, Canada

PROJECT No.		08-1111-0031
DESIGN		
CAD	DD	Aug 14, 2009
CHECK	MWK	Aug 14, 2009
REVIEW	JPD	Aug 14, 2009

FILE No. 0811110031BA0G9.dwg
SCALE AS SHOWN REV. A
DRAWING No.
G9

APPENDIX H

Details of Preferred Remediation Option

PLOT DATE: August 13, 2009
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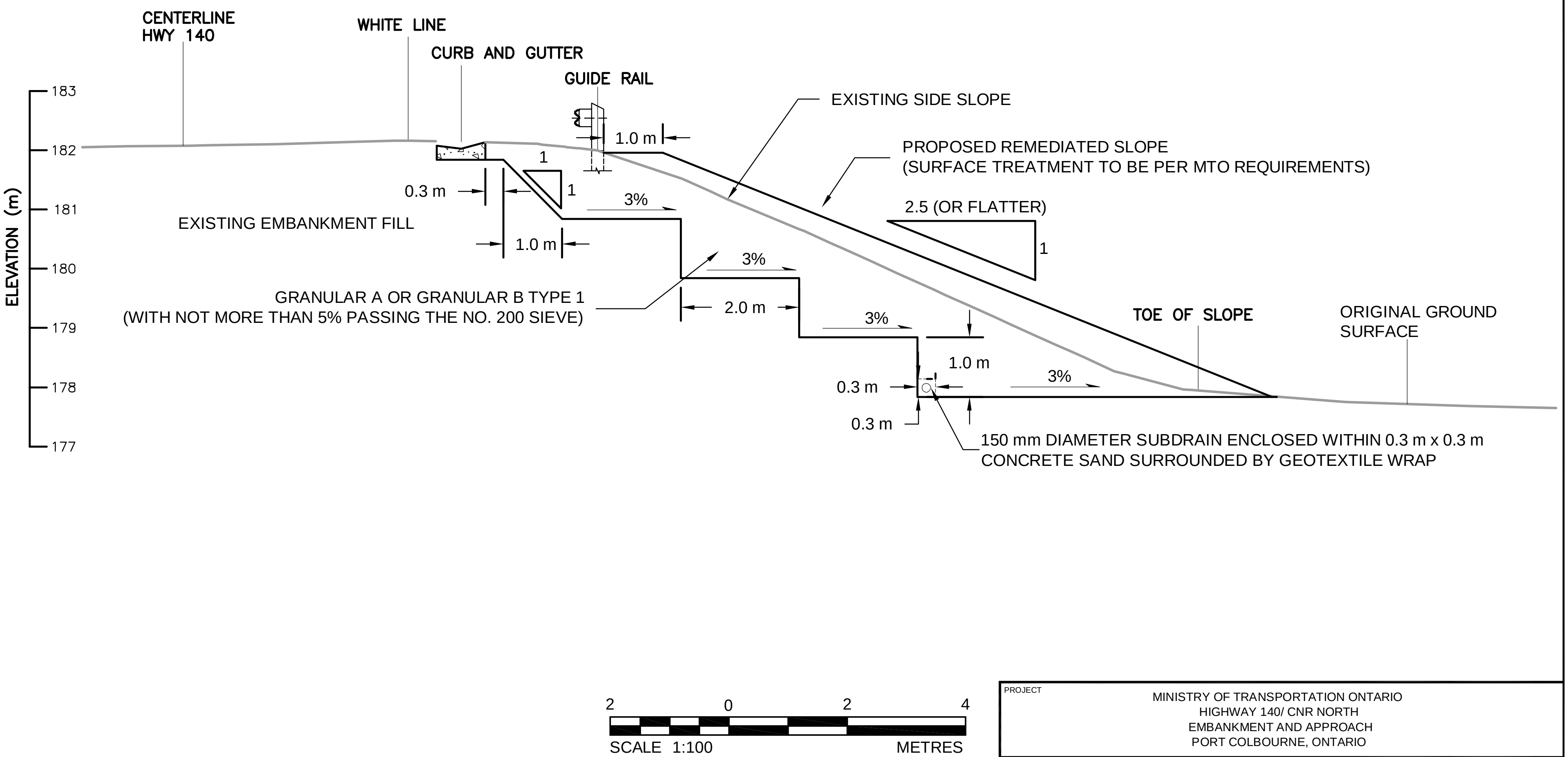

