REPORT

May 2010

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FOUNDATION INVESTIGATION AND DESIGN REPORT

Culvert Replacement at Station 9+950 Division Street Highway 401 Widening, Kingston, Ontario G.W.P. 4508-02-00

Submitted to:
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Report Number: 05-1111-031-6

GEOCRES No. 31C-197

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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 replacement of an existing 30 m long, open footing culvert located approximately 20 m to 30 m north of Highway 401 on Division Street, in Kingston, Ontario. This culvert replacement is part of the Division Street interchange improvements, including construction of a new S-W Ramp, under G.W.P. 4508-02-00.

The scope of work for the foundation engineering services is outlined in Golder's proposal submitted to McCormick Rankin Corporation, dated February 8, 2010.

2.0 SITE DESCRIPTION

The culvert passes beneath Division Street, approximately 20 m to 30 m north of Highway 401 at about Station 9+950, in the City of Kingston, in Frontenac County.

The natural ground surface in the area of the culvert is relatively flat, at about Elevation 80.5 m to 82 m. Division Street has been constructed at or near the original ground surface, with its grade in the vicinity of Highway 401 at about Elevation 82.5 m, and in the vicinity of the existing culvert at about Elevation 82.8 m. The existing culvert is an open footing concrete structure, approximately 30 m long, with a span and height of 1.85 m and 1.25 m, respectively. The existing culvert obvert is at about Elevation 82.05 m, and the channel base is at about Elevation 81.2 m.

Observations of the existing culvert structure and adjacent Division Street pavements and low embankment side slopes at the time of the investigation indicate that there does not appear to be any settlement- or instability-related distress to the existing structure or pavement.

3.0 INVESTIGATION PROCEDURES

The field work for this subsurface investigation was carried out in February 2010, at which time a total of three boreholes (Boreholes C-1, C-2, and C-4) 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 C-1 and C-2 were advanced immediately west and east of the shoulders of Division Street to depths of 10.3 m and 8.1 m, respectively. Borehole C-4 was advanced on the east edge of the Division Street pavement in the northbound lane, to a depth of 7.3 m.

Soil samples were obtained at 0.75 m intervals of depth, using a 50 mm outside diameter split-spoon sampler 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.

A standpipe piezometer was installed in Borehole C-4, within the silty clay to clay deposit. The piezometer consists of 25 mm diameter PVC pipe with a slotted tip installed within a 2.0 m thick filter sand pack. A 0.9 m thick bentonite seal was placed on top of the filter sand, followed by about 2.8 m of the silty clay soil, with a 0.6 m thick bentonite seal placed immediately below the ground surface. The remaining boreholes were backfilled to the ground surface using bentonite, in places mixed with native silty clay to clay soil (cuttings from the borehole), in accordance with Ontario Regulation 128 (amendment to Ontario Regulation 903).

The field work was supervised on a full-time basis by a member 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 laboratories in Ottawa and 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; and oedometer (consolidation) testing was completed on two relatively undisturbed thin-walled Shelby tube samples of the firm to stiff portion of the silty clay to clay deposit.

The borehole locations and ground surface elevations were measured using a Trimble Global Positioning System (GPS). The borehole locations, including MTM NAD83 northing and easting coordinates and ground surface elevations referenced to geodetic datum, are summarized in the following table and are shown on Drawing 1.

Borehole Number	Borehole Location	MTN NAD83 Northing (m)	MTN NAD83 Easting (m)	Ground Surface Elevation (m)
C-1	West of SB shoulder	4,903,292.2	304,852.6	81.8
C-2	East of NB shoulder	4,903,294.4	304,881.1	82.8
C-4	East edge of NB lane	4,903,293.3	304,874.8	82.7

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* (Chapman and Putnam, 1984). The Napanee Plain is flat to undulating, and is characterized by relatively shallow soil deposits overlying bedrock. Geologic mapping indicates that the bedrock within the Napanee Plain consists of grey limestone of the Gull River Formation (of the Trenton-Black River Group), which contains some shale partings and seams (Ontario Geological Society, 1991).

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 southern portion of the Napanee Plain, and within and adjacent to river valleys throughout the Plain.

4.2 Site Stratigraphy

As part of the subsurface investigation at the Division Street culvert site, three boreholes were advanced near the existing culvert. The borehole locations, ground surface elevations and interpreted stratigraphic conditions 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 borehole records and Figures B1 to B5 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 and bedrock conditions will vary between and beyond the borehole locations.

In summary, the soils encountered at the culvert site consist of fill overlying an approximately 2.4 m to 3.1 m thick "weathered crust" of silty clay to clay, which is underlain by an unweathered silty clay to clay deposit. Bedrock was not encountered in this investigation, although it was encountered in boreholes advanced in a previous investigation at the Division Street overpass site, approximately 20 m to 30 m to the south of the culvert site; at that location, bedrock was encountered at a depth of approximately 11 m to 13 m below the Division Street grade.

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

4.2.1 Crushed Stone to Sand and Gravel Fill

Fill was encountered immediately below the ground surface in Boreholes C-1 and C-2, and below the asphalt in Borehole C-4. The fill is approximately 1.4 m to 1.5 m thick as encountered in these boreholes, with its base between Elevations 80.4 m and 81.4 m, rising from west to east.

The encountered fill varies in composition from grey crushed stone at the top of the fill, to sand to sand and gravel at the base of the fill. The results of grain size distribution tests completed on two samples of the sand and gravel/crushed stone fill are shown on Figure B1 in Appendix B.

The measured Standard Penetration Test (SPT) "N" values within the fill range from 4 to 9 blows per 0.3 m of penetration, indicating that the fill has a loose relative density.

4.2.2 Silty Clay to Clay (Weathered Crust)

A 2.4 m to 3.1 m thick "weathered crust" of grey-brown silty clay to clay, containing trace sand, was encountered underlying the fill in all of the boreholes. The surface of this deposit was encountered at a depth of 1.4 m to 1.5 m (Elevation 80.4 m to 81.4 m, and its base was encountered at a depth of 3.8 m to 4.6 m (Elevation 77.2 m to 79.0 m).

The results of grain size distribution tests carried out on two selected samples of the silty clay to clay weathered crust are provided on Figure B2 in Appendix B. Atterberg limits testing was carried out on four selected samples of the weathered crust. The measured plastic limits vary from 22 to 29 per cent, the liquid limits from 56 to 67 per cent, and the plasticity indices from 27 to 45 per cent. These results, which are plotted on a plasticity chart on Figure B3 in Appendix B, confirm that the deposit generally consists of high plasticity clay.

The measured SPT "N" values within this material vary from 6 to 25 blows per 0.3 m of penetration; however, based on observation of the recovered samples and experience with similar soils in this area, the silty clay to clay comprising the weathered crust has a very stiff to hard consistency.

4.2.3 Silty Clay to Clay (Unweathered)

The weathered crust is underlain by an unweathered deposit of grey silty clay to clay, containing trace sand. The surface of this deposit was encountered between Elevations 77.2 m and 79.0 m, and its base is inferred between Elevations 71.5 m and 75.4 m based on resistance to vane penetration as encountered in Boreholes C-1 and C-4, or split-spoon and auger refusal as encountered in Borehole C-2. The unweathered portion of the silty clay to clay deposit therefore has a thickness of approximately 3.5 m to 5.7 m.

The results of grain size distribution tests carried out on two selected samples of the unweathered portion of the silty clay to clay deposit are provided on Figure B4 in Appendix B. Atterberg limits testing was carried out on five selected samples of this deposit. The measured plastic limits vary from 24 to 28 per cent, the liquid limits from 57 to 64 per cent, and the plasticity indices from 32 to 38 per cent. These results, which are plotted on a plasticity chart on Figure B5 in Appendix B, confirm that the deposit generally consists of high plasticity clay.

