FOUNDATION INVESTIGATION
AND DESIGN REPORT
DIVISION STREET W-N/S RAMP EMBANKMENT
HIGHWAY 401 WIDENING FROM WEST OF
SYDENHAM ROAD TO WEST OF MONTREAL STREET
KINGSTON, ONTARIO
W.P. 77-99-01

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PART A

FOUNDATION INVESTIGATION REPORT
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Borehole Number	***Borehole Location***	***MTM NAD83 Northing (m)***	***MTM NAD83 Easting (m)***	***Ground Surface Elevation (m)***
E-1	Embankment Toe	4,903,147.7	304,677.9	80.0
E-2	Embankment Toe	4,903,122.1	304,698.3	80.1
E-2B	Embankment Toe	4,903,099.3	304,714.1	79.9
E-3	Shoulder of Ramp	4,903,128.3	304,706.6	82.5

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services for the detail design of the Highway 401 widening from four to six lanes, from west of Sydenham Road to west of Montreal Street in the City of Kingston, Ontario. Foundation engineering services are required for the following components under W.P. 77-99-01:

- northward widening of the existing Division Street overpass structure;
- investigation of instability and settlement along a section of the Division Street W-N/S Ramp;
- widening of high fill embankments in the vicinity of Little Cataraqui Creek, between Sydenham Road and Sir John A. MacDonald Boulevard;
- overhead signs; and
- trenchless sewer installation.

This report addresses the investigation of instability and settlement along the Division Street W-N/S Ramp between approximately Stations 23+840 and 23+905.

The terms of reference for the original scope of work are outlined in the MTO's Request for Proposal (RFP) dated January 2005, and in Section 6.8 of MRC's *Technical Proposal* for this project. The request for foundation engineering input to assess the cause of distress along a 65 m length of the Division Street W-N/S Ramp was presented in an e-mail from MTO dated September 27, 2005; the scope of work related to this assessment was subsequently presented in Golder's letter dated November 14, 2005.

2.0 SITE DESCRIPTION

The Highway 401-Division Street interchange is located in the City of Kingston, in Frontenac County. The Division Street W-N/S Ramp is located in the southwest quadrant of the interchange, as shown on Drawing 1.

The existing Division Street W-N/S Ramp was constructed in 1997 under MTO Contract 97-54, and immediately experienced settlement over an approximately 65 m length between about Stations 23+840 and 23+905. The pavement in this section was reconstructed (via removal of the asphalt, placement of granular padding, and re-paving), then it settled again. In late 2005 and 2006, a “semi-circular” pattern of cracking was observed within these limits along the south (right) shoulder and edge of pavement.

Between Stations 23+840 and 23+905, the ramp grade is at approximately Elevation 82.5 m, and the original grade outside of the ramp is relatively flat at approximately Elevation 79 m to 80 m; the existing ramp embankment is approximately 2.5 m in height relative to the surrounding grade. To the north and south of the ramp in this area, the ground appears swampy. Twin 1.4 m diameter CSP culverts are present to the east of the affected ramp area, around Station 23+935 to 23+940.

A surveyed profile of the existing ramp is provided in Appendix A; this survey was completed by J.D. Barnes Surveying Ltd. in May 2006. This drawing also shows the original design grades projected from the Contract 97-54 design profiles. Based on comparison of these profile data, the right ramp shoulder has settled up to 400 mm relative to the original design grade at approximately Station 23+880; the settlement of the right shoulder decreases away from this point. The left ramp shoulder has settled up to 100 mm relative to the original design grade. It is noted that these settlement magnitudes do not take account of the post-1997 reconstruction, and so the actual total settlement will be greater.

3.0 INVESTIGATION PROCEDURES

The field work for this subsurface investigation was carried out in February 2006. A total of four boreholes (Boreholes E-1, E-2, E-2B and E-3) were advanced using a CME-55 track-mounted drill rig, supplied and operated by Marathon Drilling Company Ltd. of Ottawa, Ontario.

The boreholes were advanced at the locations shown on Drawing 1. Boreholes E-1, E-2 and E-2B were advanced along the western toe of the ramp embankment, and Borehole E-3 was advanced on the west shoulder of the ramp, through the ramp embankment fill. Boreholes E-1, E-2 and E-3 were drilled to total depths of 12.8 m below the ground surface at the borehole locations, to terminate within stiff to very stiff silty clay to clay soil. Borehole E-2B was terminated at 2.3 m depth below ground surface within stiff to very stiff silty clay, after determining that the surficial organic deposit was not present at this location.

Soil samples were obtained at 0.75 m to 1.5 m intervals of depth, using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. In situ vane testing, using an MTO “N”-size vane, was carried out to measure the undrained shear strength of the firm to stiff portions of the silty clay to clay deposit that was encountered at the site, and two relatively undisturbed, 75 mm diameter thin-walled Shelby tube samples of the silty clay to clay were obtained.

A standpipe piezometer was installed in Borehole E-2, within the silty clay to clay deposit. The piezometer consists of 25 mm diameter PVC pipe with a slotted tip installed within a 1.5 m thick filter sand pack. A 0.3 m thick bentonite seal was placed on top of the filter sand followed by a mixture of bentonite and clay soil to the ground surface, where a 0.3 m bentonite seal was placed around the piezometer casing. The remaining boreholes were backfilled with a mixture of bentonite and clay from the augered boreholes, in accordance with Ontario Regulation 128 (amendment to Ontario Regulation 903).

The field work was supervised on a full-time basis by members of Golder’s staff, who located the boreholes in the field, directed the drilling, sampling, and in situ testing operations, and logged the boreholes. The soil samples were identified in the field, placed in labelled containers and transported to Golder’s laboratory in Mississauga for further examination and laboratory testing. Index and classification tests consisting of water content determinations, Atterberg limits testing and grain size distribution analyses were carried out on selected soil samples. In addition, two oedometer (consolidation) tests were carried out on selected samples of the silty clay to clay deposit.

The borehole locations and ground surface elevations following the investigation were provided by J.D. Barnes Surveying Ltd. The borehole locations, including MTM NAD83 northing and easting coordinates and ground surface elevations referenced to geodetic datum, are summarized in the following table and are shown on Drawing 1.

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

The site is located in the southern portion of the physiographic region of Southern Ontario known as the Napanee Plain, as delineated in *The Physiography of Southern Ontario*[1]. The Napanee Plain is flat to undulating, and is characterized by relatively shallow soil deposits overlying bedrock. Geologic mapping[2] indicates that the bedrock within the Napanee Plain consists of grey limestone/dolostone of the Gull River Formation (of the Trenton-Black River Group), which contains some shale partings and seams.

The overburden soils within the Napanee Plain generally consist of glacial till, although alluvium is present in river and stream valleys and, in the southern portion of the Plain, low-lying areas are typically covered with deposits of stratified clay. Well records indicate that the average depth to bedrock within the Napanee Plain is approximately 2 m. However, in many areas, bedrock outcrops exist at ground surface, while deeper soil deposits (on the order of 10 m) are present in the northern and southern portion of the Plain, and within and adjacent to river valleys throughout the Plain.

4.2 Site Stratigraphy

As part of the subsurface investigation at this site, four boreholes were advanced between Stations 23+850 and 23+910 along the Division Street W-N/S Ramp. The borehole locations, ground surface elevations and interpreted stratigraphic conditions in this area are shown on Drawing 1.

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in-situ and laboratory testing are given on the Record of Borehole sheets and Figures 1 to 4 following the text of this report. The stratigraphic boundaries shown on the borehole records are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In summary, the subsoils at the site consist of ramp embankment fill or topsoil (at the toe of the embankment), over a very soft to firm, surficial organic-containing deposit that is typically about 2 m in thickness; this is in turn underlain by a deposit of firm to very stiff silty clay to clay, in which all of the boreholes were terminated. The surficial organic-containing deposit is absent in Borehole E-2B, which was drilled at about Station 23+910; this borehole was advanced just beyond the eastern limit of the affected area of the ramp embankment.

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[1] Chapman, L.J. and D.F. Putnam. *The Physiography of Southern Ontario*. Ontario Geological Survey Special Volume 2, Third Edition, 1984. Accompanied by Map P.2715, Scale 1:600,000.

[2] Map 2544, Ministry of Northern Development and Mines, 1991.

A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Ramp Embankment Fill

Approximately 3.7 m of sand and gravel fill was encountered in Borehole E-3, which was drilled through the south shoulder of the W-N/S Ramp embankment.

The Standard Penetration Test (SPT) “N” values measured within the fill range from 4 to 49 blows per 0.3 m of penetration, indicating that the fill has a variable, loose to dense relative density. The higher SPT “N” values (20 and 49 blows per 0.3 m of penetration) were measured above the water level, while the lower SPT “N” values (4 and 8 blows per 0.3 m of penetration) were measured below the water level in the open borehole. It is considered that some minor disturbance of these lower two samples may have occurred from water inflow to the borehole during sampling. However, the lower portion of the fill would still be considered to have a loose to compact relative density.

4.2.2 Topsoil

About 200 mm of topsoil was encountered immediately below the ground surface in Boreholes E-1, E-2 and E-2B, which were advanced at the south toe of the ramp embankment.

4.2.3 Surficial Organic Deposit

A surficial layer consisting of organic silty clay was encountered immediately below the fill or topsoil in Boreholes E-1, E-2 and E-3. The surface of this layer was encountered at Elevation 79.8 m to 79.9 m in the two boreholes at the ramp embankment toe, and at about Elevation 78.8 m in Borehole E-3 which was extended through the ramp embankment fill. The surficial organic deposit is approximately 1.8 m to 1.9 m in thickness as encountered in these boreholes.

Organic content testing was carried out on two samples of this deposit, and measured organic contents of approximately 19 and 28 per cent. The measured natural water contents on three samples of this material vary from 56 to 166 per cent. Atterberg limit testing was carried out on one sample of the deposit, and measured a plastic limit of 92 per cent, a liquid limit of 112 per cent, and a corresponding plasticity index of 30 per cent; this result (which would plot below the “A-line” on a plasticity chart) together with the results of organic content testing confirm that the tested material is an organic silty clay.

The measured SPT “N” values within this material vary from 1 to 4 blows per 0.3 m of penetration, indicating that this material has a very soft to firm consistency.

4.2.4 Silty Clay to Clay

The surficial organic deposit is underlain by a deposit of grey silty clay to clay. The surface of this deposit was encountered at approximately Elevation 77.9 m to 78.1 m in Boreholes E-1 and E-2, which is about 2.0 m to 2.1 m below the existing ground surface at the embankment toe area; the surface of the deposit was encountered at approximately Elevation 77.0 m in Borehole E-3, about 5.5 m below the ramp pavement grade at this location. In Borehole E-2B, which is located outside of the extent of the surficial organic deposit, the surface of the grey silty clay to clay was encountered immediately below the topsoil, at about Elevation 79.7 m. All of the boreholes were terminated within this deposit, which extends to at least 12.8 m depth (approximately Elevation 67.2 m).

The silty clay to clay generally contains trace sand and trace to no gravel; a greater proportion of sand and gravel was encountered within the deposit in one sample from Borehole E-2. The results of grain size distribution testing carried out on three selected samples of the deposit are provided on Figure 1.

Atterberg limit testing was performed on eight selected samples of the silty clay to clay deposit and measured plastic limits ranging from 21 to 25 per cent, liquid limits ranging from 40 to 56 per cent, and plasticity indices ranging from 24 to 32 per cent. These results, which are summarized on the plasticity chart on Figure 2, indicate that this deposit is a silty clay to clay of intermediate to high plasticity.

The measured SPT “N” values within the silty clay to clay range from 2 to 14 blows per 0.3 m of penetration, but are typically between 2 and 5 blows per 0.3 m of penetration. In situ vane testing carried out in the Boreholes 1, 2 and 3, measured undrained shear strengths ranging from 8 kPa to 70 kPa above about Elevation 73.5 m, while below about Elevation 73.5 m the undrained shear strength was measured to be greater than 100 kPa. These results indicate that the silty clay to clay above about Elevation 73.5 m generally has a firm to stiff consistency, however, the upper portion of the deposit has a very soft consistency as encountered at Borehole E-2. Below about Elevation 73.5 m, the silty clay to clay has a very stiff consistency.

Where undrained shear strengths of less than 100 kPa were measured, remoulded shear strengths were also measured in order to determine the sensitivity of the silty clay to clay deposit. Based on these results, the sensitivity of the clay varies from approximately 2 to 4; these results indicate that the clay has a low to medium sensitivity (according to the *Canadian Foundation Engineering Manual*).

Laboratory oedometer (consolidation) testing was carried out on two specimens from the upper portion of the silty clay to clay deposit obtained from Boreholes E-2 and Borehole E-3,. Details of the test results are shown on Figures 3 and 4, and the results are summarized in the following table.

Borehole/ Sample No.	***Elevation (m)***	σ_{vo}' ***(kPa)***	σ_p' ***(kPa)***	σ_p' - σ_{vo}' ***(kPa)***	***OCR***	e_o	C_r	C_c	c_v ***(cm²/s)***
E-2, Sa 3	76.8	25	25	0	1.0	1.55	0.035	0.35	6.3×10^{-4}
E-3, Sa 7	76.1	30	105	75	3.5	1.2	0.025	0.25	1.8×10^{-3}

where: σ_{vo}' is the effective overburden pressure in kPa
σ_p' is the preconsolidation pressure in kPa
OCR is overconsolidation ratio
e_o is initial void ratio
C_c is the compression index (based on void ratio)
C_r is the recompression index
c_v is the coefficient of consolidation in cm^2/s

Based on the oedometer test results summarized above, the upper portion of the silty clay to clay deposit outside the ramp embankment footprint is normally consolidated, whereas the upper portion of the deposit under the ramp embankment, at essentially the same elevation, is slightly preconsolidated.

4.3 Groundwater Conditions

A standpipe piezometer was installed in Borehole E-2 within the silty clay to clay stratum; details of the piezometer installation are shown on the Record of Borehole E-2 following the text of this report. The water level measured in this piezometer is summarized in the table below:

Ground Water Level Depth	***Ground Water Level Elevation***	***Date***
0.0 m	80.1 m	May 9, 2006
0.4 m above ground surface	80.5 m	January 30, 2007

The water level at the site is expected to fluctuate seasonally in response to changes in precipitation and snow melt; the water level is expected to be higher during the spring season.

5.0 CLOSURE

This Foundation Investigation Report was prepared by Mr. Brian Lapos, EIT, and reviewed by Ms. Lisa Coyne, P.Eng., an Associate and geotechnical engineer with Golder, with technical input from Mr. Murty Devata, P.Eng., a specialist foundations consultant to Golder. Mr. Fin Heffernan, P.Eng., a Designated MTO Contact with Golder, conducted an independent review of the report.

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PART B

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6.0 ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides remediation alternatives and geotechnical recommendations to address the stability and settlement at the Division Street W-N/S Ramp. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at this site. The interpretation and recommendations provided are intended to provide the highway designers with sufficient information to assess the feasibility of the remediation alternatives and to prepare the design drawings. Where comments are made on construction they are provided in order to highlight those aspects which could affect the design of the project, and for which special provisions or operational constraints may be required in the Contract Documents. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

6.2 Assessment of Existing Ramp Embankment

An approximately 65 m length of the Division Street W-N/S Ramp, between about Stations 23+840 and 23+905, experienced settlement after original construction in 1997, was reconstructed, and has since displayed ongoing evidence of slope instability (a “circular” pattern of cracking along the south shoulder and edge of pavement) and settlement. The stability and settlement of this section of the ramp embankment are assessed in the following Sections 6.2.1 and 6.2.2. Based on these findings, discussion and recommendations regarding mitigation options for the W-N/S Ramp embankment are presented in Section 6.3 and its sub-sections.

6.2.1 Embankment Stability

Static slope stability analyses of the Division Street W-N/S Ramp were carried out with the commercially available program SLOPE-W produced by Geo-Slope International Ltd., to determine whether the observed patterns of cracking (“circular” cracking along the south embankment shoulder and edge of pavement) have been caused by failure through the surficial organic silty clay deposit, or through the deeper, grey silty clay to clay deposit.

The table below summarizes the soil parameters that have been used in the stability analyses. The undrained shear strengths used in the analyses, as summarized in the table above, are based on the design shear strength profile provided on Figure 5. This figure plots the corrected undrained shear strength (based on Bjerrum’s correction method) from in situ vane testing as well as shear strengths calculated from the oedometer test results based on the formula $s_u = 0.22 \times \sigma_p'$ (in kPa).