PROJECT	
MINISTRY OF TRANSPORTATION ONTARIO HIGHWAY 140/ CNR NORTH EMBANKMENT AND APPROACH PORT COLBOURNE, ONTARIO	

TITLE

DETAILS OF PROPOSED REMEDIATION CROSS SECTION D - D' STATION 0+231

Golder Associates
Mississauga, Ontario, Canada

PROJECT No.		08-1111-0031	FILE No. 0811110031B0H1.dwg
DESIGN			SCALE AS SHOWN REV. A
CAD	DD	Aug. 13, 2009	DRAWING No.
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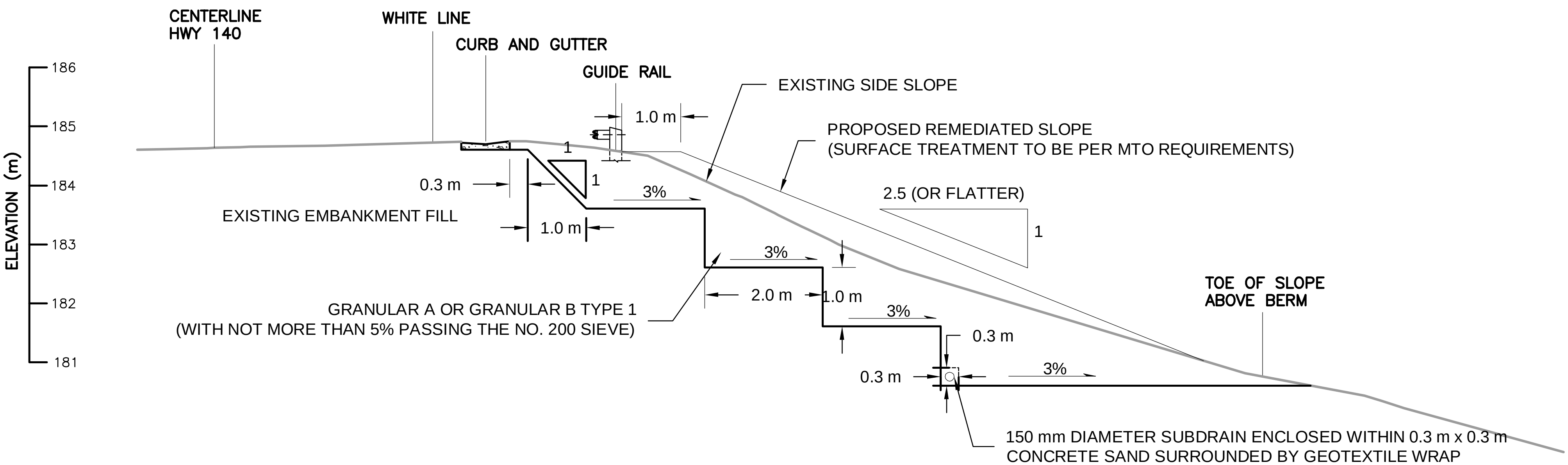