The measured SPT "N" values within the unweathered portion of the silty clay to clay deposit range from 3 to 8 blows per 0.3 m of penetration. In situ vane testing was carried out within the silty clay in Boreholes C-1 and C-3, typically measuring undrained shear strengths ranging from 46 kPa to 86 kPa, with the higher values found near the top and bottom portions of the unweathered silty clay to clay. The vane test results indicate that the unweathered, grey portion of the silty clay to clay deposit has a firm to stiff consistency.

Remoulded shear strengths were also measured to assess the sensitivity of the silty clay to clay deposit. Based on these results, the sensitivity of the clay varies from approximately 4.3 to 7.1; these results indicate that the clay has a high sensitivity (*Canadian Foundation Engineering Manual, 2006*).

Laboratory oedometer (consolidation) testing was carried out on two specimens of the unweathered silty clay to clay obtained from Boreholes C-1 and C-4. Details of the test results are shown on Figures B6 and B7 in Appendix B, and the results are summarized in the following table.

Borehole and Sample No.	Elevation (m)	σ_{vo}' (kPa)	σ_p' (kPa)	$\sigma_p' - \sigma_{vo}'$ (kPa)	OCR	e_o	C_r	C_c	c_v (cm^2/s)
C-1 Sa 7	75.5	85	400	315	4.7	1.11	0.056	0.50	6.0×10^{-3}
C-4 Sa 9	76.4	60	160	100	2.7	1.62	0.072	0.75	3.0×10^{-3}

σ_{vo}' is the effective overburden pressure
σ_p' is the preconsolidation pressure
OCR is the overconsolidation ratio
e_o is the initial void ratio
C_c is the compression index
C_r is the recompression index
c_v is the coefficient of consolidation

Based on the oedometer test results summarized above, the silty clay to clay deposit at the culvert location is overconsolidated.

4.2.4 Clayey Silt Till

Approximately 25 mm of clayey silt till was encountered at the base of Borehole C-2, underlying the unweathered grey silty clay to clay. This thin layer of till was encountered at a depth of about 8.1 m (Elevation 74.7 m). Borehole C-2 was terminated at this depth upon refusal to split-spoon and auger advance.

4.3 Groundwater Conditions

A standpipe piezometer was installed and screened within the unweathered portion of the silty clay to clay deposit in Borehole C-4; details of the piezometer installation are shown on the borehole record contained in Appendix A, following the text of this report. The water level measurements in the piezometer installed in Borehole C-4 are summarized in the following table.

Date	Depth to Groundwater Level	Groundwater Elevation
February 24, 2010 (on completion of drilling)	Dry	Dry
March 19, 2010	0.8 m	81.9 m

Based on the measurements summarized above, the stabilized groundwater level in the area is typically at or near the ground surface. This result is similar to the groundwater monitoring information obtained at the Division Street overpass site approximately 20 m to 30 m south of this culvert site. The water level at the culvert 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 Ms. Nikol Kochmanová, EIT, and reviewed by Ms. Lisa Coyne, P.Eng., a geotechnical engineer and Principal with Golder. Mr. Fin Heffernan, P.Eng., a Designated MTO Contact for Golder, conducted an independent review of this report.

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PART B

FOUNDATION DESIGN REPORT
CULVERT REPLACEMENT AT STATION 9+950 DIVISION STREET
HIGHWAY 401 WIDENING
KINGSTON, ONTARIO
G.W.P. 4508-02-00

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Report No. 05-1111-031-6

6.0 FOUNDATION ENGINEERING RECOMMENDATIONS

6.1 General

This section of the report provides foundation design recommendations for the detail design of the replacement of the existing culvert located beneath Division Street at approximately Station 9+950, just north of Highway 401 in the City of Kingston, in Frontenac County. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during this subsurface investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and to carry out the design of the structure foundations.

Where comments are made on construction, they are provided to highlight those aspects that 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 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.

6.2 Foundation Options

The existing culvert consists of a 30 m long, 1.85 m wide by 1.25 m high open footing structure. As part of the Division Street interchange improvements, this structure is to be replaced by a 3.0 m wide by 1.5 m high culvert structure, also 30 m long, which will be constructed on the same alignment as the existing culvert. The obvert of the existing culvert, and that for the proposed replacement, is at approximately Elevation 82.05 m. The final Division Street pavement grade over the new culvert will be lowered by approximately 250 mm relative to the existing grade, from about Elevation 82.8 m to about Elevation 82.55 m. A headwall and wing walls will be required at the east end of the new culvert; based on the site geometry, it is anticipated that the wing walls will be 5 m to 6 m in length, and about 0.5 m to 1.5 m high relative to the channel base.

Either a box culvert or an “open footing” (shallow foundation) culvert is feasible for the replacement of this structure; associated wing walls should be supported on shallow foundations. Deep foundations are not required for the replacement culvert or wing walls, as shallow foundations will provide sufficient bearing resistance and acceptable settlement performance. Both pre-cast concrete elements (box culvert segments or footing elements) and cast-in-place concrete elements are also feasible from a foundations perspective.

The advantages and disadvantages associated with both the box culvert and open footing culvert replacement options are summarized in Table 1 following the text of this report; this table also includes comments on the use of pre-cast concrete box culvert segments or pre-cast concrete footing sections versus cast-in-place concrete. From a foundations perspective, a pre-cast box culvert is preferred over a cast-in-place open footing culvert in terms of minimizing the depth of excavation and groundwater control requirements compared with open footings; in addition, pre-cast box culvert segments can often be installed more expeditiously than cast-in-place open footing culverts, resulting in shorter durations for dewatering and surface water pumping. However, a box culvert replacement may not satisfy fisheries requirements related to channel substrate, in which case an open footing culvert is geotechnically feasible (though not preferred).

Recommendations for both a box culvert replacement and a shallow foundation (open footing) culvert replacement are provided in the following sections.

6.3 Box Culvert Replacement

6.3.1 Founding Elevation

A box culvert replacement should be founded below any existing fill, supported on the very stiff to hard "weathered crust" portion of the silty clay to clay deposit. It is not necessary to found box culverts at a minimum depth for frost protection purposes.

Based on a design obvert level of Elevation 82.05 m, and assuming that the replacement structure has a height of 1.5 m and a base slab thickness of 250 mm, the invert of the replacement box culvert would be at about Elevation 80.3 m. It is recommended that a minimum 300 mm thick layer of Ontario Provincial Standard Specification 1010 *(Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material)* Granular A be placed below the base slab on the subgrade to form a bedding layer for the box culvert segments, and to limit the degradation of the sensitive clay subgrade. The bedding should be placed within four hours after inspection and approval of the subgrade to limit such degradation.

Excavations for the box culvert replacement would extend approximately 1.6 m below the groundwater level at the site, as measured in the piezometer in Borehole C-4. Some perched groundwater should be expected in the granular fill of the Division Street pavement structure/embankment, and minor seepage is anticipated from seams or lenses that may be present within the silty clay to clay deposit. Groundwater and surface water control will therefore be required for construction of the box culvert replacement; further discussion on this aspect is provided in Section 6.8 (Construction Considerations).

The sensitive silty clay to clay subgrade will be susceptible to disturbance and degradation on exposure to water and construction traffic. As an alternative to the placement of a minimum 300 mm thick layer of Granular A, a 100 mm thick concrete working slab could be placed on the subgrade within the culvert footprint, to protect the subgrade from such degradation. In this case, a 75 mm thick layer of OPSS 1010 Granular A or concrete fine aggregate meeting the gradation requirements set out in OPSS 1002 *(Material Specification for Aggregates - Concrete)* should be placed on top of the concrete mat to provide a "levelling pad" for the box culvert replacement. The working slab should be placed within four hours after inspection and approval of the subgrade.