Soil Type	*Bulk Unit Weight*	*Effective Angle of Friction*	*Undrained Shear Strength*
Embankment fill (sand and gravel fill)	21 kN/m^3	32º to 35º	-
Surficial organic deposit	17.5 kN/m^3	-	5 to 10 kPa
Firm to stiff silty clay to clay	17.5 kN/m^3	-	40 kPa
Soft to firm silty clay to clay	17.5 kN/m^3	-	10 kPa
Stiff to very stiff silty clay to clay	17.5 kN/m^3	-	70 kPa

The results of the slope stability analyses using these parameters indicate that the as-built ramp embankment, with side slopes oriented at 2H:1V, has a factor of safety of approximately 1.1 to 1.2 against deep-seated slope instability for a failure surface passing through the surficial organic deposit (see Figure 6). For failure surfaces passing through the deeper, firm to stiff or soft to firm portions of the grey silty clay to clay deposit, the factor of safety against slope instability increases to greater than 1.3.

A factor of safety of 1.1 to 1.2 is lower than what would normally be considered acceptable (i.e., factor of safety of 1.3) for stability of the ramp embankment side slopes; however, this factor of safety is not below 1.0 and suggests that although marginal, this mechanism of slope failure has not occurred. Rather, it is likely that progressive creep of the organic soils has occurred, leading to ongoing deformation of the slope as discussed in Section 6.2.2.

6.2.2 Embankment Settlement

A surveyed profile of the existing Division Street W-N/S Ramp, completed by J.D. Barnes Surveying Ltd. in May 2006, is provided in Appendix A; this drawing also shows the original design grades projected from the MTO Contract 97-54 design profiles. Based on comparison of these profile data, the right ramp shoulder appears to have settled about 400 mm relative to the original design grade at approximately Station 23+880; the settlement of the right shoulder decreases toward the west and east. The left ramp shoulder has settled up to about 100 mm relative to the original design grade. However, these settlement magnitudes do not take account of the post-1997 reconstruction, and so the actual total settlements are greater than those indicated above.

Settlement analyses have been carried out to determine the total magnitude and duration of settlement of the founding soils that would occur / have occurred under the existing Division Street W-N/S Ramp embankment between Stations 23+840 and 23+905. The settlement analyses were carried out using the commercially-available program UNISETTLE (Version 3.0), using the Boussinesq equation for distribution of stresses in the foundation soils. The settlement profile was assessed to a depth of 14 m below the ramp pavement; below this depth, the estimated magnitude of settlement was negligible in the very stiff silty clay to clay deposit. The overconsolidation ratio (OCR) profile used in the analysis is shown on Figure 7; this profile was established using the results of the oedometer testing as well as correlations with the results of the in situ vane tests, based on the following relationship between field vane shear strength and preconsolidation pressure:

$$s_u = 0.22\ \sigma_p'$$

The compression and recompression indices (C_c and C_r, respectively) for the soft to stiff portions of the silty clay to clay deposit have been determined from the oedometer tests and the following correlations with the Atterberg limits:

$$C_c = 0.009\ (w_L - 10)$$

$$C_c = 0.5 \text{ x GS } (PI / 100)$$

$$C_c = 0.75\ (e_o - 0.5)$$

$$C_r = C_c / 10$$

Figures 8 to 10 illustrate the void ratio, compression index and recompression index profiles for the silty clay to clay deposit at this site. The following table summarizes the general settlement parameters used in the UNISETTLE analyses.

Soil Unit	***Bulk Unit Weight***	***Initial Void Ratio, e_o***	***Recompression Index, C_r***	***Compression Index, C_c***	***Elastic Modulus***
Embankment fill	21 kN/m^3	–	–	–	–
Surficial organic deposit	17.5 kN/m^3	2.0	0.08	0.8	–
Firm to stiff silty clay to clay	17.5 kN/m^3	1.55	0.04	0.4	–
Soft to firm silty clay to clay	17.5 kN/m^3	1.2	0.025	0.25	–
Stiff to very stiff silty clay to clay	17.5 kN/m^3	1.0	–	–	20 MPa

For the ramp embankment configuration and subsoil conditions between Stations 23+840 and 23+905, the preconsolidation pressure of the surficial organic deposit and the firm to stiff silty clay to clay layers has been exceeded. The settlement analyses for this ramp section predict that a maximum of about 700 mm of primary consolidation settlement would occur along the south (right) shoulder of the ramp; of this, about 250 mm was predicted to occur in the soft to firm silty clay to clay deposit, and about 450 mm was predicted to occur in the surficial organic deposit. This predicted magnitude of settlement is corroborated by the profiles of the right ramp shoulder as surveyed by J.D. Barnes Surveying Ltd. in May 2006, as shown in Appendix A and discussed above.

It has been estimated that the time to complete ninety per cent of the primary consolidation settlement is five to six years from the end of the original embankment construction. Based on an original construction date of 1997, it is estimated that ninety per cent of the primary consolidation settlement would have been completed by 2003. After 2003, about 70 mm of primary consolidation settlement would still have to take place, and is ongoing at the present time; of this, approximately 25 mm would occur within the soft to stiff silty clay to clay deposit, and the remainder would occur within the surficial organic deposit. In addition, there is secondary creep and/or decay of organic matter present within the surficial organic deposit. The magnitude and duration of this continued settlement is difficult to predict due to the organic nature of the deposit; however, this ongoing “creep” settlement should be expected to be in the range of 30 mm to 50 mm per log cycle of time.

6.3 Embankment Stability/Settlement Mitigation Alternatives

Based on the mechanism for the embankment distress and the resulting instability and settlement, the following options can be considered for mitigation of the Division Street W-N/S Ramp between approximately Station 23+840 and 23+905.

- **Option 1:** Construct stability berms along the ramp embankment side slopes, without subexcavation of the organic layer. This mitigation option would improve the factor of safety against instability of this portion of the ramp (resulting from failure through the weak surficial organic layer), but would not eliminate ongoing creep/settlement of the ramp embankment.

- **Option 2:** Lower the existing ramp grade, without subexcavation of the surficial organic layer. This mitigation option would improve the stability of the embankment and reduce the embankment loading, thereby reducing future settlement. However, it would not completely mitigate against future settlement; some ongoing settlement due to creep and decomposition of the organic layer would continue.

- **Option 3:** Reconstruct the affected section of the embankment using either ultra-lightweight slag fill or EPS fill, without subexcavation of the surficial organic layer. This mitigation option would be similar to Option 2 in that it would improve the stability of the embankment and reduce the embankment loading, thereby reducing future settlement. However, since it will not be possible to achieve “zero net loading” with either ultra-lighweight slag fill or EPS fill, this option would not eliminate future settlement; some ongoing settlement will occur due to continued creep/decomposition of the organic layer.

- **Option 4:** Subexcavate the surficial organic layer from below the affected section of the embankment, and reinstate the ramp embankment fill.

- **Option 5:** Carry out in situ soil improvement below the ramp embankment, by using deep soil mixing or rammed aggregate piers.

- **Option 6:** Realign the W-N/S Ramp and remove poor-performing subgrade soils prior to construction. However, realignment would likely trigger a fisheries impact, and further environmental assessment, investigation and design would be required.

Some additional information regarding each of these mitigation options is provided in the sub-sections that follow, and the advantages, disadvantages, relative costs, and risks/consequences are summarized in tabular format in Table 1 following the text of this report.

Of the above, Options 1 through 3 are not recommended. Option 1 addresses only stability and so would require ongoing maintenance to address settlement resulting from continued creep/decomposition of the surficial organic deposit. Options 2 and 3 would reduce creep effects on stability and reduce the total settlement, but not eliminate it as the surficial organic layer would remain in place under these schemes, and will continue to contribute to settlement (although reduced) of the ramp embankment.

Of the above mitigation options, Option 4 is considered to be the most technically feasible, practicable and cost-effective option that will mitigate both the instability and settlement issues at the affected portion of the Division Street W-N/S Ramp; this option is anticipated to deliver the best post-reconstruction performance, and is the recommended option from a foundations perspective. Option 5, involving in situ soil improvement, is considered a feasible alternative to Option 4, though such techniques are not commonly used on MTO projects and may be uneconomical for this relatively small project.

Option 6, though feasible from a geotechnical perspective and also likely to deliver superior post-construction performance, would likely require additional environmental assessment due to fisheries impact, and would require additional investigation and design.

While consideration could be given to the use of geotextile/geogrid reinforcement in combination with some of the settlement mitigation alternatives discussed above, additional design assessment would be required. A geotextile/geogrid reinforcement layer would enhance stability; however, the reinforcement would not substantially reduce settlement unless installed over a sufficiently large area to provide for lateral anchorage and bridging of the affected area. In addition, reinforcement would not affect the component of settlement that would result from decomposition of the organic material. If the organic silty clay to clay layer were removed and a geosynthetic reinforcement layer used over the silty clay to clay deposit in the affected area, the ramp surface could exhibit a smoother settlement profile rather than manifest distress in specific areas.

6.3.1 Option 1 – Construct Stability Berms

The stability of the existing embankment could be improved by constructing a stability berm along the embankment side slopes between Stations 23+840 and 23+905. The use of a 2 m wide berm extended to the mid-height of the embankment would increase the factor of safety against a slope failure passing through the surficial organic deposit to greater than 1.3; however, creep movement of the organic soils would still occur and would manifest itself (in the form of cracking and/or subsidence) at the top of the berm or ramp shoulder.

In addition, construction of a berm with no other mitigation measures would not address the ongoing settlement of the ramp. Continued settlement would occur due to creep/decomposition within the surficial organic deposit, and this would be exacerbated by the additional loading from the berms. The magnitude and duration of this continued settlement is difficult to predict due to the organic nature of the surficial deposit; however, the settlement should be expected to be much greater than 25 mm (i.e., ongoing maintenance would be necessary). The continued settlement would result in cracking and/or subsidence of the pavement and shoulder, and would require periodic road maintenance.

6.3.2 Option 2 – Lower Existing Ramp Grade

Lowering the existing grade of the Division Street W-N/S Ramp would improve the stability and reduce the total final settlement of the embankment. The feasibility of this option would need to be confirmed based on highway geometric design considerations; it is understood that the existing ramp geometry would likely require major reconstruction to achieve a grade lowering.

If feasible from a highway design perspective, the height of the embankment could be reduced by 0.5 m to improve the factor of safety against failure through the surficial organic deposit to greater than 1.3; however, some creep movement of the organic soils could still occur and could manifest itself (in the form of cracking and/or subsidence) at the ramp shoulder.

The decreased embankment height would also reduce the total stress applied to the ground, contributing to smaller future settlements. However, this option would only partially mitigate the ongoing settlement since the surficial organic deposit would not be removed; the organic materials would continue to decay/creep over time, resulting in continued settlement. The magnitude and duration of this continued settlement is difficult to predict due to the organic nature of the surficial deposit; however, even with a grade reduction of 0.5 m to 1 m, the settlement should be expected to be in the range of 15 mm to 30 mm per log cycle of time (i.e., ongoing maintenance would be necessary). As for Option 1, the continued settlement would result in cracking and/or subsidence of the pavement and shoulder, and would require periodic road maintenance.

6.3.3 Option 3 – Reconstruct Embankment Using Lightweight Fill

Under this option, the existing embankment fill (sand and gravel) would be removed and replaced with lightweight fill, to reduce the total loading on the foundation subsoils. The use of either ultra-lightweight slag fill (having a bulk unit weight of approximately 11.5 kN/ m^3) or extruded polystyrene (EPS) fill (having a bulk unit weight of less than 1 kN/ m^3) could be considered.

Ultra-lightweight slag fill could be used for ramp embankment construction provided that it meets Ontario Regulation 347 leachate criteria for “inert fill”. However, even if the slag meets these criteria, leachate produced from the slag that then interacts with the groundwater could result in precipitation of calcium carbonate from the groundwater, potentially leading to plugging of adjacent subdrains if present. Further, if slag fill is used below the water table, it will absorb water contributing to a significant increase in the unit weight of the material.

EPS fill would have to be maintained above the water level at the site to avoid flotation. EPS fill, if adopted, would require a minimum of 1.5 m of conventional fill/pavement structure cover on top of the embankment and side slopes in order to minimize icing potential on the road surface; in addition, the EPS would have to be encapsulated within polyethylene sheeting or other material as recommended by the manufacturer/supplier to protect against degradation from exposure to hydrocarbons, and it should be covered by a concrete slab to improve the long-term performance under traffic loading.

Based on the above considerations, it is not possible to achieve a “zero net loading” configuration for the lightweight fill materials, though the total embankment loading could be reduced from the existing. The reduced embankment loading would produce similar results to Option 2, though likely with greater expense due to the cost of the supply, shipping and placement of the lightweight fill materials. The factor of safety against slope failure through the surficial organic deposit would be improved to greater than 1.3; however, some creep movement of the organic soils could still occur and could manifest itself in the form of cracking and/or subsidence at the ramp shoulder.

As with Option 2, the decreased embankment loading would also reduce the total stress applied to the ground, contributing to smaller future settlements. However, this option would only partially mitigate the ongoing settlement since the surficial organic deposit would not be removed; the organic materials would continue to decay/creep over time, resulting in continued settlement. The magnitude and duration of this continued settlement is difficult to predict due to the organic nature of the surficial deposit; however, even with reduced loading due to the use of lightweight fill, the estimated settlement should be expected to be approximately 15 mm to 30 mm per log cycle of time (i.e. ongoing maintenance would be required). As for Option 1, and similar to Option 2, the continued settlement would result in cracking and/or subsidence of the pavement and shoulder, and would require periodic road maintenance.

6.3.4 Option 4 – Subexcavate Surficial Organic Deposit

This option would involve removal of the affected portion of the ramp and subexcavation of the approximately 2 m thick surficial deposit below the full width of the affected portion of the Division Street W-N/S Ramp, between approximately Stations 23+840 and 23+905. The subexcavation width should be defined by lines drawn downward at 1H:1V from the toe of the existing and/or new ramp embankment (whichever is wider at any given point along the subexcavation area) to the base of the excavation. Since the existing ramp is only a single lane in width, removal of the affected portion of the ramp and sub-excavation of the underlying surficial deposit will require that the ramp be closed and traffic rerouted to an alternate exit.

Following the removal of the surficial organic deposit, the subexcavated area should be backfilled and the ramp reconstructed using suitable fill material. As discussed in Section

6.4.2, the subexcavated area should be replaced with backfill consisting of OPSS 1010 Granular A Type II or well-graded sand and gravel; it is recommended that an Operational Constraint or Non-Standard Special Provision be included in the Contract Documents to require that the subexcavation backfill be placed quickly following completion of subexcavation, to minimize degradation/disturbance of the silty clay to clay subgrade. The ramp embankment can then be reconstructed using suitable earth fill, granular fill or rock fill.

Subexcavation of the surficial organic deposit would improve the stability and reduce settlement of the affected embankment area, and would minimize cracking of the pavement, since it will eliminate the ongoing creep/decomposition associated with the surficial organic deposit. The factor of safety against deep-seated slope instability of the reconstructed embankment would be greater than 2.0, as shown on the typical analysis result on Figure 11 (based on geotechnical engineering parameters as provided in Section 6.2 of this report).

The total final settlement of the soils below the reconstructed embankment is expected to be minimal since effective “preloading” of the silty clay to clay subsoils has occurred since the original construction of the ramp embankment in 1997; further, with the removal of the surficial organic deposit, no creep/decomposition movements will occur. Based on the settlement parameters and analyses as discussed in Section 6.2, it is estimated that as of 2003 the silty clay to clay subsoils had achieved 90 per cent of the primary consolidation settlement under the loading from the existing embankment; as of 2007, it is estimated that less than 25 mm of primary consolidation settlement remained under the existing ramp embankment loading. Some additional settlement will result from the increase in net loading due to replacement of the surficial organic soil with heavier fill; this increase is expected to result in approximately 15 mm of additional primary consolidation settlement of the firm to stiff silty clay to clay soils.

6.3.5 Option 5 – Soil Improvement Below Existing Ramp Embankment

As an alternative to conventional subexcavation of the organic-containing surficial deposit below the ramp embankment, the use of deep soil mixing or rammed aggregate piers could be considered to improve the performance of the surficial organic deposit.

The mobilization costs associated with the deep soil mixing equipment would be high; discussions with a specialized contractor have indicated that it would not be practical to mobilize equipment to Kingston for the relatively limited improvement works required for this approximately 65 m length of ramp, and so a deep soil mixing option for subsoil improvement is not considered feasible.

Rammed aggregate piers, which can be installed to a maximum of about 7.5 m depth, could be feasible for improvement below the Division Street W-N/S Ramp based on the height of the embankment and thickness of the surficial organic deposit. The rammed aggregate piers would likely be installed in an array under the full width and length of the affected embankment area. The ramp would have to be closed during active work periods; however, it may be possible to re-open the ramp periodically during construction since this option would not require complete removal of the existing embankment fill.

Discussions with a specialist contractor have indicated that rammed aggregate piers could be competitive with conventional subexcavation, since it would not be necessary to remove the existing ramp embankment fill to improve the underlying organic deposit. In addition, rammed aggregate piers could be extended to about 7.5 m depth (Elevation 75 m), which would extend below the soft to firm zone of the grey silty clay to clay deposit. Provided that the surficial organic deposit is effectively treated using this method, the post-construction consolidation settlement of the underlying grey silty clay to clay should be less than 25 mm.