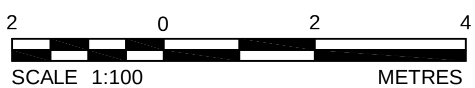

PROJECT

MINISTRY OF TRANSPORTATION ONTARIO
HIGHWAY 140/ CNR NORTH
EMBANKMENT AND APPROACH
PORT COLBOURNE, ONTARIO

TITLE

DETAILS OF PROPOSED REMEDIATION
CROSS SECTION E - E'
STATION 0+140

Golder Associates
Mississauga, Ontario, Canada

PROJECT No.		08-1111-0031	FILE No. 0811110031B0H2.dwg
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PLOT DATE: August 13, 2009
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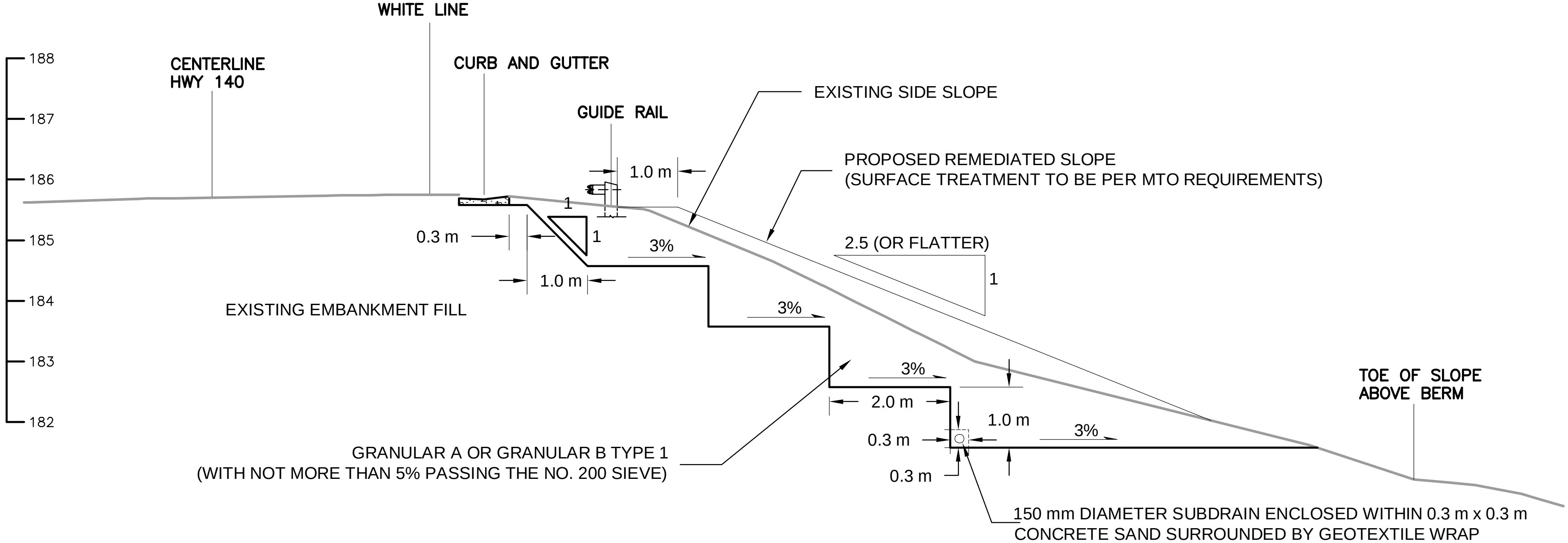

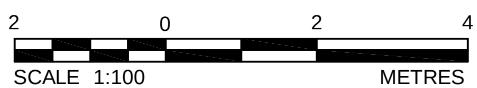

PROJECT

MINISTRY OF TRANSPORTATION ONTARIO
HIGHWAY 140/ CNR NORTH
EMBANKMENT AND APPROACH
PORT COLBOURNE, ONTARIO

TITLE

DETAILS OF PROPOSED REMEDIATION CROSS SECTION F - F' STATION 0+095

Golder Associates
Mississauga, Ontario, Canada

PROJECT No.		08-1111-0031
DESIGN		
CAD	DD	Aug. 13, 2009
CHECK	MWK	Aug. 13, 2009
REVIEW	JPD	Aug. 13, 2009

FILE No. 0811110031B0H3.dwg
SCALE AS SHOWN | REV. A
DRAWING No.
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PLOT DATE: August 12, 2009
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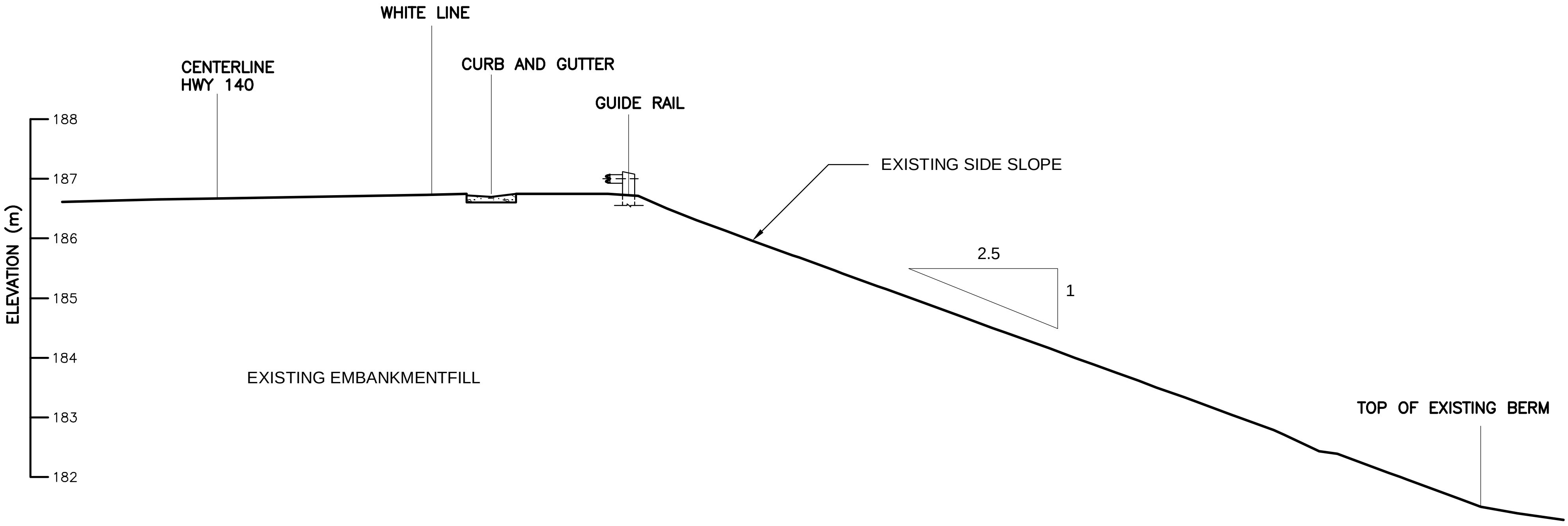
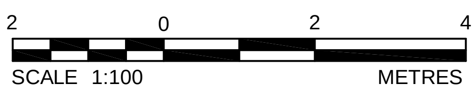