6.3.2 Geotechnical Resistance

A box culvert placed on the properly prepared subgrade, at or below founding elevation identified above, should be designed based on the following factored geotechnical resistance at Ultimate Limit States (ULS) and geotechnical resistance at Serviceability Limit States (SLS):

Culvert Span	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS*
3.0 m	225 kPa	150 kPa

* For 25 mm of total settlement.

The ULS resistance and settlement are dependent on the footing size, configuration and applied loads; the geotechnical resistances should, therefore, be reviewed if the culvert span or founding elevation differs significantly from that given above.

The geotechnical resistances provided above are based on loading applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Section 6.7.2 of the *Canadian Highway Bridge Design Code (CHBDC).*

6.3.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the culvert base slab and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The following values for the coefficient of friction, tan ϕ' or tan δ, can be used between the pre-cast concrete box culvert segments and the Granular A bedding, and between the bedding or the working slab and the properly prepared, very stiff to hard silty clay to clay subgrade:

Footing Type	Coefficient of Friction
Precast box culvert on Granular A bedding	tan δ = 0.5
Granular A bedding on very stiff to hard silty clay to clay subgrade	tan ϕ' = 0.5
Concrete working slab on very stiff to hard silty clay to clay subgrade	tan ϕ' = 0.45

6.4 Open Footing Culvert Replacement and Shallow Foundations for Wing Walls/Retaining Walls

6.4.1 Founding Elevation

An open footing culvert replacement, and any associated wing walls or retaining walls, can be supported on strip footings founded below the existing fill on the very stiff to hard, “weathered crust” portion of the silty clay to clay deposit. Strip footings should be founded at a minimum depth of 1.5 m below the lowest surrounding grade, to provide adequate protection against frost penetration, as per OPSD 3090.101 *(Foundation Frost Depths for Southern Ontario).* The maximum (highest) founding level for the replacement culvert footings and wing wall or retaining wall footings is provided in the following table, based on a channel base elevation of 81.2 m; a minimum (lowest) founding level is also provided in this table, to maintain the footings within the stiffer weathered crust.

Channel Base Elevation	Maximum (Highest) Founding Elevation	Minimum (Lowest) Founding Elevation
81.2 m	79.7 m	79.3 m

If a box culvert replacement is adopted in conjunction with wing walls, the footings for the wing walls will be up to about 0.6 m below the invert level of the culvert. It is recommended that excavation and construction of the wing wall footings be completed before placement of the box culvert sections, to avoid undermining the culvert end as could potentially occur if the wing walls were constructed after the culvert.

The maximum founding level identified above will require excavation to a depth of up to about 2.2 m below the groundwater level at the site, as measured in the piezometer in Borehole C-4. Some perched groundwater should be expected in the granular fill of the Division Street pavement structure/embankment, and minor seepage is anticipated from seams or lenses that may be present within the silty clay to clay deposit. Groundwater and surface water control will therefore be required for construction of the footings; further discussion on this aspect is provided in Section 6.8 (Construction Considerations).

The subgrade for the culvert and/or wall footings will be susceptible to disturbance and degradation on exposure to water and construction traffic. It is strongly recommended that a 100 mm thick concrete working slab be placed within four hours following inspection and approval of the subgrade, to protect the subgrade from softening; this aspect is discussed further in Section 6.8 (Construction Considerations).

6.4.2 Geotechnical Resistance

Strip footings placed on the properly prepared subgrade, at or below the maximum (highest) founding elevation identified above, should be designed based on the following factored geotechnical resistances at ULS and geotechnical resistances at SLS.

Footing Width	Factored Geotechnical Resistance at ULS	Geotechnical Resistance at SLS*
0.6 m	150 kPa	125 kPa
0.9 m	160 kPa	125 kPa
1.5 m	180 kPa	125 kPa

* For 25 mm of total settlement for the given footing width.

The structural engineer must ensure that the selected footing width is sufficient to resist overturning. The ULS resistance and settlement are dependent on the footing size, configuration and applied loads; the geotechnical resistances should, therefore, be reviewed if the selected footing width or founding elevation differs significantly from those given above.

The geotechnical resistances provided above are given under the assumption that the loads will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the surface of the footing, inclination of the load should be taken into account in accordance with Section 6.7.2 of the *CHBDC.*

6.4.3 Resistance to Lateral Loads

Resistance to lateral forces / sliding resistance between the concrete footings and the subgrade should be calculated in accordance with Section 6.7.5 of the *CHBDC*. The following values for the coefficient of friction, tan ϕ' or tan δ, can be used for cast-in-place and pre-cast concrete footings founded on a concrete working slab, and for the concrete working slab on the properly prepared, very stiff to hard silty clay to clay subgrade:

Footing Type	Coefficient of Friction
Cast-in-place concrete footing on concrete working slab	tan δ = 0.55
Pre-cast concrete footing on concrete working slab	tan δ = 0.45
Concrete working slab on very stiff to hard silty clay to clay subgrade	tan ϕ' = 0.45

6.4.4 Global Stability of Wing Walls/Retaining Walls

It is understood that wing walls or retaining walls are required at the east end of the replacement culvert, and that these wing walls will be about 0.5 m to 1.5 m high relative to the creek channel base. At 1.5 m high, the top of the wing wall will be slightly below the finished grade of Division Street.

The static global stability of maximum 1.5 m high wing walls has been analyzed using the commercially available program SLOPE/W produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the factor of safety of numerous potential failure surfaces was computed in order to establish the minimum factor of safety. A target factor of safety of 1.5 against global failure of concrete retaining walls would normally be used for design under static conditions. This factor of safety is considered appropriate for the concrete walls at this site, considering the design requirements and the field data available.

Based on the analysis results, the factor of safety against global instability of a 1.5 m high concrete wing wall/retaining wall at this site is greater than 1.5.

6.5 Culvert Bedding, Backfill and Erosion Protection

For a box culvert replacement, the bedding levelling pad and backfill requirements should be in accordance with OPSS 422 *(Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut)* for pre-cast rigid frame culverts. Box culvert replacements should be provided with at least 300 mm of OPSS 1010 Granular A material for bedding purposes, or alternatively a 100 mm thick concrete working slab with 75 mm of bedding material, as discussed in Section 6.3.

Backfill and cover for concrete culverts should be completed in accordance with Ontario Provincial Standard Drawing (OPSD) 803.010 *(Backfill and Cover for Concrete Culverts)*. Backfill to box culvert walls should consist of granular fill meeting the requirements of OPSS 1010 Granular A or Granular B Type II, but with less than 5 per cent passing the No. 200 sieve. The backfill should be placed and compacted in accordance with MTO's Special Provision SP105S10 (Amendment to OPSS 501). The fill depth during placement should be maintained equal on both sides of the culvert walls, with one side not exceeding the other by more than 500 mm. The culvert replacement should be designed for the full overburden pressure and live load, assuming an embankment fill unit weight of 22 kN/m^3 for Granular A, and 21 kN/m^3 for Granular B Type II or select earth fill above and/or surrounding the culvert.

To prevent surface water from flowing either beneath the culvert (potentially causing undermining and scouring) or around the culvert (creating seepage through the embankment fill, and potentially causing erosion and loss of fine soil particles), a clay seal or concrete cut-off wall should be provided at the upstream end of the culvert replacement. If a clay seal is adopted, the clay material should meet the requirements of OPSS 1205 *(Material Specification for Clay Seal)*. The clay seal should have a thickness of 1 m, and the seal should extend from a depth of 1 m below the scour level to a minimum horizontal distance of 2 m on either side of the culvert inlet opening, and a minimum vertical height equivalent to the high water level including treatment of the adjacent side slopes. Alternatively, a clay blanket may be constructed, extending upstream to a distance equal to three times the culvert height. Normally, a clay blanket would extend along the adjacent embankment side slopes to a height of two times the culvert height or the high water level, whichever is higher; however, at this site where the cover over the culvert is relatively thin, it is recommended that a clay blanket, if adopted, extend to the top of the embankment side slope.