However, cost advantages would likely be lost due to the relatively small size of this project. In addition, although their use is not uncommon in the United States, rammed aggregate piers have not been used to date on an MTO project and there is potential for impacts to the schedule. Further design would be required if this option is pursued.

6.3.6 Option 6 – Realign Ramp

With this option, settlement and stability of a new ramp would be addressed at the investigation and design stage; in addition, this option could allow the current ramp to remain open as a detour during the investigation, design and construction of the ramp realignment.

However, additional environmental assessments/studies would be required for a ramp realignment. Taking into account the investigation, environmental studies, design and construction, this option is expected to be the most costly.

6.4 Construction Considerations

6.4.1 Excavation and Groundwater / Surface Water Control

The natural ground surface in the area is relatively flat, at approximately Elevation 80 m. In order to remove the surficial organic deposit, sub-excavation to between Elevation 77 m and 78 m (i.e. 2 m to 3 m below the natural ground surface) will be required.

The sub-excavation will extend through the existing ramp embankment fill (sand and gravel) and the surficial organic deposit into the underlying silty clay to clay deposit. Open-cut excavations into these materials should be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act (OHSA) for Construction Activities. The surficial organic soils would be classified as Type 4 soil; it is recommended that temporary excavation side slopes through the surficial organic deposit be maintained at 3 horizontal to 1 vertical (3H:1V).

The piezometric level associated with the silty clay to clay deposit at this site is relatively high, near the ground surface. Minor seepage from the silty clay to clay deposit should be expected. In addition, any granular materials existing or placed at the site should be expected to be water-bearing, with water “perched” on top of the relatively impermeable clay deposit, particularly during wet periods of the year. It is anticipated that the groundwater seepage into the foundation excavations can be adequately controlled by pumping from properly filtered sumps.

6.4.2 Subgrade Preparation and Embankment Reconstruction

Assuming that Option 4 is adopted, and as discussed in Section 6.3.4, the existing ramp embankment should be removed and the underlying organic silty clay deposit should be subexcavated between approximately Stations 23+840 and 23+905. The zone of subexcavation should be defined by lines drawn outward and downward at 1H:1V from the toe of the existing and/or new ramp embankment (whichever is wider at any given point along the subexcavation area) to the base of the organic silty clay layer. The subexcavation subgrade should be inspected following subexcavation to ensure that all organic soils have been removed.

Following inspection and approval of the subgrade, the subexcavated area should be replaced with approved backfill. It is recommended that the subexcavation backfill consist of cohesionless fill such as OPSS 1010 Granular “B” Type II, or well-graded sand and gravel. It is noted that the silty clay to clay subgrade in the subexcavation area will be sensitive to disturbance from ponded water and construction traffic. It is recommended that an Operational Constraint be included in the Contract Documents to restrict travelling over the exposed silty clay to clay subgrade to minimize such disturbance and to require timely placement of the first lift(s) of subexcavation backfill. A sample Operational Constraint is included in Appendix B.

The backfill and embankment fill should be placed and compacted in accordance with MTO’s Special Provision 105S10. To minimize differential settlement between the repaired portion of the embankment and the unaffected portion of the embankment due to settlement of the fill itself, the use of granular fill is recommended over the use of cohesive fill, since the majority of settlement of granular fills will occur during construction whereas some settlement of cohesive fills, if used, would occur post-construction. Consideration could also be given to the use of surplus rock fill for the ramp embankment reconstruction; rock fill material would be acceptable provided that the clay subgrade is covered with a minimum 300 mm thick sand and gravel blanket (OPSS 1010 Granular B Type II or similar) before placement of the first lift of rock fill.

The new embankment section should be keyed into the existing embankment sections at either end, in accordance with OPSD 208.010.

7.0 CLOSURE

This Foundation Design Report was prepared by Mr. Brian Lapos, EIT and Ms. Lisa Coyne, P.Eng., an Associate and geotechnical engineer with Golder, with technical input from Mr. Murty Devata, P.Eng., a specialist foundations consultant to Golder. Mr. Fin Heffernan, P.Eng., a Designated MTO Contact with Golder, conducted an independent review of the report.

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TABLE 1
DIVISION STREET W-N/S RAMP – COMPARISON OF EMBANKMENT MITIGATION ALTERNATIVES
HIGHWAY 401 WIDENING, W.P. 77-99-01

Embankment Remediation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Option 1. Construct a mid-height stability berm on the south side of the embankment along the affected area	• Feasible provided that sufficient property is available or can be obtained to construct the stability berms	• Embankment stability will be improved • Ramp closure not required; no interruption to traffic	• The surficial organic deposit below the embankment will not be removed, and so creep/ decomposition of these organic soils will continue; continued maintenance will be necessary	• Least expensive option for construction	• Periodic road maintenance will continue to be required, since surficial organic deposit below embankment will continue to decompose/settle, contributing to cracking and/or subsidence of the pavement and shoulder
Option 2. Lower the existing ramp grade in the vicinity of Station 23+840 to 23+905	• Feasible from a geotechnical viewpoint; however, this option may not be feasible from a road design viewpoint	• Lowering embankment height improves the embankment stability • Decreases the embankment loading, in turn decreasing the total final settlement	• Requires removal of pavement structure, removal of a portion of the embankment fill and re-instating the pavement structure • The surficial organic deposit below the embankment will not be removed, and so creep/decomposition of these organic soils will continue • Ramp closure will be required during construction work	• Likely quite expensive since reconstruction of a fairly long section of ramp might be required for proper geometric design.	• Periodic road maintenance will continue to be required, since organic soil layer below embankment will continue to decompose/settle, contributing to cracking and/or subsidence of the pavement and shoulder • Lowering the grade may not maintain proper driving sight lines/ramp geometrics • Ramp closure will need to be coordinated with other construction works related to this project
Option 3. Remove affected portion of embankment and reconstruct with ultra-lightweight slag fill or EPS fill	• Feasible from a geotechnical and constructability viewpoint	• Reduced embankment loading improves the embankment stability • Reduced embankment loading also reduces total final settlement of the organic soil layer	• Requires removal of pavement structure and existing embankment fill and replacement with ultra-lightweight slag fill or EPS fill • Not possible to achieve zero net loading using lightweight fill at this site, and since the surficial organic deposit below the embankment is not removed under this option, creep/decomposition of these organic soils will continue • Ramp closure will be required during construction work	• More expensive than Option 1 • Due to relatively high cost of ultra-lightweight slag or EPS fill, expected to be similar in cost to Options 2 and 4, and less expensive than Options 5 and 6	• Periodic road maintenance will continue to be required since surficial organic deposit below embankment will continue to creep/ decompose, contributing to cracking and/or subsidence of the pavement and shoulder • Ramp closure will need to be coordinated with other construction works related to this project

TABLE 1
DIVISION STREET W-N/S RAMP – COMPARISON OF EMBANKMENT MITIGATION ALTERNATIVES
HIGHWAY 401 WIDENING, W.P. 77-99-01

Embankment Remediation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Option 4. Subexcavate the approximately 2 m thick organic soil layer below the full width of the ramp embankment within the 65 m long affected section, then reconstruct the ramp embankment	• Feasible with ramp closure • Preferred option from a foundations perspective • Depth of subexcavation is relatively limited (maximum of about 2 m to 3 m below original ground surface)	• Removes surficial organic deposit to found embankment on the silty clay to clay stratum • Embankment stability will be improved • Ongoing settlement of the embankment will be minimized and creep/decomposition of organic soils will be eliminated • Future maintenance (due to settlement/instability) minimized	• Ramp closure required since ramp is a single lane • Extensive construction efforts (i.e. removal of existing embankment, reconstruction of embankment and road structure, and possible replacement of nearby existing culvert) • Environmental mitigation measures required for working adjacent to creek	• More expensive than Option 1; probably similar cost to Options 2 and 3; expected to be less expensive than Options 5 and 6	• Ramp closure will need to be coordinated with other construction works related to this project
Option 5. Improve the surficial organic layer in situ by using deep soil mixing or rammed aggregate piers	• Feasible with ramp closure • Equipment for rammed aggregate piers can penetrate to approximately 7.5 m depth	• Improvements completed in situ without subexcavation of fill or the surficial organic layer; could be an advantage with respect to watercourse • Embankment stability will be improved • Ongoing settlement of the embankment will be minimized	• Ramp closure likely required since ramp is a single lane • Deep soil mixing not cost-effective due to high mobilization costs for equipment • Rammed aggregate piers may not be cost-effective on this relatively small project; in addition, they have not been used to date on an MTO project and there is a possible impact to schedule if "unfamiliar" technology adopted	• Expected to be higher than Option 4; further design and costing by specialist would be required if rammed aggregate piers are pursued	• Ramp closure will need to be coordinated with other construction works related to this contract
Option 6. Realign the ramp	• Feasible if property can be acquired and if proper ramp geometry can be achieved	• New ramp alignment can be investigated, designed and constructed to prevent or minimize any settlement and instability problems	• Existing ramp already on geometrically suitable alignment; new alignment may not satisfy appropriate road design guidelines • Additional geotechnical investigation will be required • Additional environmental assessment may be required • HADD likely	• Expected to be the most expensive option due to major construction operations, subsurface investigation and possible land acquisition	• May require land acquisition, environmental assessment, etc.

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, 60° 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**	**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.

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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
v	poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma$-u)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1+\sigma_2+\sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (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
l_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 (ϕ = 0 analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 + \sigma_3)/2$ or $(\sigma'_1 + \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 + \sigma_3)$
S_t	sensitivity

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

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

PROJECT 05-1111-031

RECORD OF BOREHOLE No E-1

 METRIC

W.P. 77-99-01	LOCATION N 4903147.7 ;E 304677.9	ORIGINATED BY DM	
DIST ___ HWY 401	BOREHOLE TYPE C.M.E. 55, 108mm I.D. Hollow Stem Augers	COMPILED BY BL	
DATUM Geodetic	DATE February 16, 2006	CHECKED BY LCC	

SOIL PROFILE / 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, X REMOULDED; 20 40 60 80 100) / PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L — WATER CONTENT (%) 25 50 75 / UNIT WEIGHT γ kN/m³ / REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH	REMARKS & GRAIN SIZE (GR SA SI CL)
80.0	GROUND SURFACE						
0.0	TOPSOIL						
0.2	Organic SILTY CLAY Very soft to soft Black Wet	1	SS	2	79		O.C = 18.8 (w = 100.6)
		2	SS	1	78		
77.9 2.1	SILTY CLAY to CLAY, trace sand Stiff to very stiff Grey Moist to wet	3	SS	PM	77		
		4	SS	PM	76	+ 3.3; + 2.1	
		5	SS	5	75	+ >100	0 1 20 79
		6	SS	5	74, 73		
		7	SS	4	72, 71		
		8	SS	4	70		
		9	SS	2	69, 68	+ >100	
		10	SS	2			
67.2 12.8	End of Borehole						

Note:

1. Open borehole dry upon completion of drilling operations.

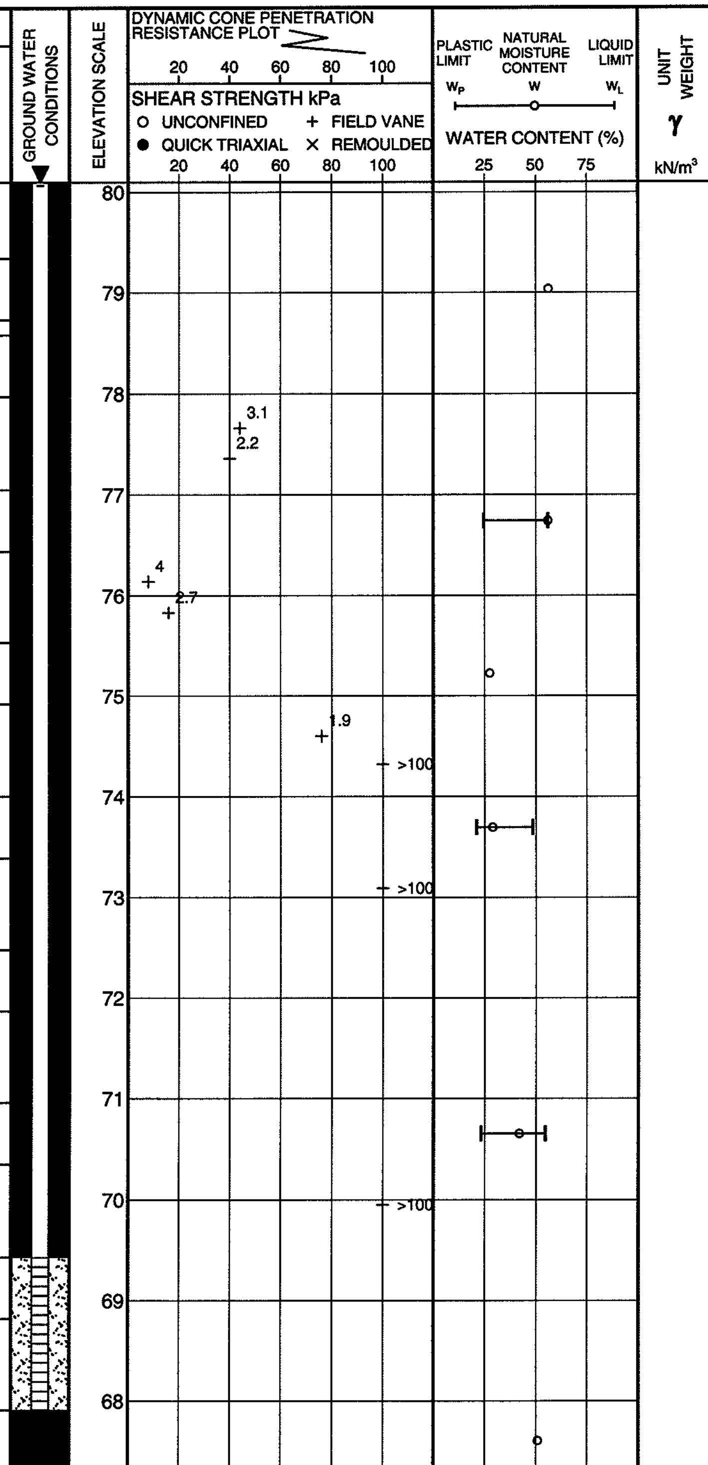
$+^3, \times^3$: Numbers refer to Sensitivity O $^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No E-2

 METRIC

PROJECT 05-1111-031
W.P. 77-99-01 LOCATION N 4903122.1 ;E 304698.3 ORIGINATED BY DM
DIST HWY 401 BOREHOLE TYPE C.M.E. 55, 108mm I.D. Hollow Stem Augers COMPILED BY BL
DATUM Geodetic DATE February 20, 2006 CHECKED BY LCC

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
80.1	GROUND SURFACE				
0.0	TOPSOIL				
0.2	SILTY CLAY containing organics and wood fragments Soft to firm Black to dark grey Moist	1	SS	4	
		2	SS	2	
78.1 / 2.0	SILTY CLAY to CLAY, trace sand Very soft to very stiff Grey Moist to wet				
		3	TO	PH	C D_R = 2.78
	Higher sand and gravel content in Sample 4	4	SS	2	7 21 19 53
		5	SS	10	
		6	SS	4	
		7	SS	4	
		8	SS	2	
		9	SS	2	
67.3 / 12.8					



Continued Next Page

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

MIS-MTO 001 05111031.GPJ GAL-MISS.GDT 2/15/07

Golder Associates

PROJECT 05-1111-031 **RECORD OF BOREHOLE No E-2** **METRIC**

W.P. 77-99-01 LOCATION N 4903122.1 ;E 304698.3 ORIGINATED BY DM

DIST HWY 401 BOREHOLE TYPE C.M.E. 55, 108mm I.D. Hollow Stem Augers COMPILED BY BL

DATUM Geodetic DATE February 20, 2006 CHECKED BY LCC

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION --- CONTINUED FROM PREVIOUS PAGE ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				GR SA SI CL
	End of Borehole Notes: 1. Water level in open borehole at 3.1m depth (Elev. 77 m) below ground surface upon completion of drilling operations. 2. Water level in standpipe measured at ground surface (Elev. 80.1 m) on May 9, 2006, and at 0.4 m above ground surface (at Elev. 80.5 m) on January 30, 2007.										