NOTE: NO GRANULAR SLOPE FLATTENING
REQUIRED AT THIS SECTION



PROJECT

MINISTRY OF TRANSPORTATION ONTARIO
HIGHWAY 140/ CNR NORTH
EMBANKMENT AND APPROACH
PORT COLBOURNE, ONTARIO

TITLE

DETAILS OF PROPOSED REMEDIATION
CROSS SECTION G - G'
STATION 0+041

Golder Associates
Mississauga, Ontario, Canada

PROJECT No.		08-1111-0031
DESIGN		
CAD	DD	Aug. 12, 2009
CHECK	MWK	Aug. 12, 2009
REVIEW	JPD	Aug. 12, 2009

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DRAWING No.

H4

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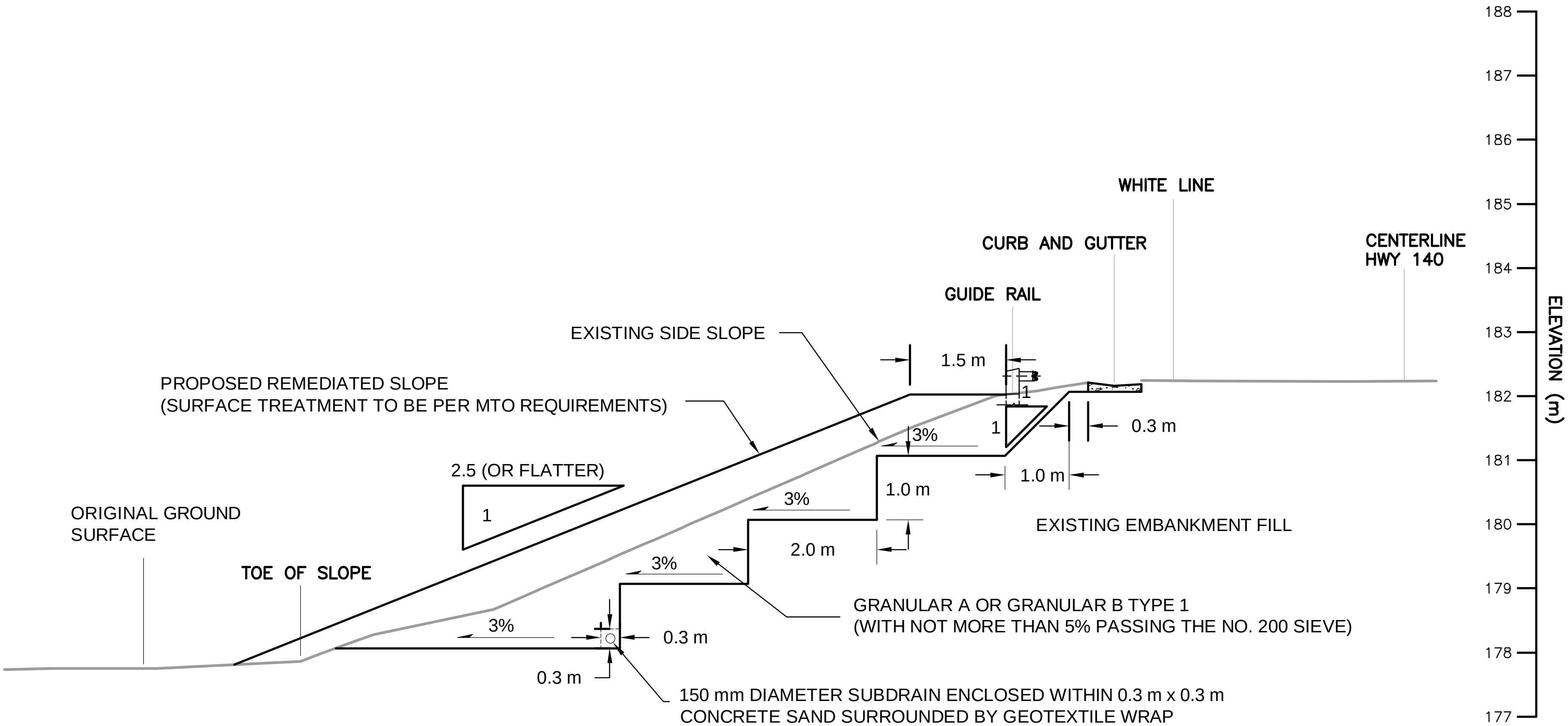