If the creek flow velocities are sufficiently high, provision should be made for scour and erosion protection (suitable non-woven geotextiles and/or rip-rap) at the culvert inlet and outlet, including in front of any wing walls/retaining walls adjacent to the creek channel. The requirements for and design of erosion protection measures for the culvert inlet should be assessed by the hydraulic design engineer. As a minimum, rip-rap treatment for the culvert outlet should be consistent with the standard Treatment Type A presented in OPSD 810.010 *(Rip-Rap Treatment for Sewer and Culvert Outlets),* with the rip-rap placed up to the toe of slope level, in combination with the cut-off measures noted above. Similarly, rip-rap should be provided over the full extent of the clay blanket if adopted, including the creek side slopes and embankment fill slope adjacent to the culverts.

6.6 Lateral Earth Pressures for Design

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

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

- Select, free-draining granular fill meeting the specifications of OPSS 1010 Granular A or Granular B Type II but with less than 5 percent passing the 200 sieve should be used as backfill behind the walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub drains and frost taper should be in accordance with OPSD 3101.150 *(Abutment Walls, Backfill – Minimum Granular Requirements)* and OPSD 3121.150 *(Retaining Walls, Backfill – Minimum Granular Requirements*).

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the culvert walls, in accordance with *CHBDC* Section 6.9.3 and Figure 6.6. Compaction equipment should be used in accordance with MTO's Special Provision SP105S10 *(Amendment to OPSS 501).* Other surcharge loadings should be accounted for in the design as required.

- The granular fill may be placed either in a zone with the width equal to at least 1.5 m behind the back of the walls (see Case A in Figure C6.20(a) of the *Commentary* to the *CHBDC*), or within the wedge shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (see Case B in Figure C6.20(b) of the *Commentary* to the *CHBDC*).

- For Case A, the pressures are based on the existing embankment fill materials and the existing overburden soils and the following parameters (unfactored) may be used:

	Existing Fill
Soil unit weight:	20 kN/m^3
Coefficients of static lateral earth pressure:	
Active, K_a	0.33
At rest, K_o	0.50

- For Case B, where the pressures are based on OPSS 1010 Granular A or Granular B Type II fill behind the wall, the following parameters (unfactored) may be assumed:

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

Where the culvert wall support does not allow lateral yielding, at-rest earth pressures should be assumed for the geotechnical design. Where associated wing wall/retaining wall support allows lateral yielding of the stem, active earth pressures should be used in the geotechnical design of the structure. The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.9.1 and Table C6.6 of the *Commentary* to the *CHBDC*.

6.6.1 Seismic Considerations

Seismic (earthquake) loading should be assessed in the design in accordance with Section 4.6.4 of *CHBDC,* as significant seismic loading would result in increased lateral earth pressures acting on the culvert walls, wing walls and/or retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the applicable earthquake-induced dynamic earth pressure. The earthquake-induced dynamic pressure distribution is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

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

where

- K is either the static active earth pressure coefficient (K_a) or the static at-rest earth pressure coefficient (K_o);
- K_{AE} is the seismic active earth pressure coefficient;
- γ' is the effective unit weight of the soil (kN/m^3)
  - taken as soil unit weights given above for fill materials
  - taken as 20 kN/m^3 for the native materials
- d is the depth below the top of the wall (m); and
- H is the height of the wall above the toe (m).

According to Table C4.2 of the *Commentary* to the *CHBDC*, this site is located in Seismic Zone 2, and the site-specific zonal acceleration ratio for the Kingston area is 0.1. For the thickness and type of overburden soils at this site, a site coefficient of 1.0 and an amplification factor of 1.33 are recommended. Therefore, the recommended ground surface acceleration is 0.133g.

The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of A = 0.133. These coefficients have been determined in accordance with Sections 4.6.4 and C4.6.4 of the *CHBDC* and its *Commentary,* include the effect of wall friction, and assume that the back of the wall is vertical and the ground surface behind the wall is essentially flat. For the low zonal acceleration ratio for this site, these seismic K_{AE} values are less than or similar to the static values of K_a and K_o reported above.

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case A	Case B	
	Earth Fill	**Granular ‘A’**	**Granular ‘B’ Type II**
Yielding Wall	0.32	0.29	0.29
Non-Yielding Wall	0.44	0.40	0.40

The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A (mm), where A is the design zonal acceleration ratio of 0.133. This corresponds to displacements of up to approximately 35 mm at this site.

6.7 Settlement and Culvert-Wing Wall Connection Requirements

It is understood that the Division Street grade will be lowered by approximately 250 mm over the replacement culvert (i.e., no increase in loading along the existing culvert alignment). However, some settlement of the foundation soils could occur near the ends of the replacement culvert if additional fill is placed or the geometry of the existing embankment side slope is modified as part of the Division Street interchange improvements.

Assuming placement of a total thickness of up to 1.5 m of fill behind the new wing walls/retaining walls at the east end of the replacement culvert, it is predicted that less than 20 mm of settlement will occur. All of this settlement will be completed within approximately six months following the placement of any additional fill.

It is recommended that the structural designer determine, based on this predicted magnitude of settlement and the actual change in embankment geometry and loading, whether a rigid connection or an articulated joint is required between the replacement culvert and the associated wing walls/retaining walls. In this case, it is understood that a shear key connection joint is preferred over a rigid connection.

The settlement analyses were carried out with the commercially-available program Settle-3D from Rocscience, using the consolidation parameters and estimated elastic deformation moduli given in the table below. These parameters have been assessed based on the oedometer test results, as well as correlations with the undrained shear strength, Atterberg limits and SPT "N" values, together with engineering judgement from experience with similar soils in this region of Ontario.

Soil Deposit	Bulk Unit Weight	Elastic Modulus	P_c'	e_o	C_c	C_r
Embankment fill	21 kN/m^3	–	–	–	–	–
Very stiff to hard silty clay to clay (weathered crust)	19 kN/m^3	20 MPa	–	–	–	–
Firm to stiff silty clay to clay (unweathered)	19 kN/m^3	–-	200 kPa	1.5	0.65	0.06

6.8 Construction Considerations

6.8.1 Groundwater and Surface Water Control for Foundation Excavation

Control of the surface water and groundwater will be necessary for the construction of the culvert replacement and associated wing walls/retaining walls, to allow excavation and foundation construction to be carried out in dry conditions.

Depending on the creek flow at the time of construction, the surface water flow could be passed through the culvert area by means of a temporary pipe, or diverted by pumping from behind a temporary cofferdam. If a sheetpile cofferdam is installed, it is recommended that the installation and removal of the sheetpile elements be completed with non-vibratory methods, to avoid "remoulding" (weakening) the sensitive clayey soils and potentially impacting the performance of the culvert and wing wall foundations. An Operational Constraint or Non-Standard Special Provision (NSSP) is recommended to address this requirement, or alternatively OPSS 539 *(Construction Specification for Temporary Protection Systems)* could be modified to include this requirement.

Surface water should be directed away from the excavation areas, to prevent ponding of water that could result in disturbance and weakening of the sensitive silty clay to clay subgrade soils; further discussion on this aspect is provided in Section 6.8.3.

As discussed in Sections 6.3 and 6.4, the culvert foundation excavations will extend below the groundwater level at this site, and groundwater control will be required to handle seepage from the granular fill and from seams or lenses in the silty clay to clay deposit. It is anticipated that the groundwater seepage will be managed by pumping from properly installed sumps within the excavations. An NSSP is not considered necessary to address the control of groundwater and surface water at this site; rather, the required dewatering is considered to be covered by OPSS 902 *(Construction Specification for Concrete Structures),* under Section 902.07.04 regarding dewatering of structure excavations.