MIS-MTO 001 05111031.GPJ GAL-MISS.GDT 2/15/07

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

Golder Associates

RECORD OF BOREHOLE No E-2B

 METRIC

PROJECT 05-1111-031
W.P. 77-99-01
DIST ___ HWY 401
DATUM Geodetic

LOCATION N 4903099.3 ;E 304714.1
BOREHOLE TYPE C.M.E. 55, 108mm I.D. Hollow Stem Augers
DATE February 20, 2006

ORIGINATED BY DM
COMPILED BY BL
CHECKED BY LCC

SOIL PROFILE			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, X REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (25 50 75)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
79.9	GROUND SURFACE										
0.0	TOPSOIL										
0.2	CLAYEY SILT to SILTY CLAY, trace sand Firm to stiff Brown Moist										
			1	SS	4		79				
			2	SS	14		78				
77.6 2.3	End of Borehole										

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

Golder Associates

RECORD OF BOREHOLE No E-3

 METRIC

PROJECT 05-1111-031			
W.P. 77-99-01	LOCATION N 4903128.3 ;E 304706.6		ORIGINATED BY DM
DIST	HWY 401	BOREHOLE TYPE C.M.E. 55, 108mm I.D. Hollow Stem Augers	COMPILED BY BL
DATUM Geodetic	DATE February 20, 2006		CHECKED BY LCC

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
82.5	GROUND SURFACE
0.0	ASPHALT
0.1	Sand and gravel (FILL) Dense to loose Dark grey Moist Becoming wet below 2.4 m depth
78.8 3.7	Organic SILTY CLAY Soft to firm Black Wet
77.0 5.5	SILTY CLAY to CLAY, trace sand Firm to very stiff Grey Moist
69.7 12.8	End of Borehole

Note:

1. Water level upon completion of drilling not recorded.

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	49	
2	SS	20	
3	SS	4	
4	SS	8	
5	SS	4	
6	TO	PH	O.C = 27.6
7	TO	PH	0 1 29 70 C D_R = 2.78
8	SS	4	
9	SS	4	
10	SS	2	
11	SS	3	

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

Field vane results: 2.6, 1.7, 2.4, 2.4, >100, >100, >100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 25 50 75 (112)

UNIT WEIGHT γ kN/m³

ELEVATION SCALE: 82 81 80 79 78 77 76 75 74 73 72 71 70

GROUND WATER CONDITIONS

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

MIS-MTO 001 05111031.GPJ GAL-MISS.GDT 2/16/07

MINISTRY OF TRANSPORTATION, ONTARIO

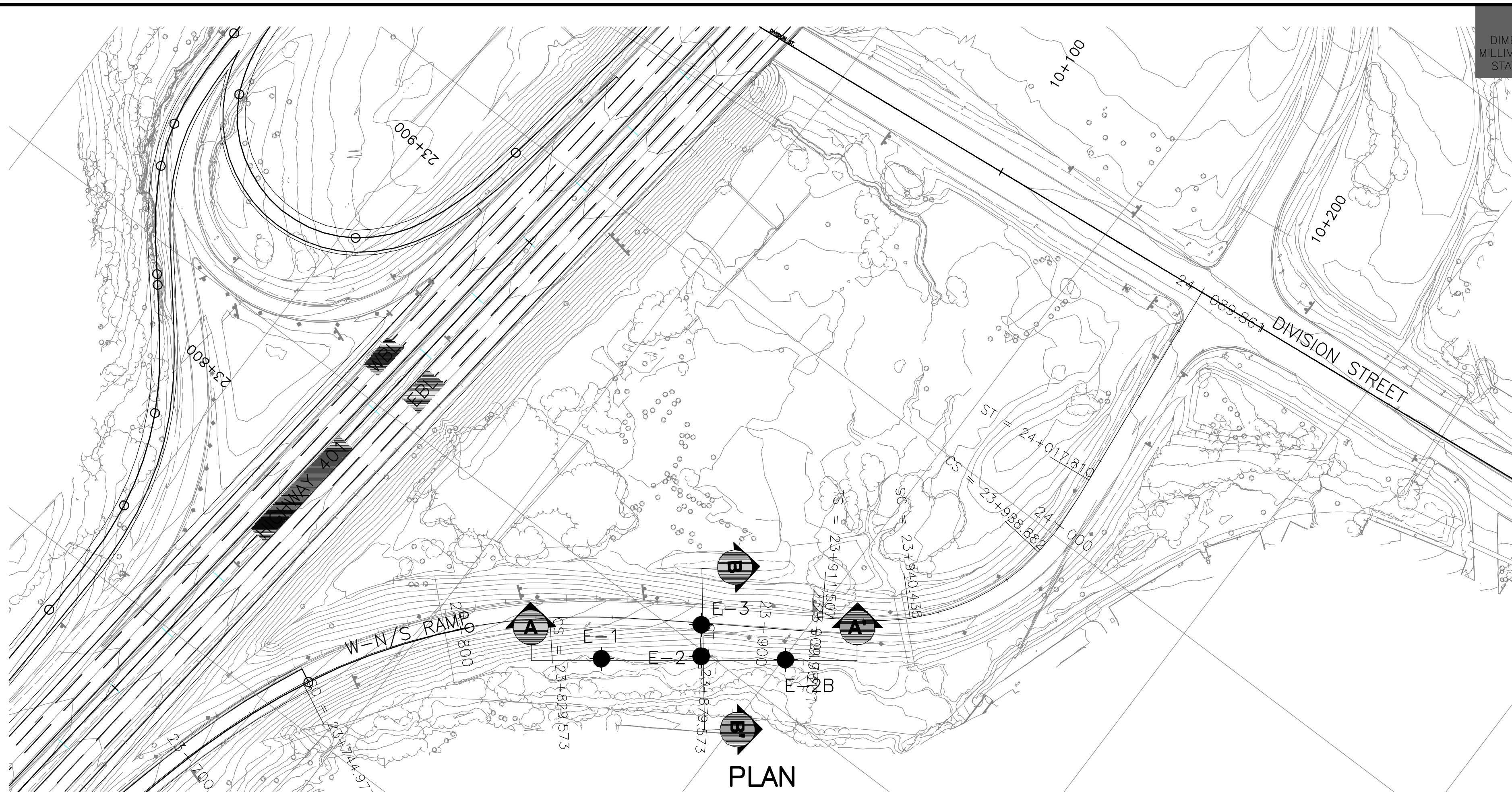
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 77-99-01

DIVISION STREET W-N/S RAMP
HWY 401 WIDENING
BOREHOLE LOCATIONS AND
SOIL STRATA

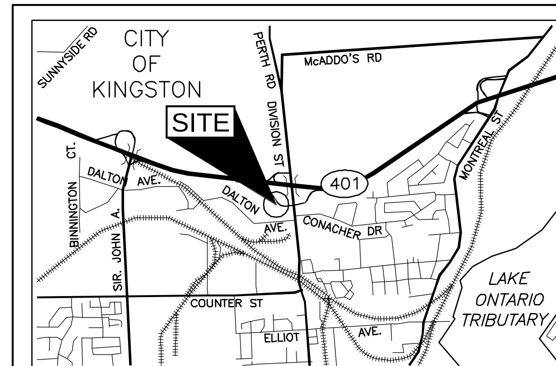
SHEET

Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN

SCALE 20 0 20 40 m



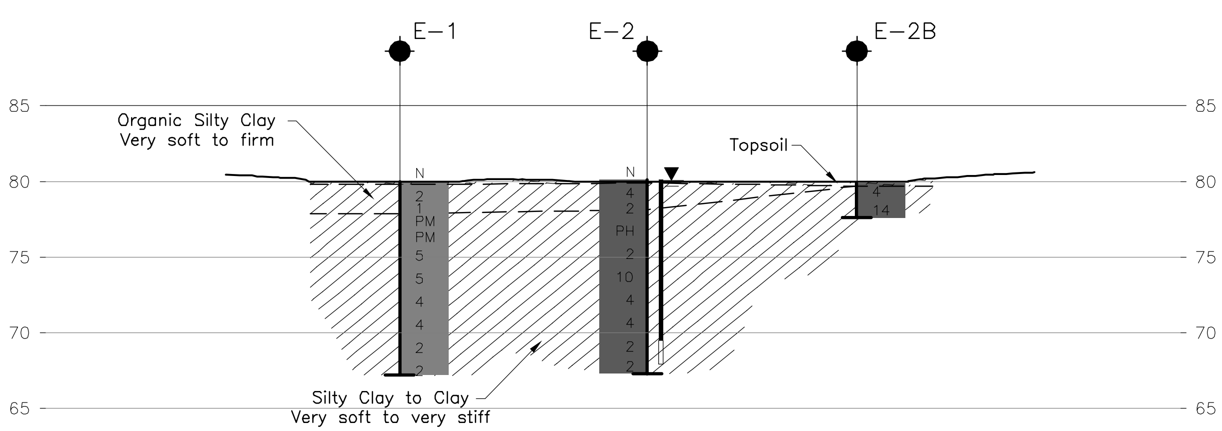
KEY PLAN

SCALE 700 0 700 1400m

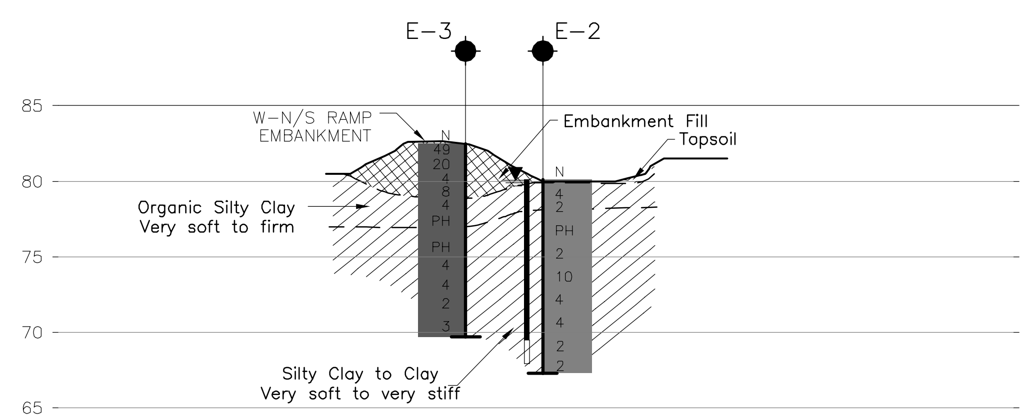
LEGEND

- Borehole – Current Investigation
- Seal
- Piezometer
- N — Standard Penetration Test Value
- 16 — Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL in piezometer

No.	ELEVATION	CO-ORDINATES NORTHING	EASTING
E-1	80.0	4903147.7	304677.9
E-2	80.1	4903122.1	304698.3
E-2B	79.9	4903099.3	304714.1
E-3	81.2	4903128.3	304706.6



A-A' PROFILE AT EMBANKMENT TOE



B-B' SECTION THROUGH RAMP EMBANKMENT

SCALE VERT. 10 0 10 20 m

SCALE HORIZ. 5 0 5 10 m

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 McCormick Rankin Corporation, Drawing Nos. H6230XA01.dwg,H6230XB01.dwg, H6230XB02.dwg, H6230XB03.dwg, H6230XB04.dwg, H6230XM01.dwg, H6230Xn01-2.dwg, received January 10, 2006 and 6230-P1 Division Street.dwg received March 22, 2006.

NO.	DATE	BY	REVISION

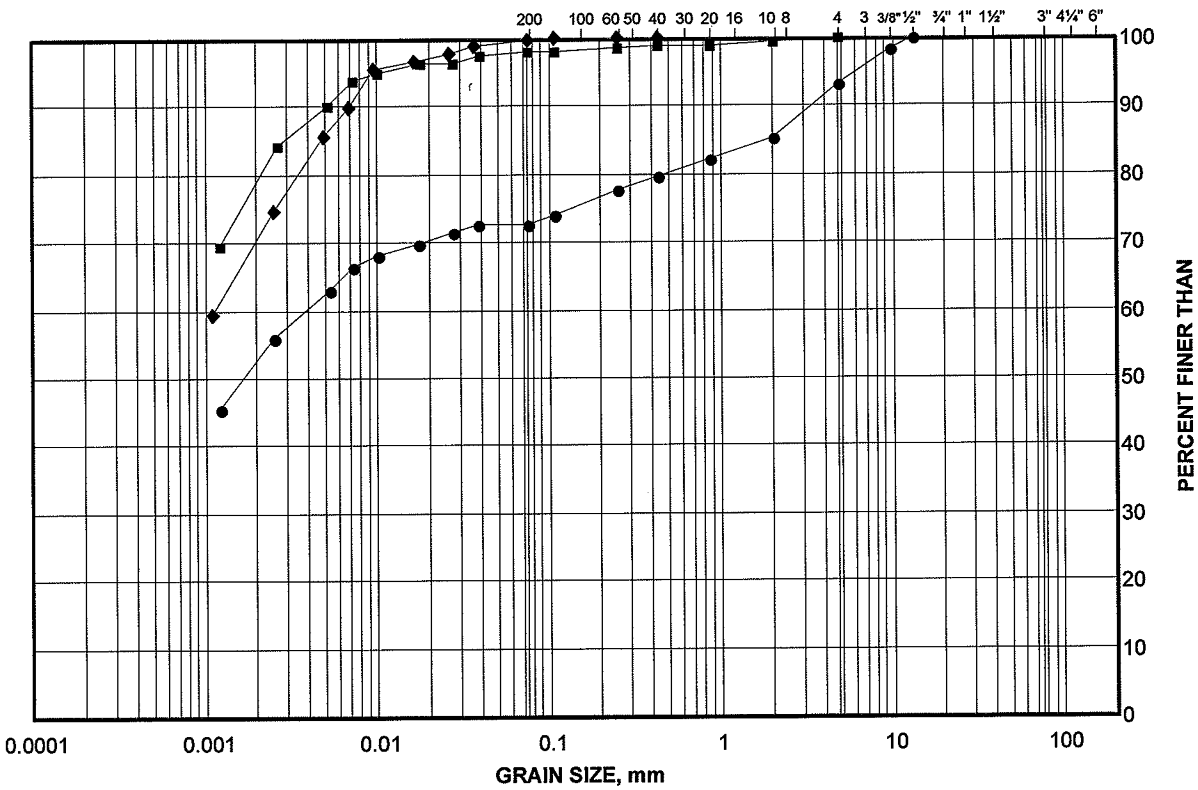
Geocres No. 31C – 177

HWY. 401	PROJECT NO. 05-1111-031		DIST. 41
SUBM'D. BML	CHKD. LCC	DATE: JAN 2007	SITE:
DRAWN: MSM/RJ	CHKD. BML	APPD. LCC/JMAC	DWG. 1

PLOT DATE: November 05, 2007
FILENAME: T:\Projects\2005\05-1111-031 (MRC, Kingston, Ontario)\-CB- (W-S Ramp)\0511103100B001.dwg

GRAIN SIZE DISTRIBUTION TEST RESULTS
Silty Clay to Clay

FIGURE 1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	E-2	4	75.2
■	E-1	5	75.1
◆	E-3	7	76.0

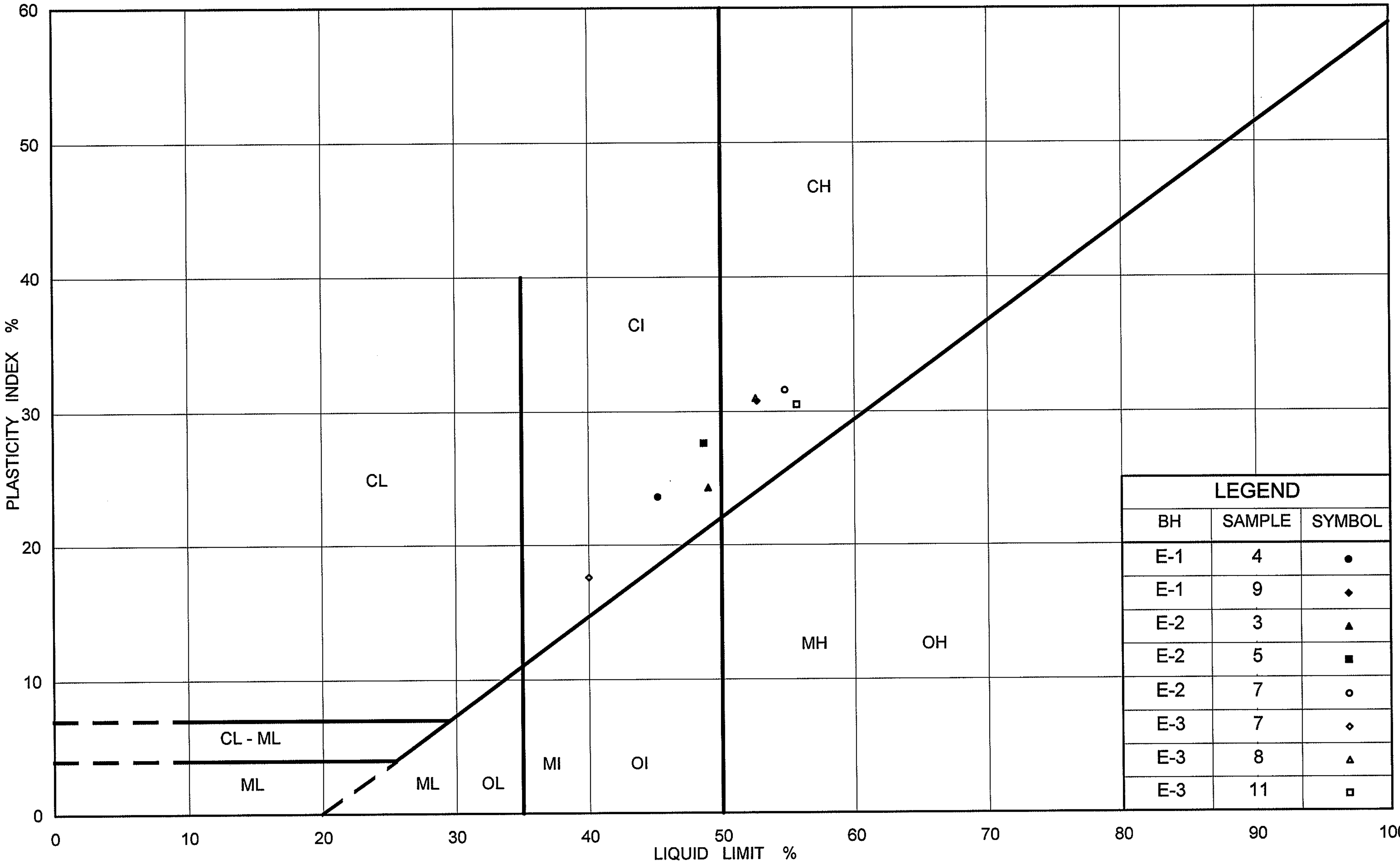
Project Number: 05-1111-031

Checked By: ___

Golder Associates

Date: 15-Feb-07

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
E-1	4	●
E-1	9	◆
E-2	3	▲
E-2	5	■
E-2	7	○
E-3	7	◇
E-3	8	△
E-3	11	□