2 0 2 4
SCALE 1:100 METRES

PROJECT
MINISTRY OF TRANSPORTATION ONTARIO
HIGHWAY 140/ CNR NORTH
EMBANKMENT AND APPROACH
PORT COLBOURNE, ONTARIO

TITLE
DETAILS OF PROPOSED REMEDIATION CROSS SECTION H-H' STATION 0+226

Golder Associates
Mississauga, Ontario, Canada

PROJECT No.		08-1111-0031	FILE No. 0811110031B0H5.dwg
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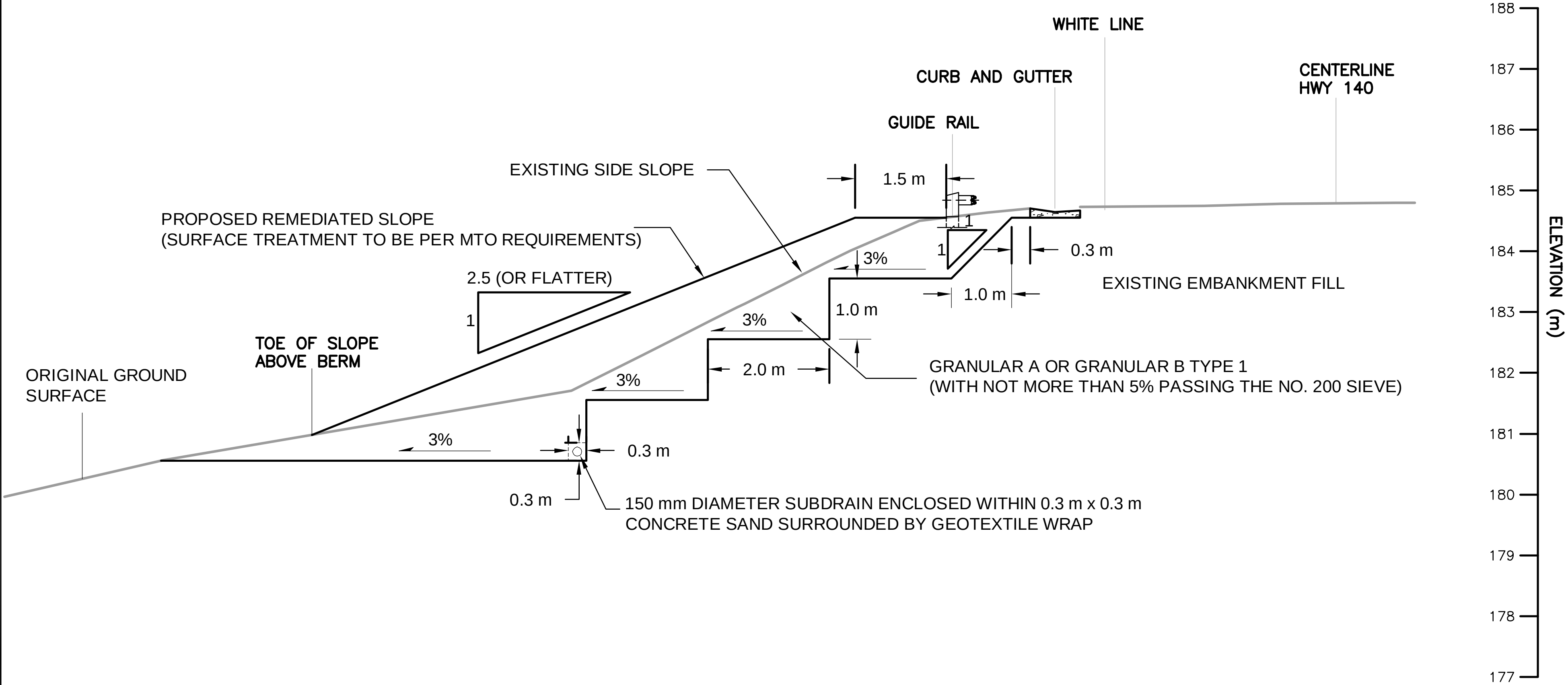