6.8.2 Excavation and Temporary Roadway Protection

Temporary excavations for the culvert replacement will be made through the existing fill and are expected to terminate in the very stiff to hard weathered crust portion of the silty clay to clay deposit. Excavation works must be carried out in accordance with the guidelines outlined in the Occupational Health and Safety Act and Regulations for Construction Projects. The existing fill would be classified as Type 3 soil, according to the OHSA. Where space permits, temporary open-cut excavations through these materials should be made with side slopes formed no steeper than 1H:1V, assuming proper groundwater and surface water control is in place.

It is expected that a temporary protection system will be required along the centreline of Division Street to facilitate the culvert replacement in two halves. The temporary excavation support systems should be designed and constructed in accordance with OPSS 539 *(Construction Specification for Temporary Protection Systems).* The lateral movement of the temporary shoring system should meet Performance Level 2 as specified in OPSS 539, provided that any adjacent utilities can tolerate this magnitude of deformation.

The protection system will be required for a maximum excavation depth of approximately 3 m. It is considered that either a driven, interlocking sheetpile system or a soldier pile and timber lagging system would be suitable for the roadway protection at this site, based on the subsurface soil and groundwater conditions. An interlocking sheetpile system would contribute to both ground and groundwater control – it would allow control of seepage of groundwater perched in the granular fill. For the soldier pile and lagging system, it will be necessary to control seepage from the granular fill or include measures to mitigate loss of soil particles through the lagging boards.

The sheetpiles or soldier piles would have to be socketted to sufficient depth to provide the necessary passive resistance for the retained soil height of up to about 3 m; lateral support to the sheetpiles or soldier piles could be provided in the form of rakers or temporary anchors.

The selection and design of the protection system will be the responsibility of the Contractor.

6.8.3 Subgrade Protection

The sensitive silty clay to clay soils exposed at the footing subgrade level will be susceptible to disturbance from construction traffic and/or ponded water. To limit this degradation, it is recommended that a concrete working slab be placed on the subgrade within four hours after preparation, inspection and approval of the footing subgrade. This requirement can be addressed with a note on the General Arrangement drawing and/or with an NSSP. A sample NSSP is included in Appendix C.

7.0 CLOSURE

This Foundation Design Report was prepared by Ms. Nikol Kochmanová, EIT, and reviewed by Ms. Lisa Coyne, P.Eng., a geotechnical engineer and Principal with Golder. Mr. Fin Heffernan, P.Eng., a Designated MTO Contact for Golder, conducted an independent review of this report.

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NK/LCC/FJH/nk/lcc

n:\active\2005\1111\05-1111-031 mrc hwy 401 kingston\phase 7000 division street culvert\5 – report\05-1111-031 drt06 10apr division street culvert replacement.docx

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Ontario Provincial Standard Specifications (OPSS)

OPSS 422	Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut
OPSS 539	Construction Specification for Temporary Protection Systems
OPSS 902	Construction Specification for Concrete Structures
OPSS 1002	Material Specification for Aggregates - Concrete
OPSS 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material
OPSS 1205	Material Specification for Clay Seal

Ontario Provincial Standard Drawings (OPSD)

OPSD 803.010	Backfill and Cover for Concrete Culverts
OPSD 810.010	Rip-Rap Treatment for Sewer and Culvert Outlets
OPSD 3090.101	Foundation Frost Depths for Southern Ontario
OPSD 3101.150	Abutment Walls, Backfill – Minimum Granular Requirements
OPSD 3121.150	Retaining Walls, Backfill – Minimum Granular Requirements

Special Provisions (SP)

SP105S10	Amendment to OPSS 501

TABLE 1
COMPARISON OF FOUNDATION ALTERNATIVES
CULVERT REPLACEMENT AT STATION 9+950 DIVISION STREET
HIGHWAY 401 WIDENING, KINGSTON, ONTARIO
W.P. 77-99-01

Option	Advantages	Disadvantages	Risks/Consequences
Box culvert replacement	• Minimizes depth of excavation, excavation support and dewatering requirements compared to open footing option • Pre-cast box sections may allow faster construction than cast-in-place open footings, with shorter time duration for dewatering and surface water pumping	• Excavation would extend to a depth of about 1.6 m below groundwater level, and dewatering (control and collection of seepage) would be required	• Some risk of disturbance of the sensitive silty clay to clay subgrade; however, excavation will be maintained in very stiff to hard portion of the silty clay to clay deposit
Open footing culvert replacement	• Would satisfy any fisheries requirements related to natural channel substrate, if applicable • May be feasible to build culvert replacement on pre-cast footing sections, to accelerate construction schedule and reduce time for dewatering and surface water pumping	• Excavation would extend to a depth of about 2.2 m below groundwater level, and dewatering (control and collection of seepage) would be required • Cast-in-place footings may require a longer duration for construction, including dewatering and surface water pumping, as compared with pre-cast footing elements	• Some risk of disturbance of the sensitive silty clay to clay subgrade; in addition, excavation will be slightly deeper than for box culvert replacement, and there is some risk that the footings will come close to or extend into the unweathered, firm to stiff portion of the silty clay to clay deposit

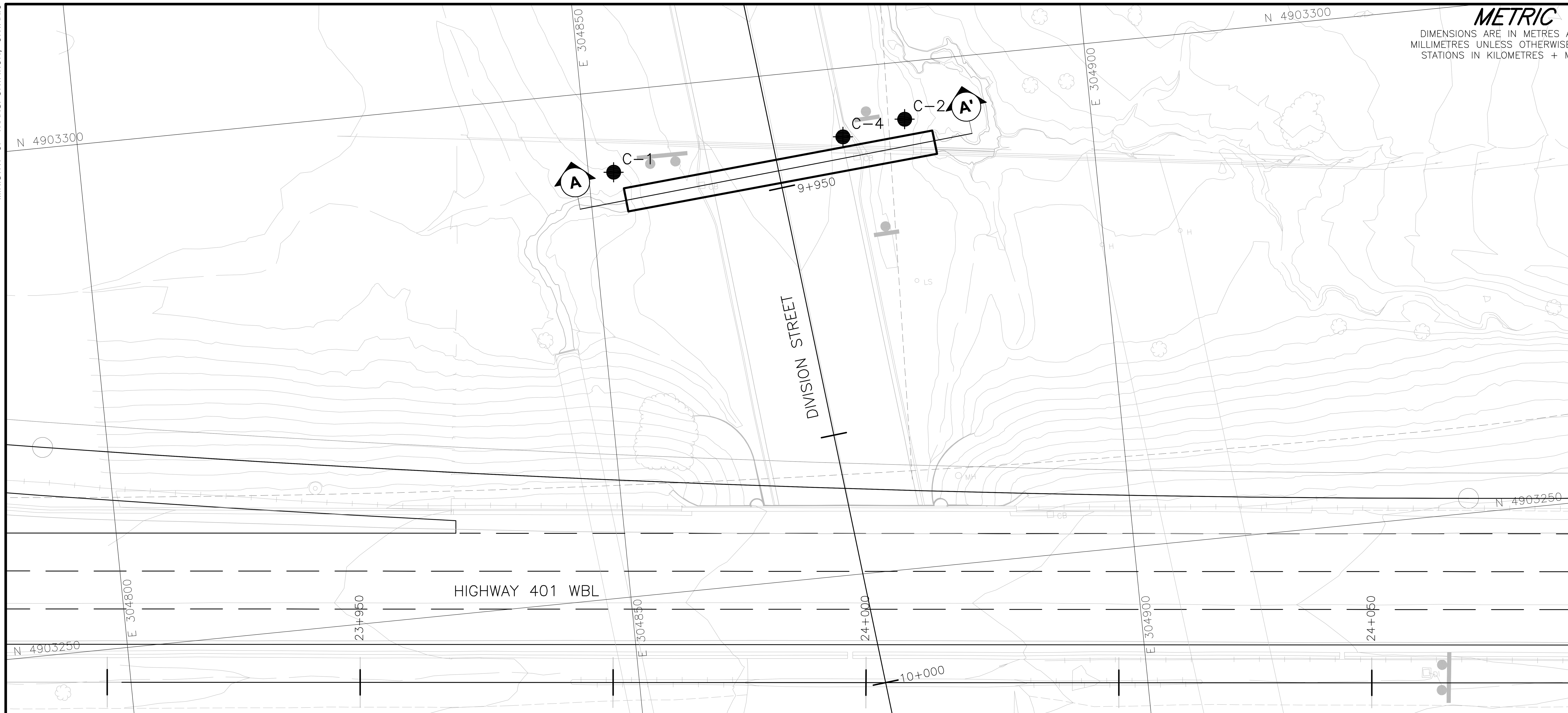
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No. 4508–02–00