Ministry of Transportation

Ontario

PLASTICITY CHART
Silty Clay to Clay

FIG No. 2

Project No. 05-1111-031

Date: February, 2007

OEDOMETER CONSOLIDATION SUMMARY

Silty Clay to Clay (Borehole E-3)

SAMPLE IDENTIFICATION

Project Number	05-1111-031	Sample Number	7
Borehole Number	E-3	Sample Depth, m	6.1-6.7

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	7		
Date Started	02/01/2006		
Date Completed	02/17/2006		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m^3	17.54
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	12.17
Area, cm^2	31.65	Specific Gravity, measured	2.78
Volume, cm^3	60.13	Solids Height, cm	0.848
Water Content, %	44.14	Volume of Solids, cm^3	26.85
Wet Mass, g	107.57	Volume of Voids, cm^3	33.29
Dry Mass, g	74.63	Degree of Saturation, %	99.0

TEST COMPUTATIONS

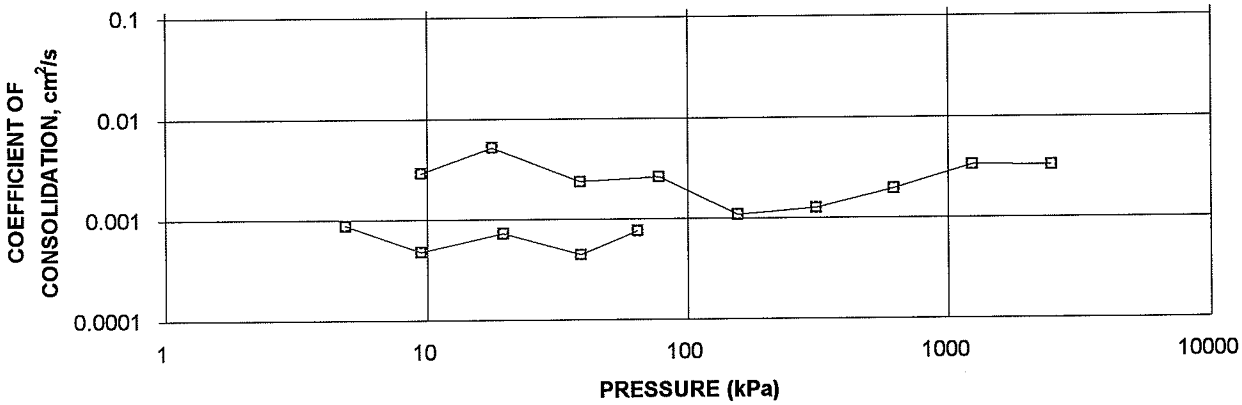
Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	1.900	1.240	1.900				
4.83	1.849	1.180	1.875	844	8.83E-04	5.56E-03	4.81E-07
9.46	1.830	1.157	1.840	1500	4.78E-04	2.16E-03	1.01E-07
19.51	1.799	1.121	1.815	960	7.27E-04	1.62E-03	1.16E-07
38.91	1.759	1.074	1.779	1500	4.47E-04	1.09E-03	4.76E-08
64.22	1.724	1.033	1.742	844	7.62E-04	7.28E-04	5.43E-08
38.91	1.727	1.036	1.726				
19.51	1.733	1.043	1.730				
9.54	1.742	1.054	1.738				
4.83	1.750	1.063	1.746				
9.46	1.747	1.060	1.749	225	2.88E-03	3.41E-04	9.63E-08
17.74	1.742	1.054	1.745	124	5.20E-03	3.18E-04	1.62E-07
38.70	1.730	1.040	1.736	271	2.36E-03	3.01E-04	6.96E-08
77.57	1.703	1.008	1.717	240	2.60E-03	3.66E-04	9.32E-08
155.05	1.644	0.938	1.674	540	1.10E-03	4.01E-04	4.32E-08
309.43	1.578	0.860	1.611	433	1.27E-03	2.25E-04	2.80E-08
619.59	1.500	0.768	1.539	255	1.97E-03	1.32E-04	2.55E-08
1238.50	1.425	0.680	1.463	135	3.36E-03	6.38E-05	2.10E-08
2476.46	1.349	0.590	1.387	124	3.29E-03	3.23E-05	1.04E-08
1238.50	1.362	0.606					
309.43	1.394	0.643					
77.43	1.430	0.686					
19.51	1.473	0.737					
4.83	1.504	0.773					

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

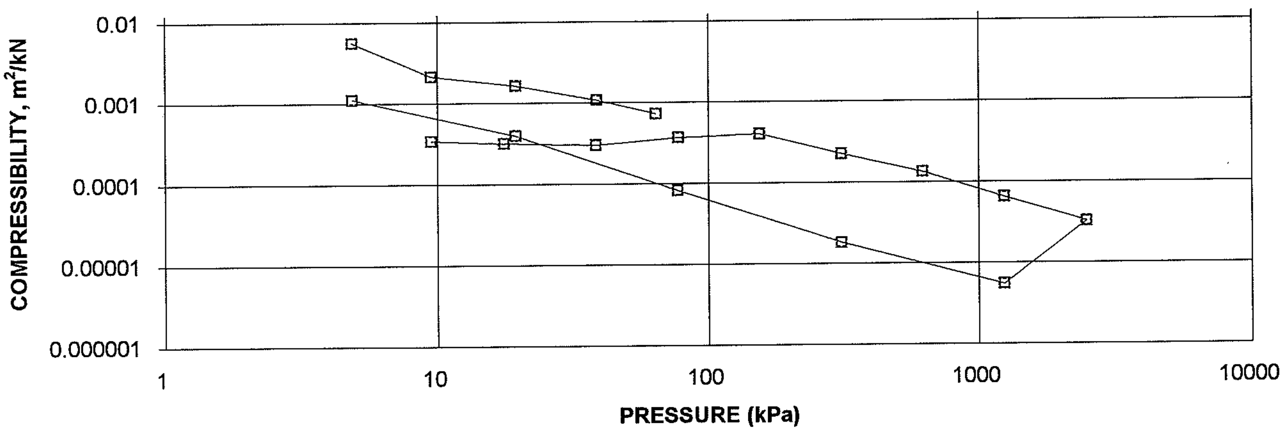
SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.50	Unit Weight, kN/m^3	19.74
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	15.38
Area, cm^2	31.65	Specific Gravity, measured	2.78
Volume, cm^3	47.60	Solids Height, cm	0.848
Water Content, %	28.39	Volume of Solids, cm^3	26.85
Wet Mass, g	95.82	Volume of Voids, cm^3	20.76
Dry Mass, g	74.63		

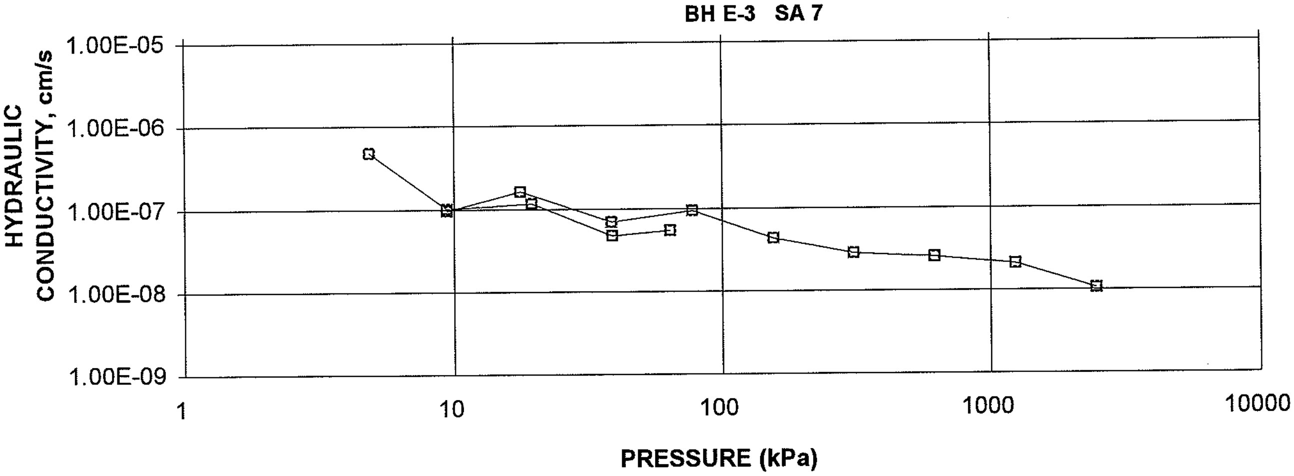
CONSOLIDATION TEST
CV cm²/s VS PRESSURE (kPa)
BH E-3 SA 7



CONSOLIDATION TEST
MV m²/kN vs PRESSURE (kPa)
BH E-3 SA 7

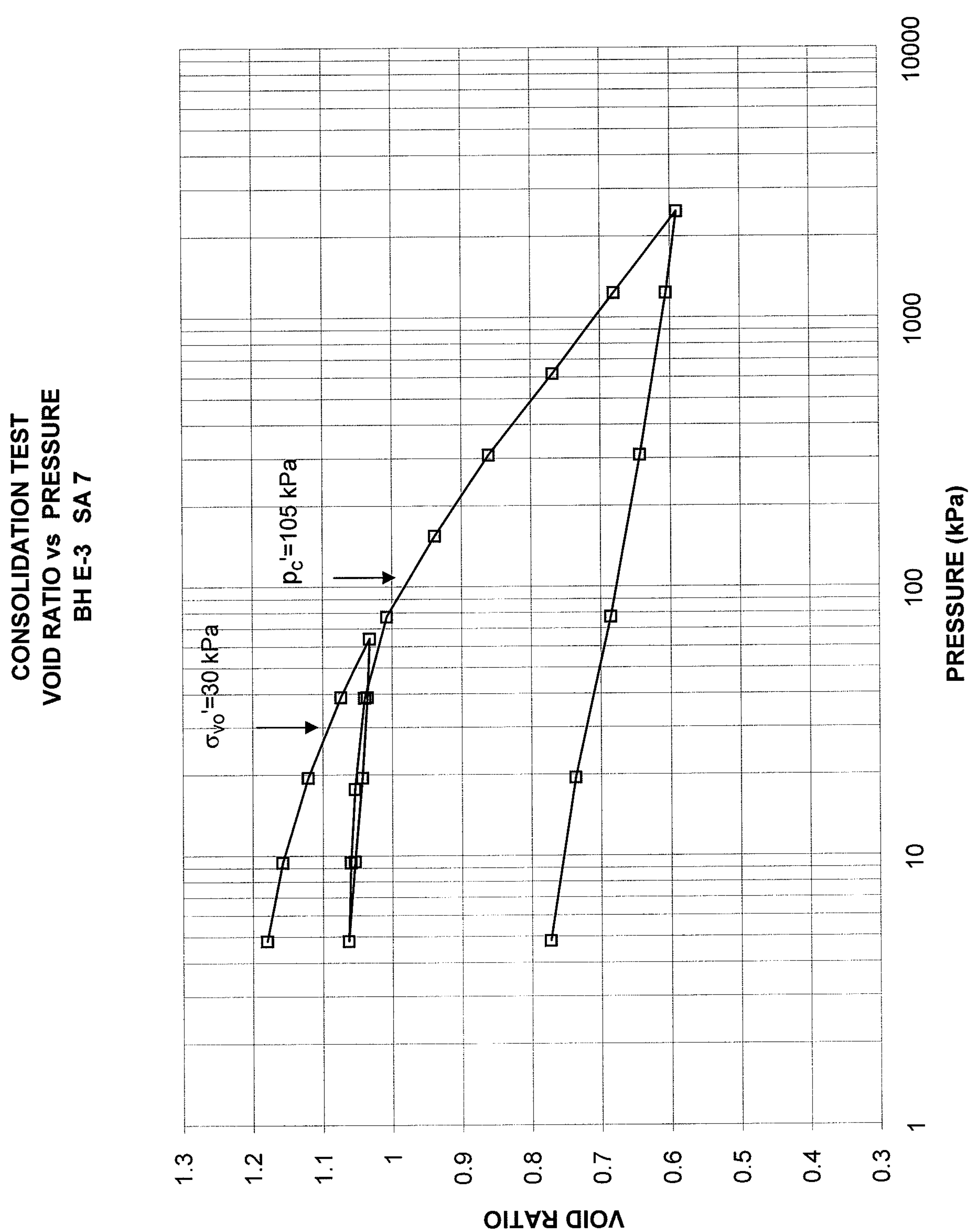


CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH E-3 SA 7



CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE
Silty Clay to Clay (Borehole E-3)

FIGURE 3
Page 3 of 4



Project No.05-1111-031

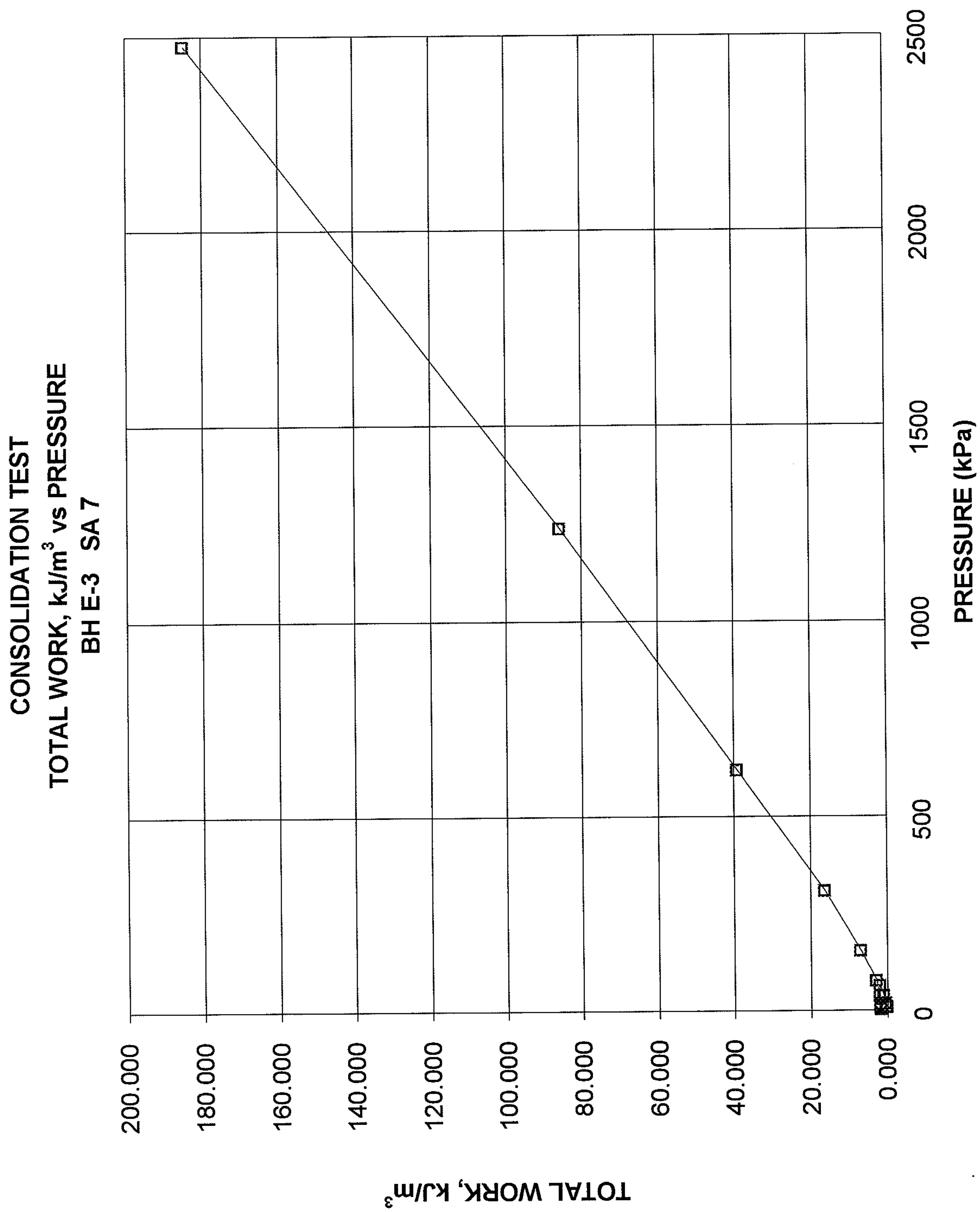
Prepared By: LFG

Golder Associates

Checked By: MM

CONSOLIDATION TEST
TOTAL WORK VS. PRESSURE
Silty Clay to Clay (Borehole E-3)