2 0 2 4

SCALE 1:100 METRES

PROJECT	MINISTRY OF TRANSPORTATION ONTARIO HIGHWAY 140/ CNR NORTH EMBANKMENT AND APPROACH PORT COLBOURNE, ONTARIO
TITLE	**DETAILS OF PROPOSED REMEDIATION CROSS SECTION I-I' STATION 0+135**

Golder Associates
Mississauga, Ontario, Canada

PROJECT No.		08-1111-0031	FILE No. 0811110031B0H6.dwg	
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PLOT DATE: August 14, 2009
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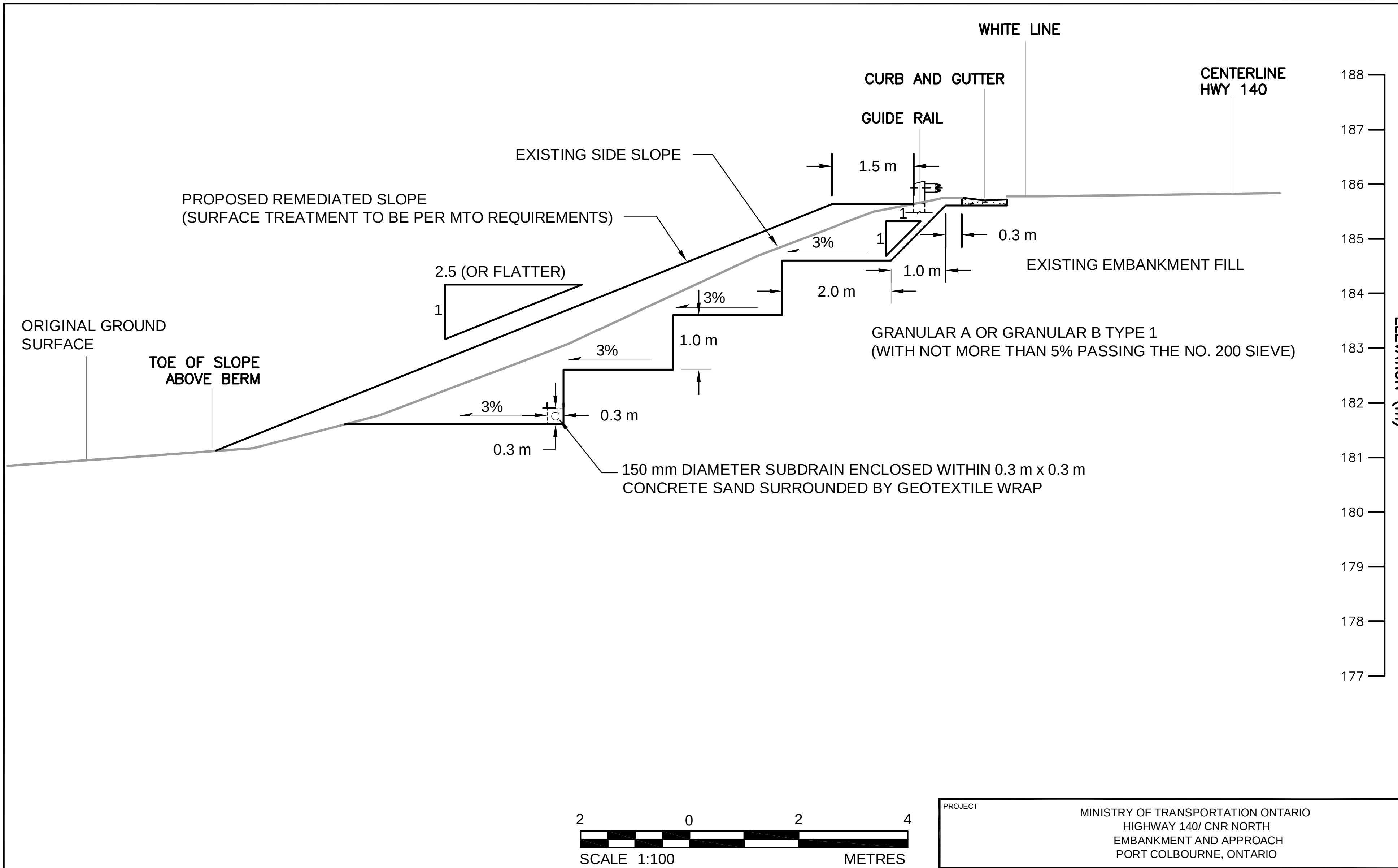