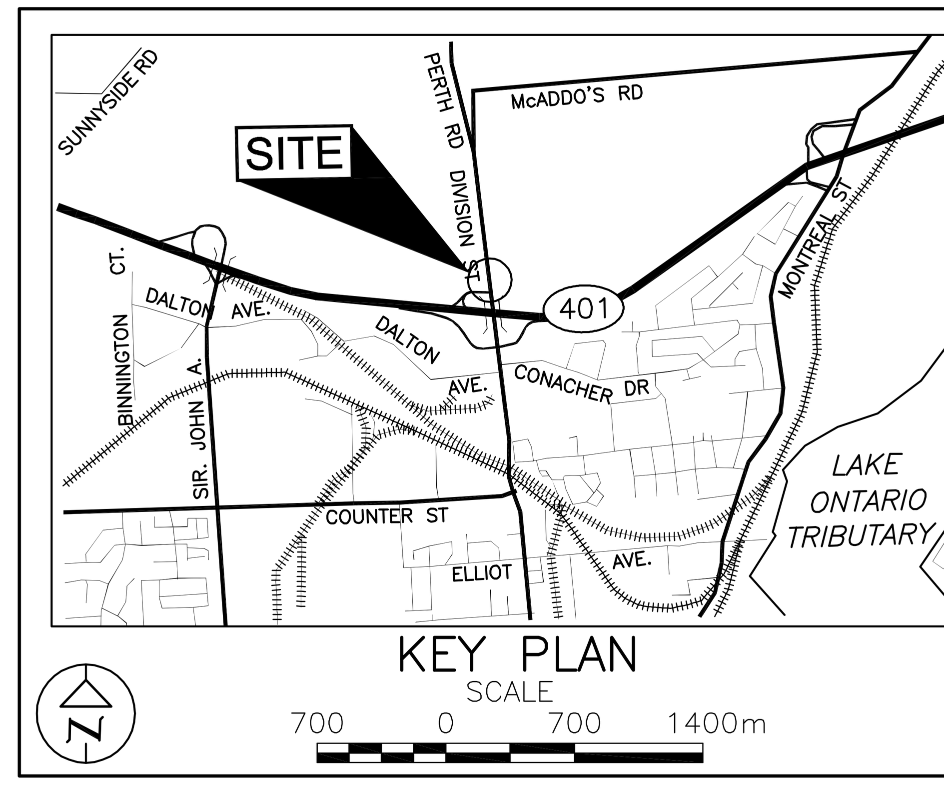
SHEET

HIGHWAY 401 WIDENING
DIVISION STREET CULVERT REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN
SCALE
5 0 5 10 m

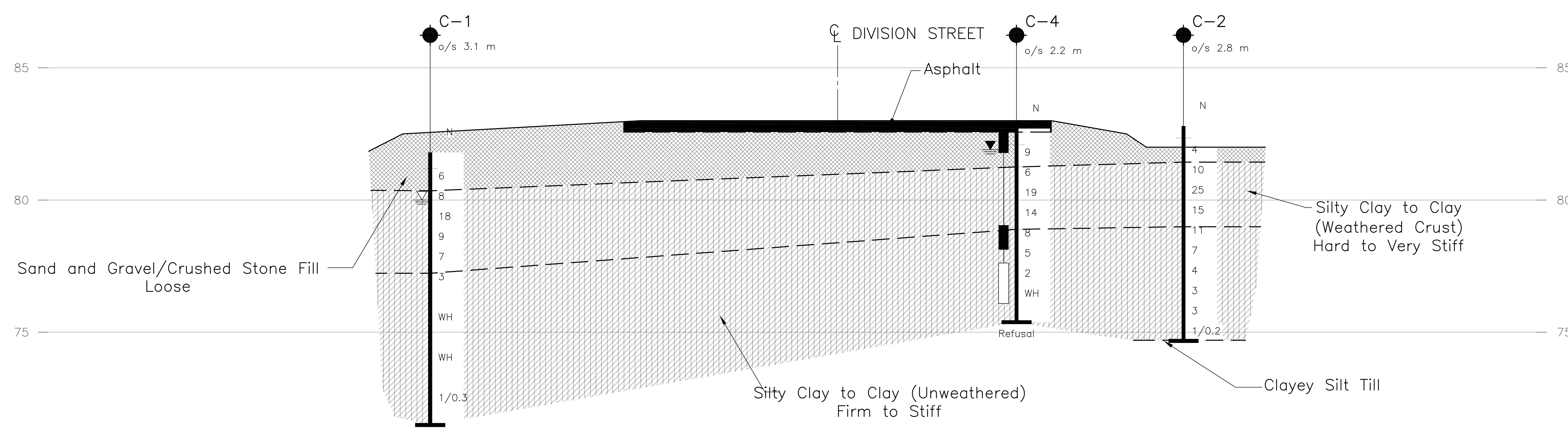


KEY PLAN
SCALE
700 0 700 1400m

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, measured on March 19, 2010
- WL upon completion of drilling

No.	ELEVATION	CO–ORDINATES NORTHING	CO–ORDINATES EASTING
C–1	81.8	4903292.2	304852.6
C–2	82.8	4903294.4	304881.1
C–4	82.7	4903293.3	304874.8



CROSS SECTION A–A'
SCALE
2 0 2 4 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 plan provided in digital format by McCormick Rankin Corporation, (Drawing File No. 07437–002.dwg, dated December 22, 2009).

NO.	DATE	BY	REVISION

Geocres No. 31C–197

HWY. 401		PROJECT NO. 05–1111–031–6	DIST.
SUBM'D. NK	CHKD. LCC	DATE: April 2010	SITE:
DRAWN: JFC	CHKD. NK	APPD. LCC	DWG. 1

APPENDIX A

Borehole Records

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.) split-spoon 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.

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

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

(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 $= \tan\delta$
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

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

Golder Associates

RECORD OF BOREHOLE No C-1

 METRIC

PROJECT 05-1111-031

G.W.P. 4508-02-00 LOCATION N 4903292.2 ;E 304852.6 ORIGINATED BY DG

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

DATUM Geodetic DATE February 25, 2010 CHECKED BY LCC

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	FIELD VANE (SENSITIVITY)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
81.8	GROUND SURFACE									
0.0	Crushed stone (FILL) Grey									
81.2 / 0.6	Sand to sand and gravel (FILL) Loose Brown Moist	1	SS	6						
80.4 / 1.5	SILTY CLAY to CLAY, trace sand, containing pebbles (Weathered Crust) Hard to very stiff Grey - brown Moist	2	SS	8						
		3	SS	18			0	2	36	62
		4	SS	9						
		5	SS	7						
77.2 / 4.6	SILTY CLAY to CLAY, trace sand Firm to stiff Dark grey Moist to wet	6	SS	3						
					4.3					
					4.5					
		7	SS	WH		19.6				
					5.1					
					5.8					
		8	SS	WH						
					7.1					
					6.8					
		9	SS	1/0.3						
					6.3					
71.5 / 10.3	END OF BOREHOLE RESISTANCE TO VANE PENETRATION									

NOTES:
1. Water level in open borehole at 1.8 m (Elev. 80.0 m) upon completion of drilling.