FIGURE 3
Page 4 of 4



Project No.05-1111-031

Prepared By: LFG

Golder Associates

Checked By: MM

OEDOMETER CONSOLIDATION SUMMARY

Silty Clay to Clay (Borehole E-2)

SAMPLE IDENTIFICATION

Project Number	05-1111-031	Sample Number	3
Borehole Number	E-2	Sample Depth, m	3.0-3.7

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	6		
Date Started	02/01/2006		
Date Completed	02/17/2006		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m^3	16.40
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	10.50
Area, cm^2	31.67	Specific Gravity, measured	2.78
Volume, cm^3	60.17	Solids Height, cm	0.732
Water Content, %	56.19	Volume of Solids, cm^3	23.17
Wet Mass, g	100.60	Volume of Voids, cm^3	37.00
Dry Mass, g	64.41	Degree of Saturation, %	97.8

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	1.900	1.597	1.900				
4.75	1.875	1.563	1.888	540	1.40E-03	2.77E-03	3.80E-07
9.54	1.854	1.534	1.865	634	1.16E-03	2.31E-03	2.63E-07
19.25	1.810	1.474	1.832	844	8.43E-04	2.38E-03	1.97E-07
38.68	1.722	1.354	1.766	1500	4.41E-04	2.38E-03	1.03E-07
19.25	1.726	1.359	1.724				
9.54	1.733	1.369	1.730				
4.75	1.742	1.381	1.738				
9.54	1.739	1.377	1.741	197	3.26E-03	3.30E-04	1.05E-07
19.25	1.731	1.366	1.735	124	5.15E-03	4.34E-04	2.19E-07
38.68	1.709	1.336	1.720	96	6.53E-03	5.96E-04	3.82E-07
77.38	1.612	1.203	1.661	960	6.09E-04	1.32E-03	7.87E-08
154.81	1.513	1.068	1.563	748	6.92E-04	6.73E-04	4.56E-08
309.73	1.423	0.945	1.468	443	1.03E-03	3.06E-04	3.09E-08
618.93	1.337	0.828	1.380	240	1.68E-03	1.46E-04	2.41E-08
1237.83	1.260	0.722	1.299	184	1.94E-03	6.55E-05	1.25E-08
2475.13	1.186	0.621	1.223	146			
1237.83	1.198	0.638					
309.73	1.222	0.670					
77.38	1.254	0.714					
19.25	1.298	0.774					
4.75	1.326	0.812					

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

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.33	Unit Weight, kN/m^3	19.49
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	15.04
Area, cm^2	31.67	Specific Gravity, measured	2.78
Volume, cm^3	41.99	Solids Height, cm	0.732
Water Content, %	29.61	Volume of Solids, cm^3	23.17
Wet Mass, g	83.48	Volume of Voids, cm^3	18.82
Dry Mass, g	64.41		

Prepared By: LFG

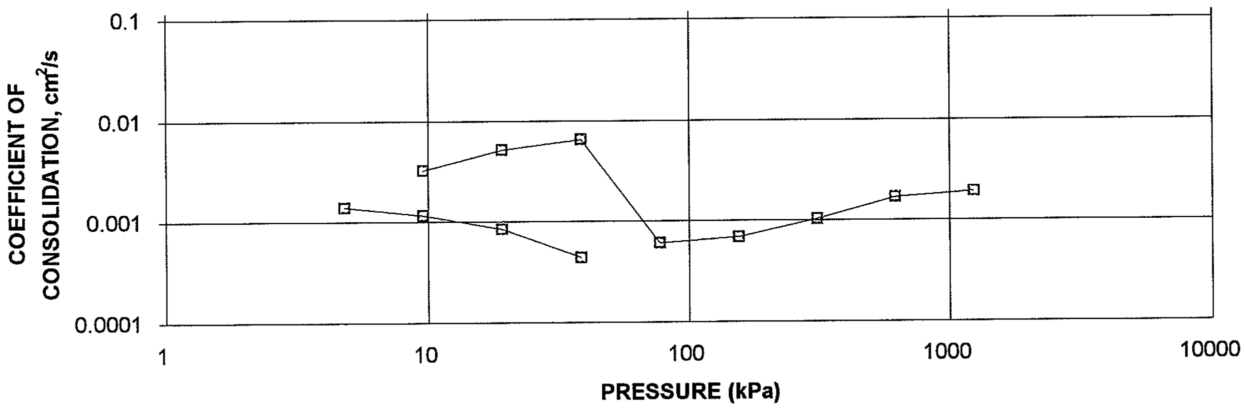
Golder Associates

Checked By: MM

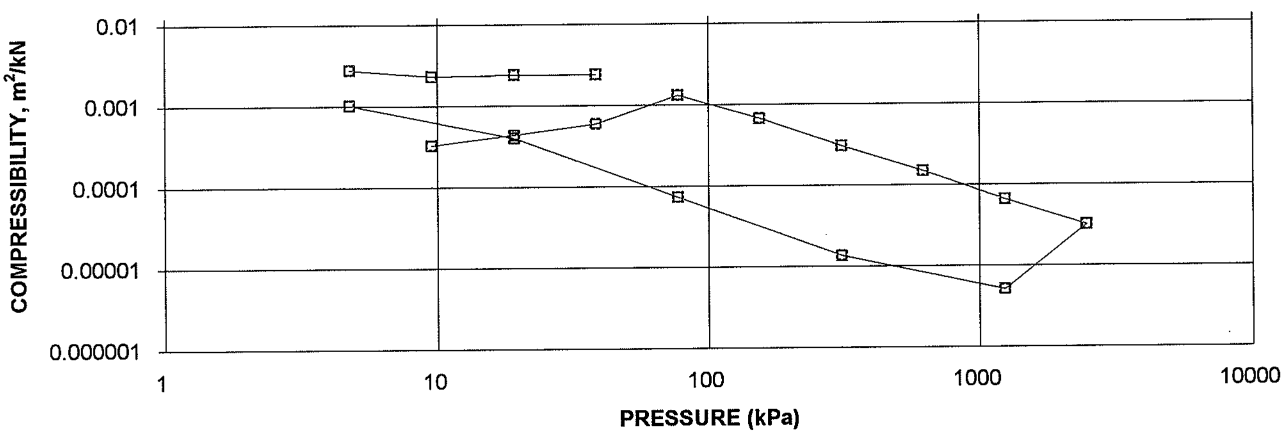
OEDOMETER CONSOLIDATION SUMMARY

Silty Clay to Clay (Borehole E-2)

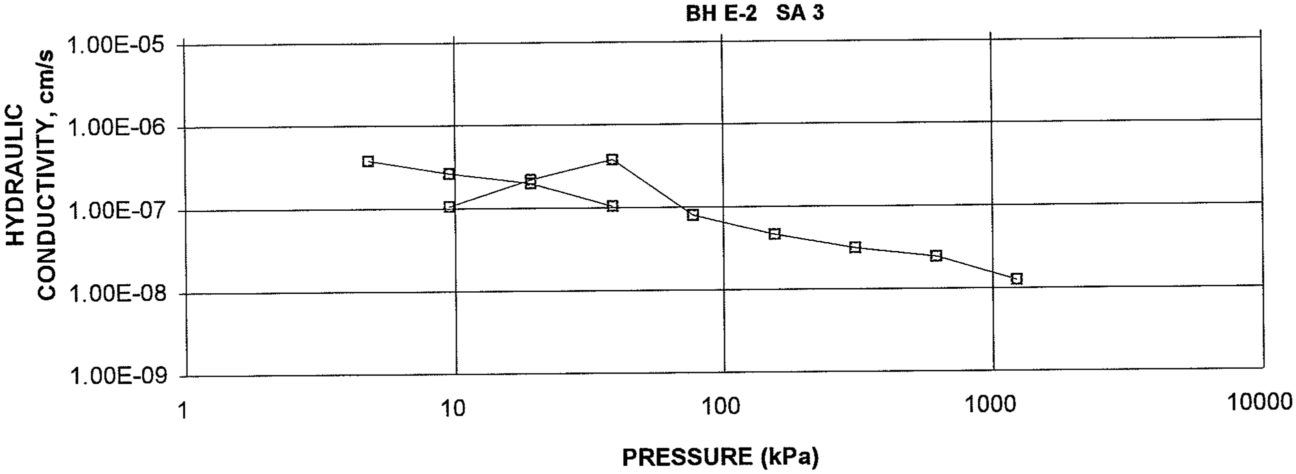
CONSOLIDATION TEST
CV cm^2/s VS PRESSURE (kPa,
BH E-2 SA 3



CONSOLIDATION TEST
MV m^2/kN vs PRESSURE (kPa)
BH E-2 SA 3



CONSOLIDATION TEST
HYDRAULIC CONDUCTIVITY vs PRESSURE
BH E-2 SA 3

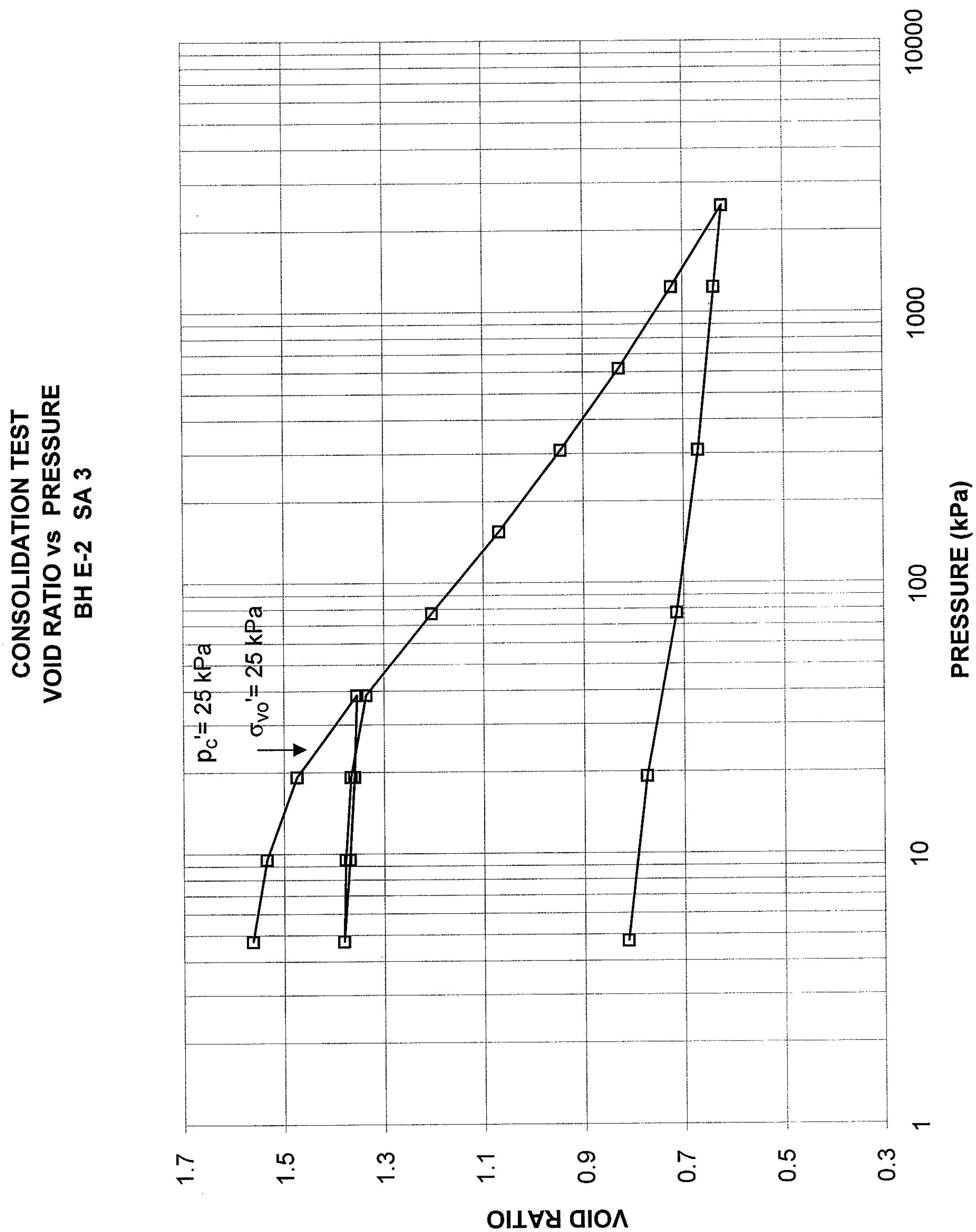


Project No. 05-1111-031
Prepared By: LFG

Golder Associates

Checkey By: MM

CONSOLIDATION TEST **VOID RATIO VS. LOG PRESSURE** **Silty Clay to Clay (Borehole E-2)**	**FIGURE 4** **Page 3 of 4**



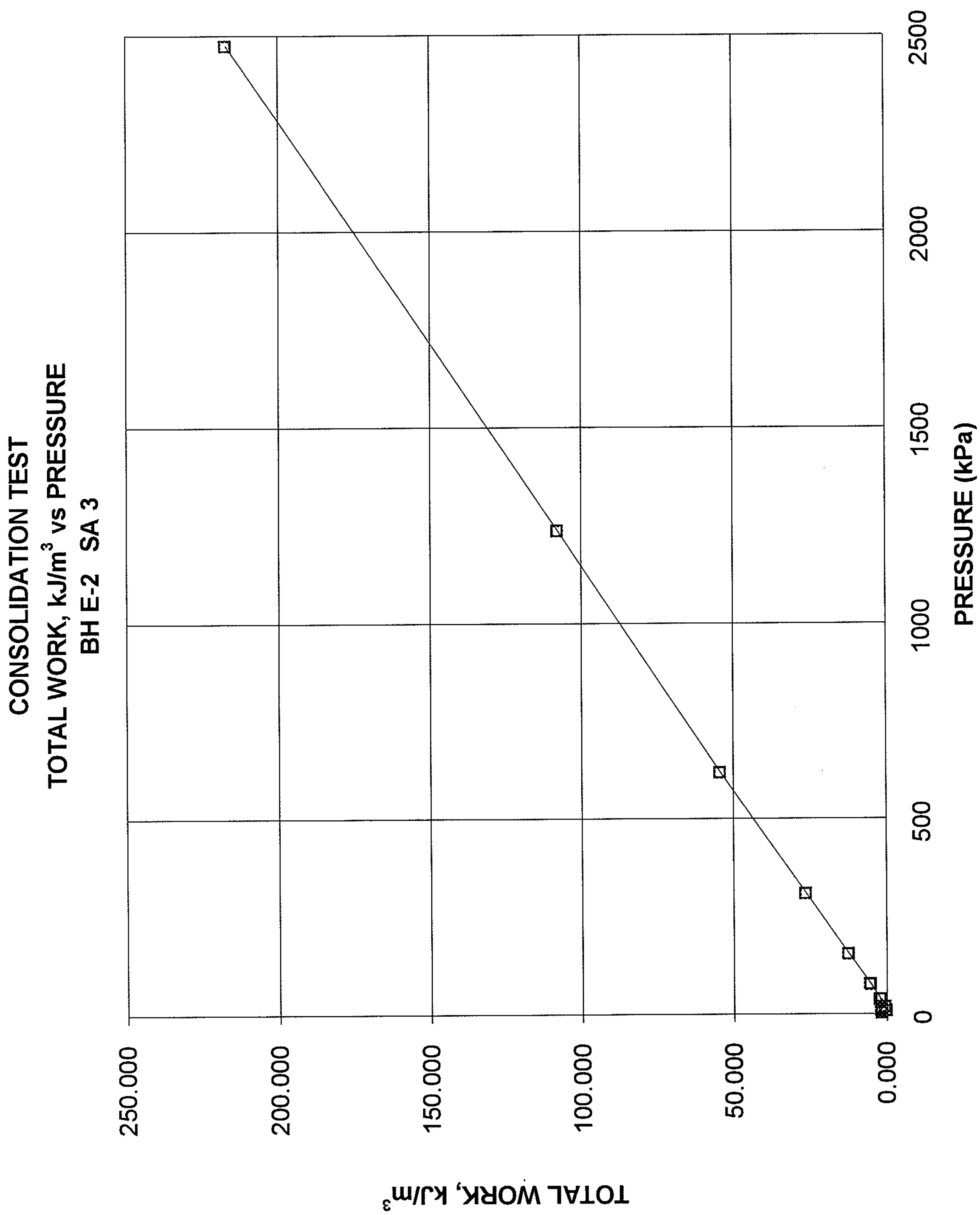
Project No.05-1111-031

Prepared By: LFG

Golder Associates

Checked By: MM

CONSOLIDATION TEST
TOTAL WORK VS. PRESSURE
Silty Clay to Clay (Borehole E-2)