PROJECT

MINISTRY OF TRANSPORTATION ONTARIO
HIGHWAY 140/ CNR NORTH
EMBANKMENT AND APPROACH
PORT COLBOURNE, ONTARIO

TITLE

DETAILS OF PROPOSED REMEDIATION CROSS SECTION J-J' STATION 0+090

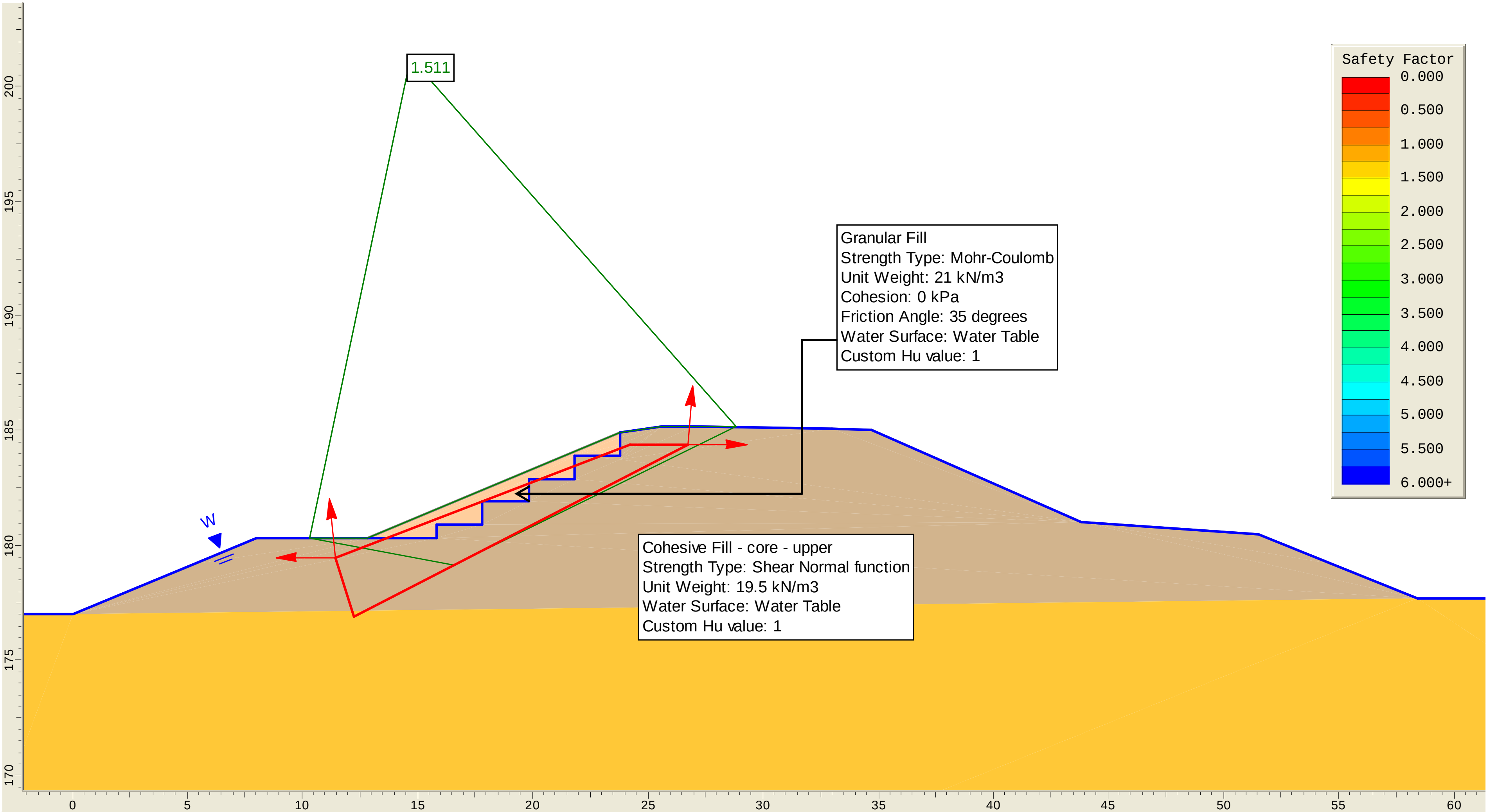
Golder Associates
Mississauga, Ontario, Canada

PROJECT No.		08-1111-0031	FILE No. 0811110031B0H7.dwg	
DESIGN			SCALE AS SHOWN	REV. A
CAD	DD	Aug. 14, 2009	DRAWING No.	
CHECK	MWK	Aug. 14, 2009	H7	
REVIEW	JPD	Aug. 14, 2009		

Stability Analysis – 9.5 m High Embankment
08-1111-031 MTO/Hwy 140/Embankment
Strength of embankment fill given by fully defined strength envelope based on Golder and MTO triaxial test results