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (72–81); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 1000 2000 3000 4000 5000; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

Golder Associates

RECORD OF BOREHOLE No C-2

 METRIC

PROJECT	05-1111-031		
G.W.P.	4508-02-00	LOCATION	N 4903294.4 ;E 304881.1
DIST	Eastern HWY 401	BOREHOLE TYPE	C.M.E. 55, 108mm I.D. Hollow Stem Augers
DATUM	Geodetic	DATE	February 24, 2010
ORIGINATED BY	DG	COMPILED BY	NK
CHECKED BY	LCC		

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
82.8	GROUND SURFACE
0.0	Blast Rock (FILL)
82.3 / 0.5	Sand and gravel (FILL) Loose Brown
81.4 / 1.4	SILTY CLAY to CLAY, trace sand, containing pebbles (Weathered Crust) Hard to very stiff Grey - brown Moist
79.0 / 3.8	SILTY CLAY to CLAY, trace sand Stiff to very stiff Dark grey Moist to wet
74.7 / 8.1	CLAYEY SILT, with sand (TILL) END OF BOREHOLE REFUSAL TO SPLIT-SPOON AND AUGER ADVANCE

NOTES:
1. Borehole dry upon completion of drilling.

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
1	SS	4	45	48	6	1
2	SS	10				
3	SS	25				
4	SS	15				
5	SS	11				
6	SS	7	0	2	31	67
7	SS	4				
8	SS	3				
9	SS	3				
10	SS	1/0.2				

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE (82, 81, 80, 79, 78, 77, 76, 75); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED (1000, 2000, 3000, 4000, 5000); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (25, 50, 75); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

RECORD OF BOREHOLE No C-4

 METRIC

PROJECT 05-1111-031
G.W.P. 4508-02-00 LOCATION N 4903293.3 ;E 304874.8 ORIGINATED BY DG
DIST Eastern HWY 401 BOREHOLE TYPE C.M.E. 55, 108mm I.D. Hollow Stem Augers COMPILED BY NK
DATUM Geodetic DATE February 24, 2010 CHECKED BY LCC

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
82.7	GROUND SURFACE										
0.0	ASPHALT										
0.1	Crushed stone (FILL) Grey Wet	1	AS		82			50	38	9	3
82.1 / 0.6	Crushed stone (FILL) Loose Grey Wet	2	SS	9							
81.3 / 1.5	SILTY CLAY to CLAY, trace sand (Weathered Crust) Hard to very stiff Grey - brown Moist	3	SS	6	81						
		4	SS	19	80			0	2	46	52
		5	SS	14	79						
78.9 / 3.8	SILTY CLAY to CLAY, trace sand Stiff to very stiff Dark grey Moist to wet	6	SS	8				0	1	30	69
		7	SS	5	78						
		8	SS	2	77						
		9	SS	WH	76	5.6 (field vane)	19.6				
75.4 / 7.3	END OF BOREHOLE RESISTANCE TO VANE PENETRATION										

NOTES:
1. Borehole dry upon completion of drilling.

2. Water level in piezometer measured at a depth of 0.8 m (Elev. 81.9 m) on March 19, 2010.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 1000 2000 3000 4000 5000
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75