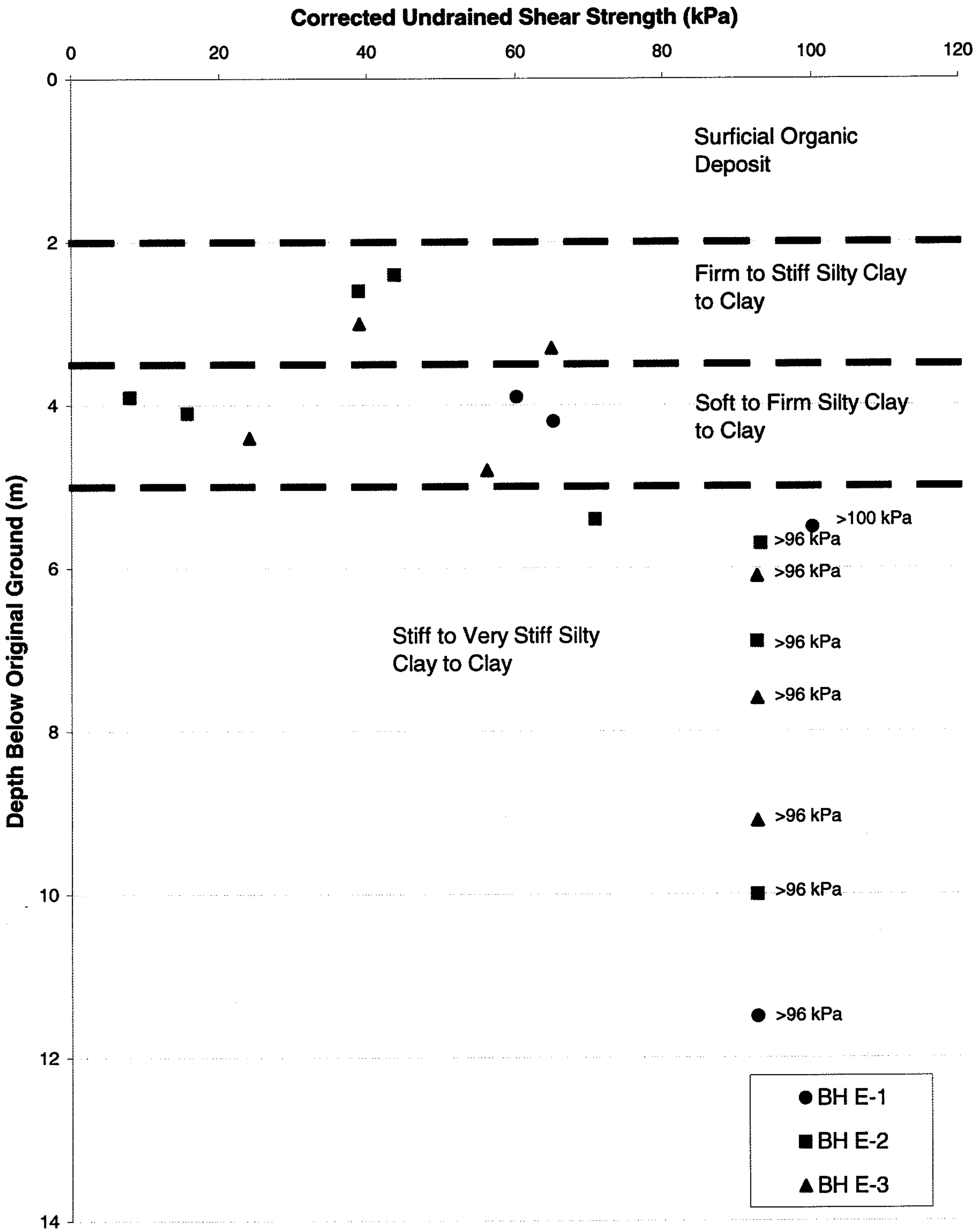
Project No.05-1111-031

Golder Associates

Prepared By: LFG

Checked By: MM

Corrected Undrained Shear Strength Profile Division Street W-N/S Ramp	FIGURE 5

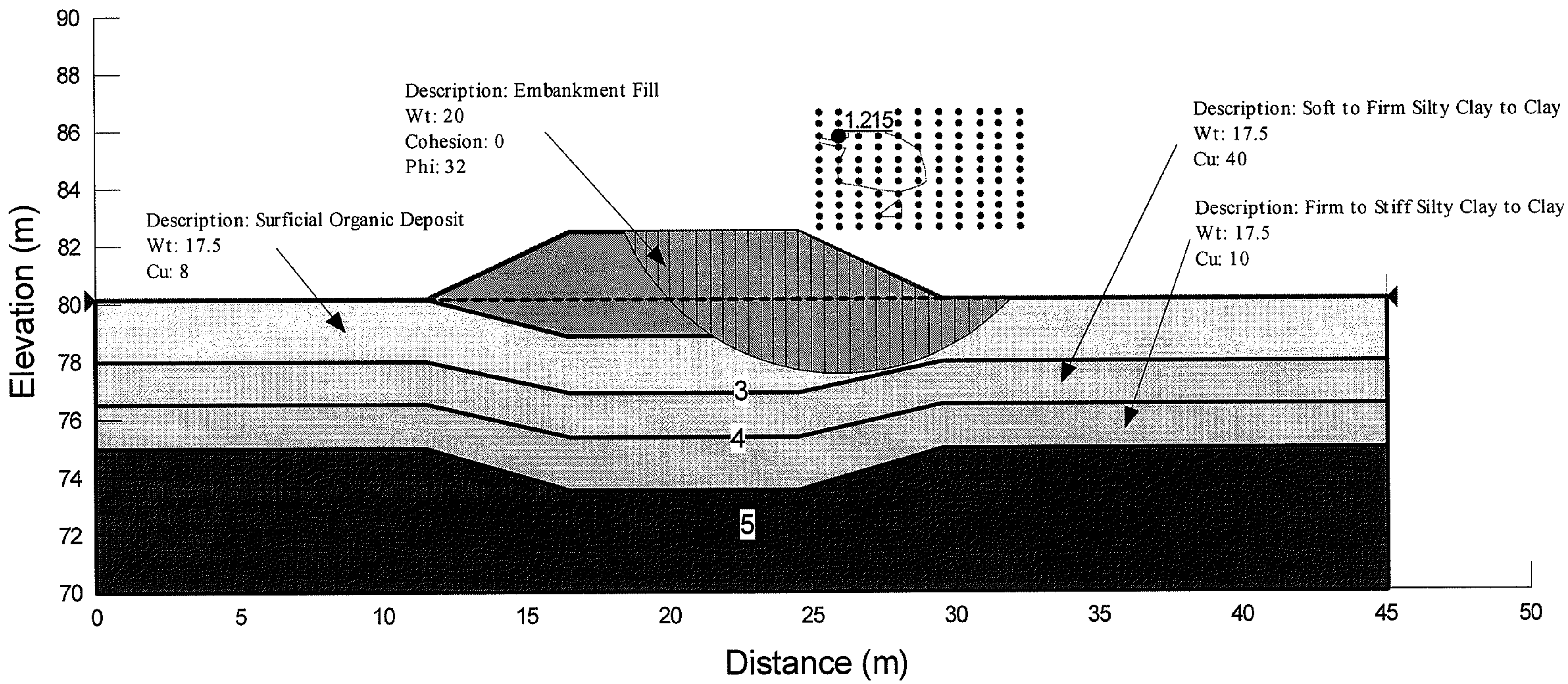


Date: February 2007
Project: 05-1111-031

Golder Associates

Drawn by: BML
Checked by: LCC

Slope Stability Assessment Division Street W-N/S Ramp Embankment	FIGURE 6

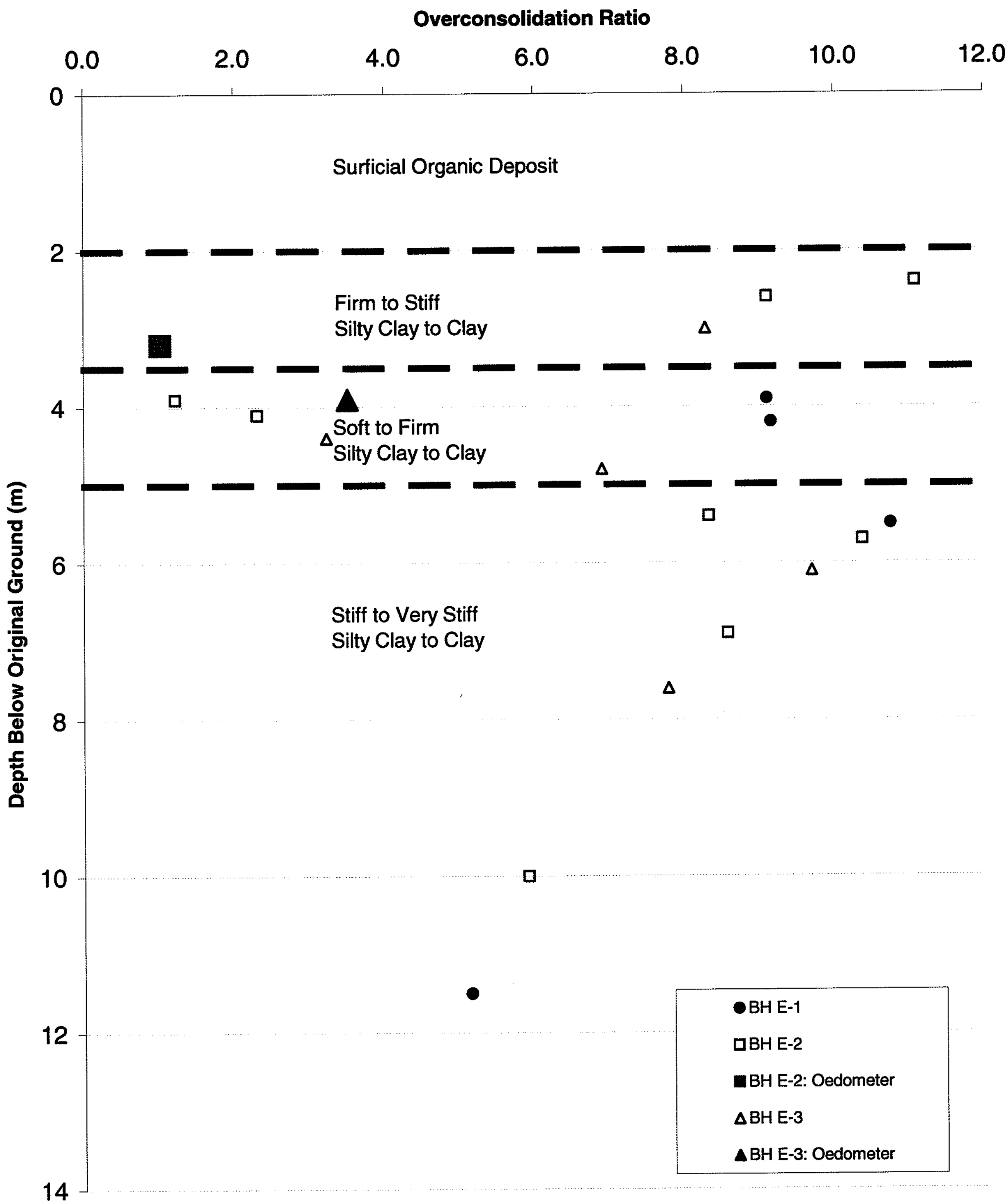


Date: February 2006
Project: 05-1111-031

Golder Associates

Drawn: KEG
Checked: LCC

Overconsolidation Ratio Profile Division Street W-N/S Ramp	FIGURE 7

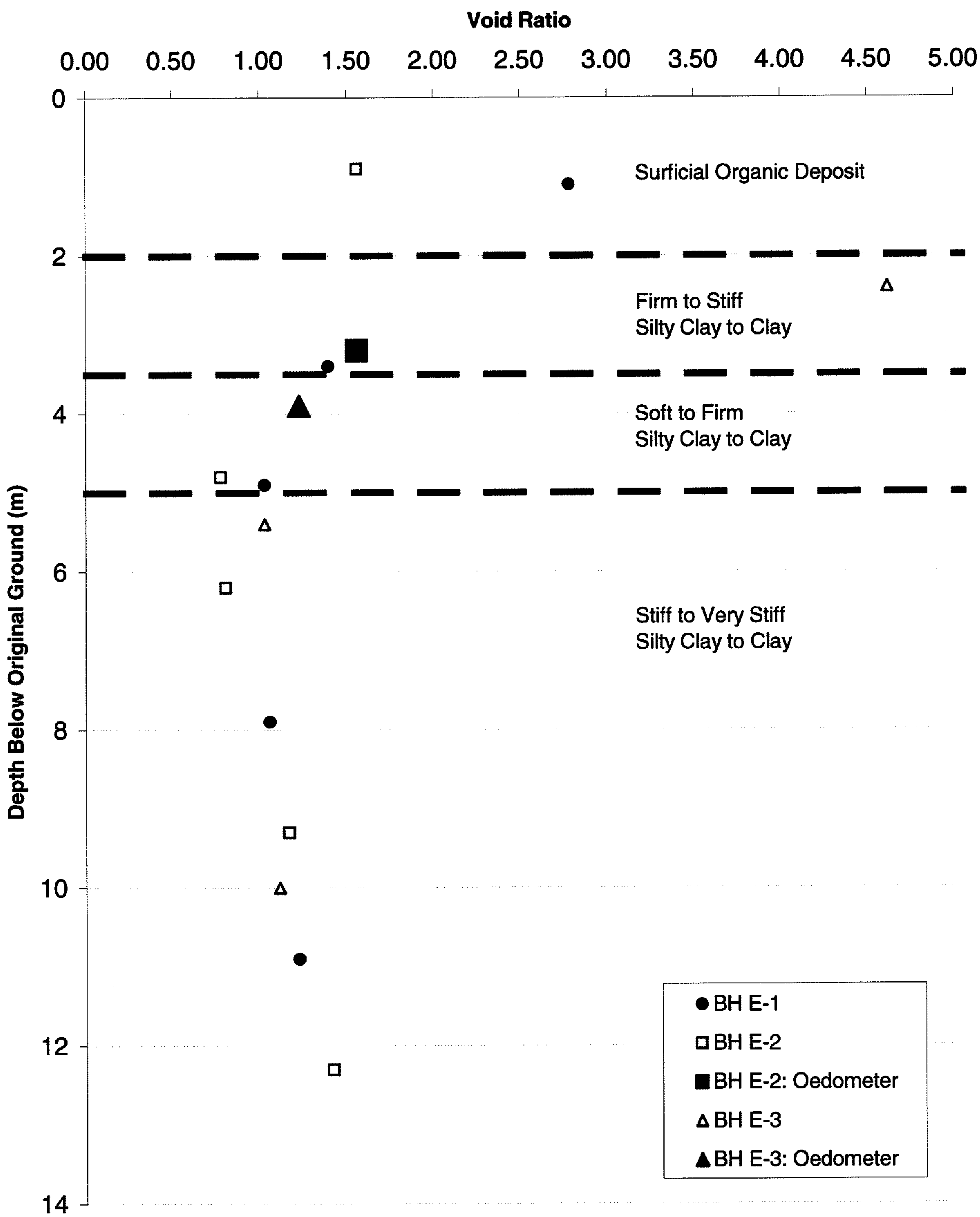


Date: February 2007
Project: 05-1111-031

Golder Associates

Drawn by: BML
Checked by: LCC

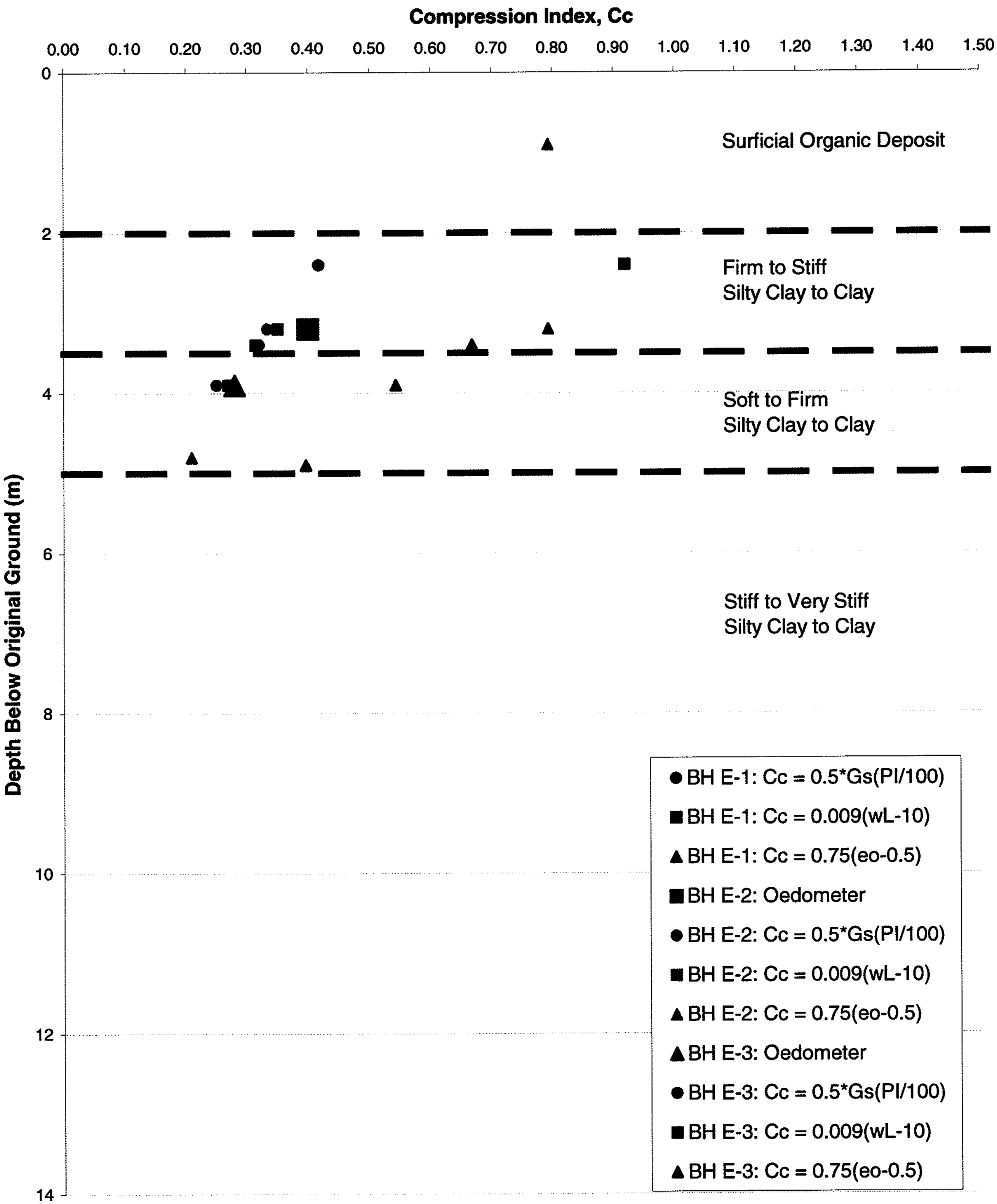
Void Ratio Profile Division Street W-N/S Ramp	FIGURE 8



Date:	February 2007		Drawn by:	BML
Project:	05-1111-031	**Golder Associates**	Checked by:	LCC

Compression Index Profile
Division Street W-N/S Ramp

FIGURE 9

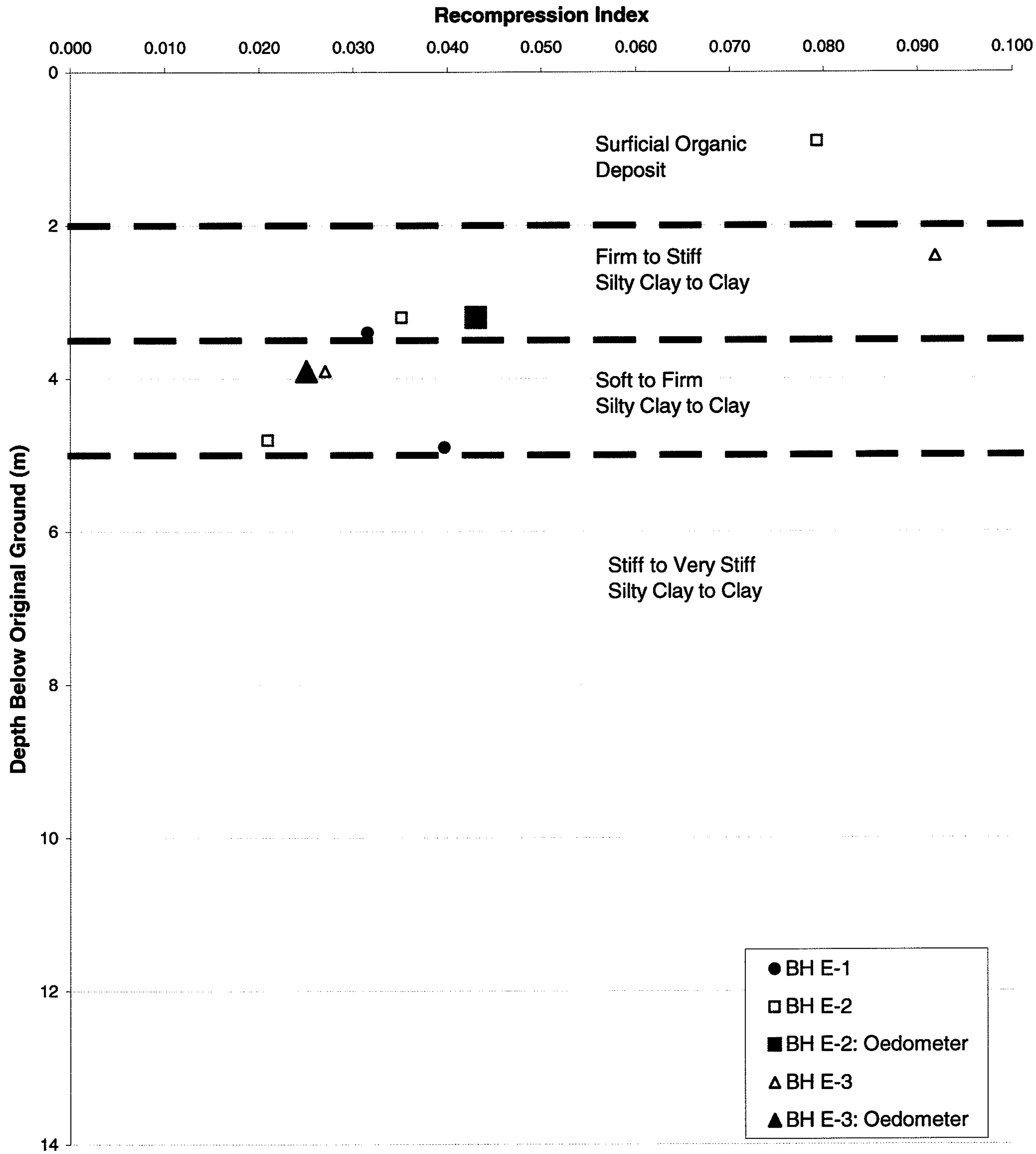


Date:	February 2007	Drawn by:	BML
Project:	05-1111-031	Checked by:	LCC

Golder Associates

Recompression Index Profile
Division Street W-N/S Ramp

FIGURE 10

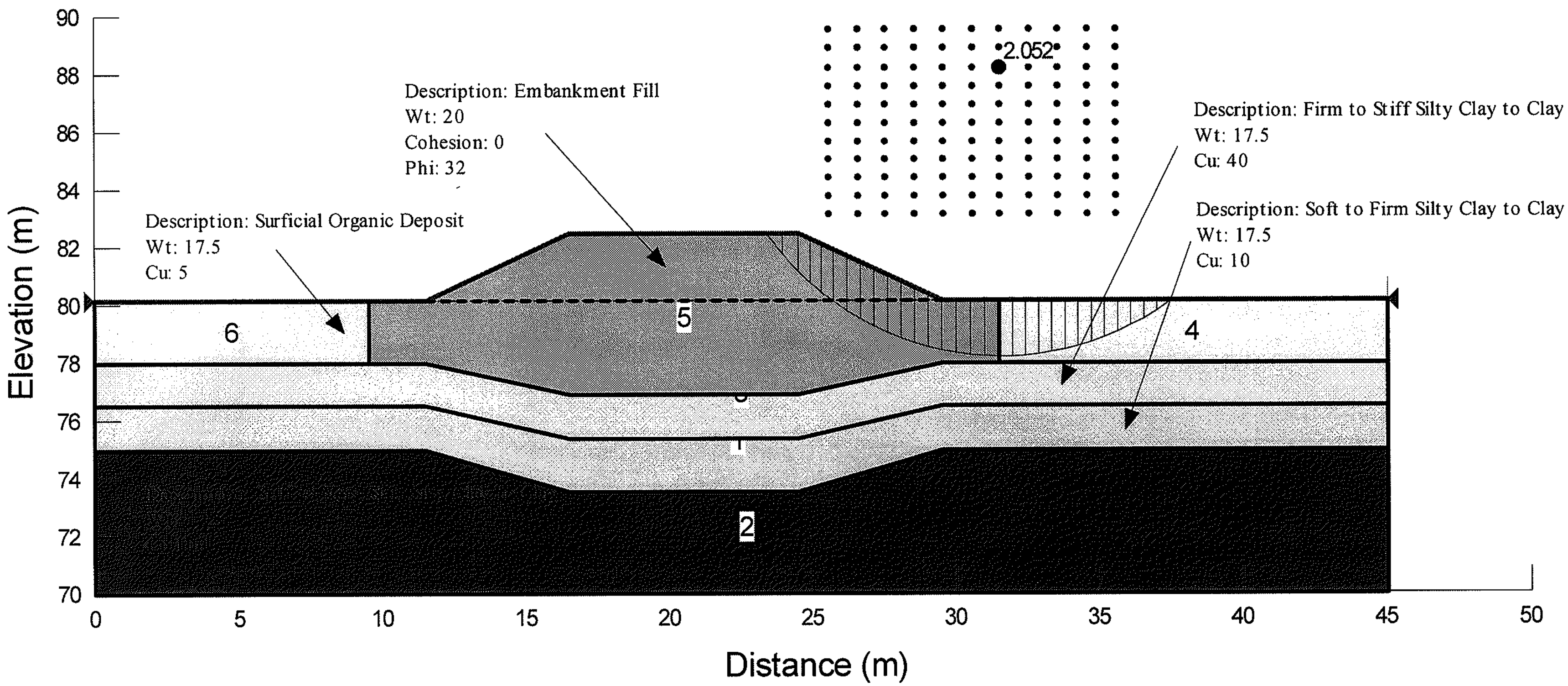


Date: February 2007
Project: 05-1111-031

Golder Associates

Drawn by: BML
Checked by: LCC

Slope Stability Assessment Division Street W-N/S Ramp Embankment Following Sub-Excavation of Surficial Organic Deposit	FIGURE 11



Date: February 2006
Project: 05-1111-031

Golder Associates