Surficial Stability Analysis – Remediation Option #1

FIGURE H8



DATE: MAY 2009

PROJECT: 08-1111-0031

Golder Associates

DWG: MWK

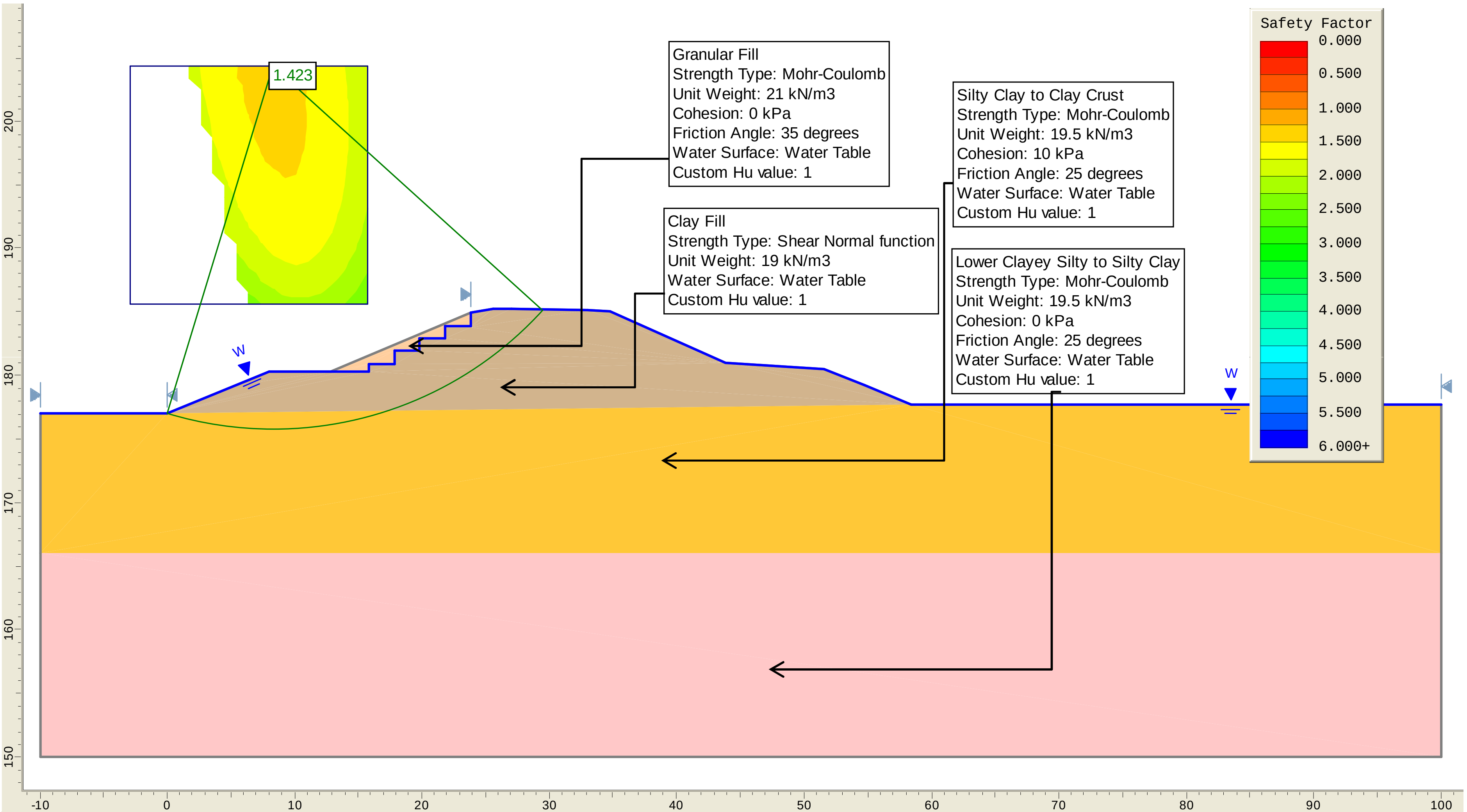
CHK: JPD

Project: 08-1111-0031 Drawn: MWK Reviewed: JPD Rev.: May 2009

Stability Analysis – 9.5 m High Embankment
08-1111-031 MTO/Hwy 140/Embankment
Strength of embankment fill given by fully defined strength envelope based on Golder and MTO triaxial test results

Global Stability Analysis – Remediation Option #1

FIGURE H9



DATE: MAY 2009

PROJECT: 08-1111-0031

DWG: MWK

CHK: JPD

Africa	+ 27 11 254 4800
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North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

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