Drawn: KEG
Checked: LCC

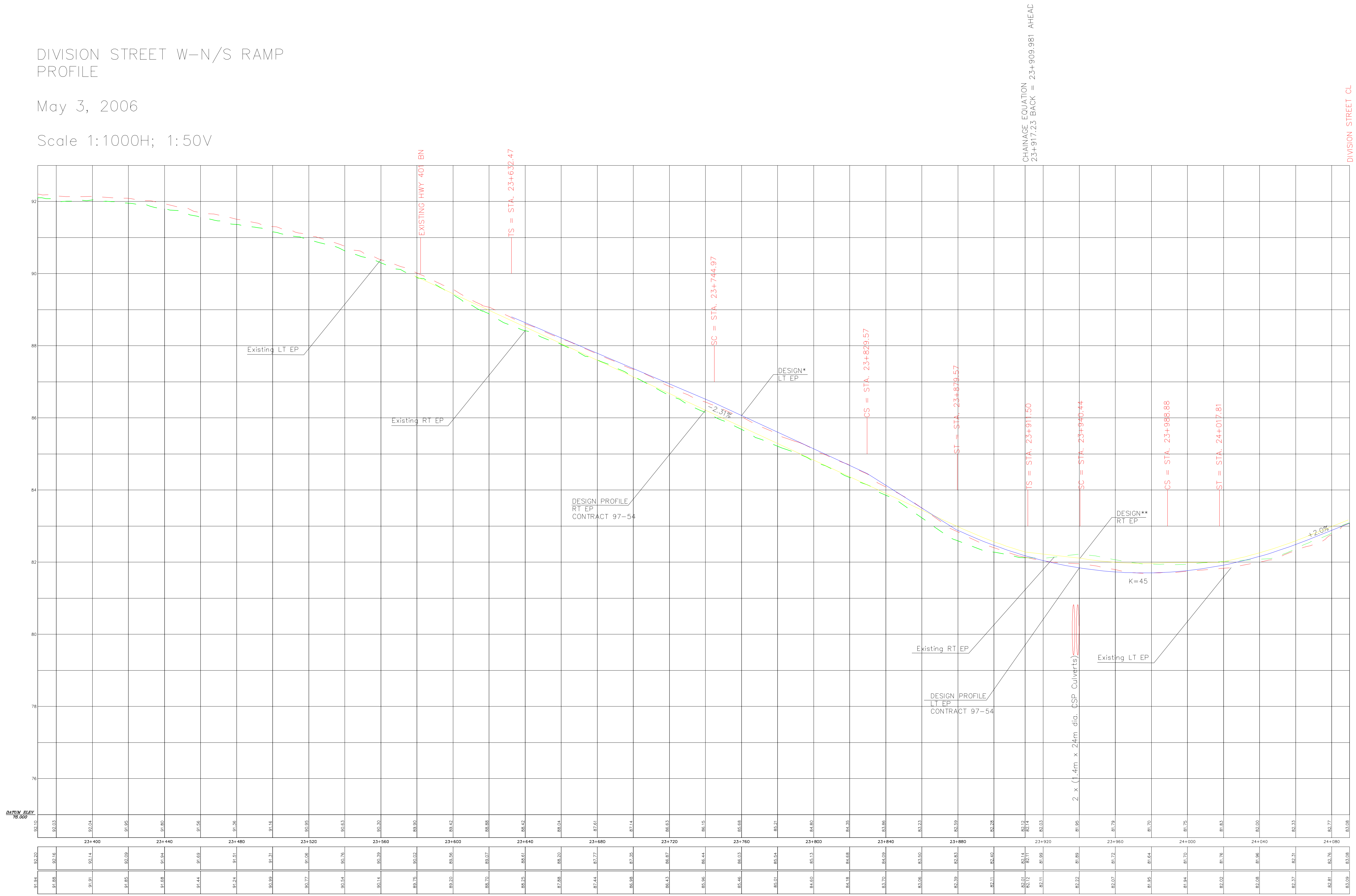
APPENDIX A

DIVISION STREET W-N/S RAMP PROFILE

DIVISION STREET W-N/S RAMP
PROFILE

May 3, 2006

Scale 1:1000H; 1:50V



NOTES:
*Projected at standard rate of superelevation (e=6%) for an R=200m curve from Contract 97-54 Design Profile.
**Projected at standard rate of superelevation (e=5.6%) for an R=70m curve from Contract 97-54 Design Profile.
***A constant 4.75m EP to EP pavement width is used throughout for projection projection purposes.
****Existing ground EP profiles taken along alignment centreline and at a 4.75m constant offset from the alignment centreline.

PLOT DATE: February 16, 2007
FILENAME: T:\Projects\2005\05-1111-031 (MRC, Kingston, Ontario)\-DA- Appendix\0511111031DA001.dwg

APPENDIX B

NON-STANDARD SPECIAL PROVISIONS

OPERATIONAL CONSTRAINT

Special Provision

Protection of Subgrade Soils at Division Street W-N/S Ramp Site

In order to limit disturbance to the clayey subgrade soils that will be exposed during subexcavation at the Division Street W-N/S Ramp site, following stripping of the surficial organic-containing deposit:

- Construction equipment shall not travel over the clayey subgrade soils until Granular "B" Type II fill is placed on top of the clayey subgrade. The Granular "B" fill should be end-dumped and spread with a light bulldozer to a minimum thickness of 0.3 m before any truck or heavy equipment traffic is permitted to work over the subexcavation area for reconstruction of the ramp embankment.