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

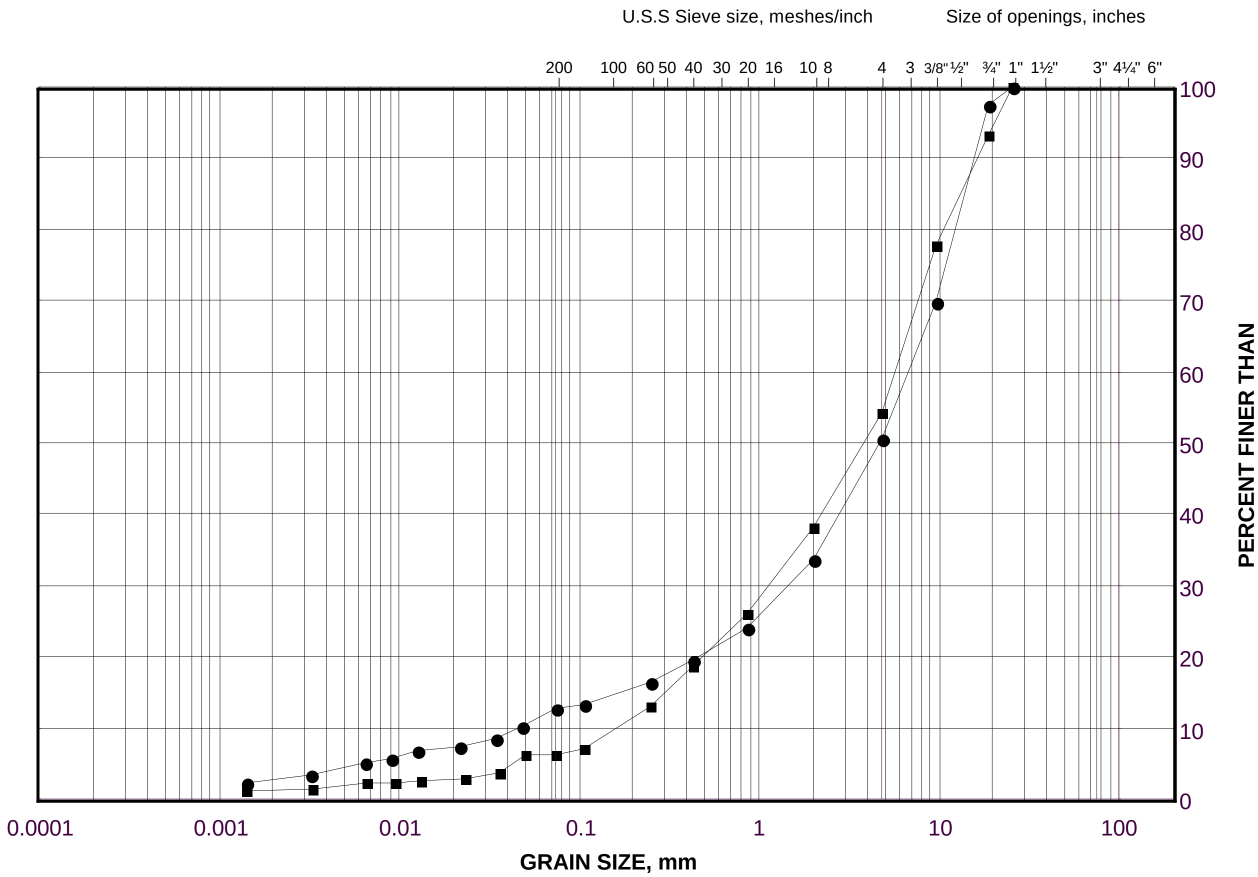
APPENDIX B

Laboratory Test Results

GRAIN SIZE DISTRIBUTION

Sand and Gravel / Crushed Stone Fill

FIGURE B1



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	C4	1	82.3
■	C2	1	81.7

Project Number: 05-1111-031

Checked By: ____________

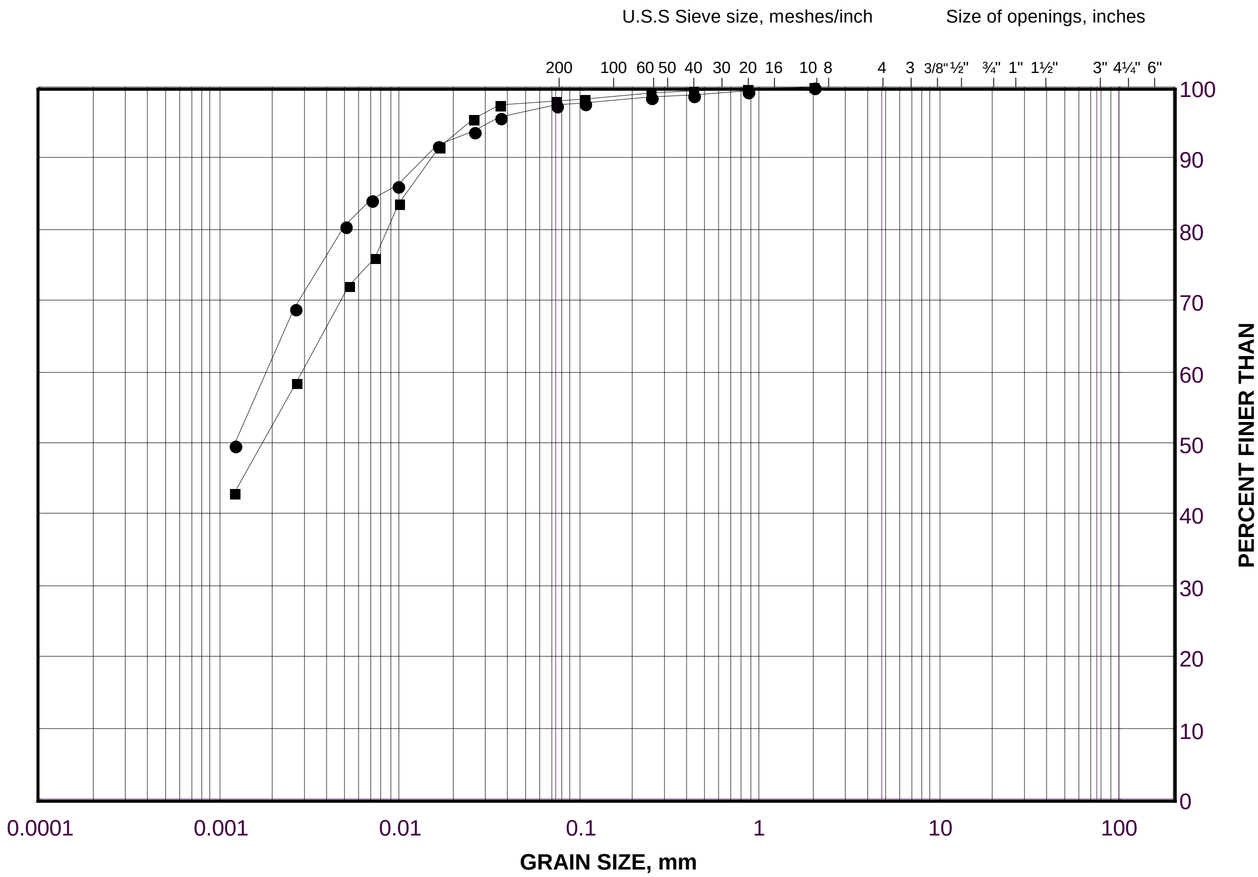
Golder Associates

Date: 22-Mar-10

GRAIN SIZE DISTRIBUTION

Silty Clay to Clay / Weathered Crust

FIGURE B2



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

LEGEND

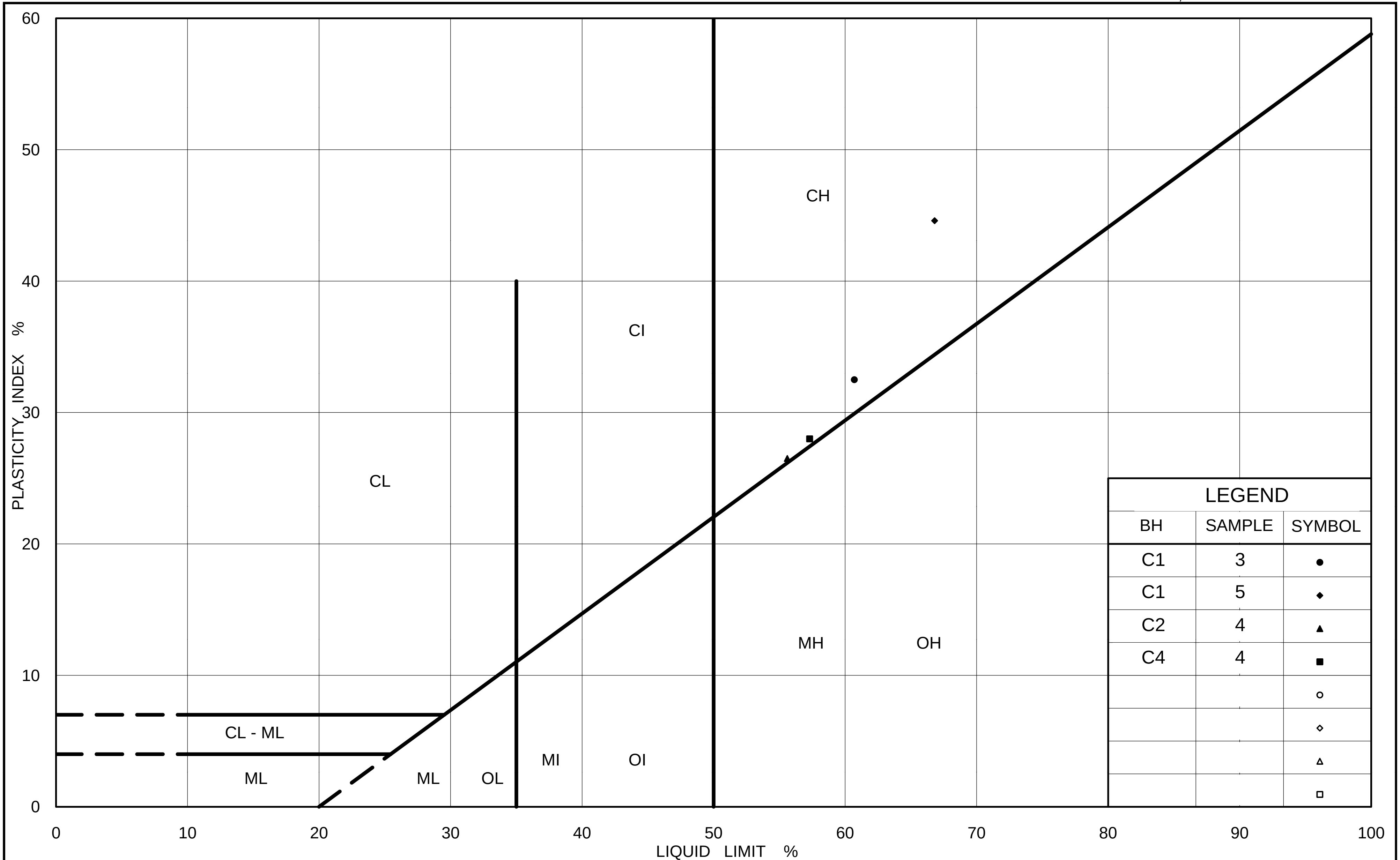
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	C1	3	79.2
■	C4	4	80.1

Project Number: 05-1111-031

Checked By: ______________________

Golder Associates

Date: 22-Mar-10

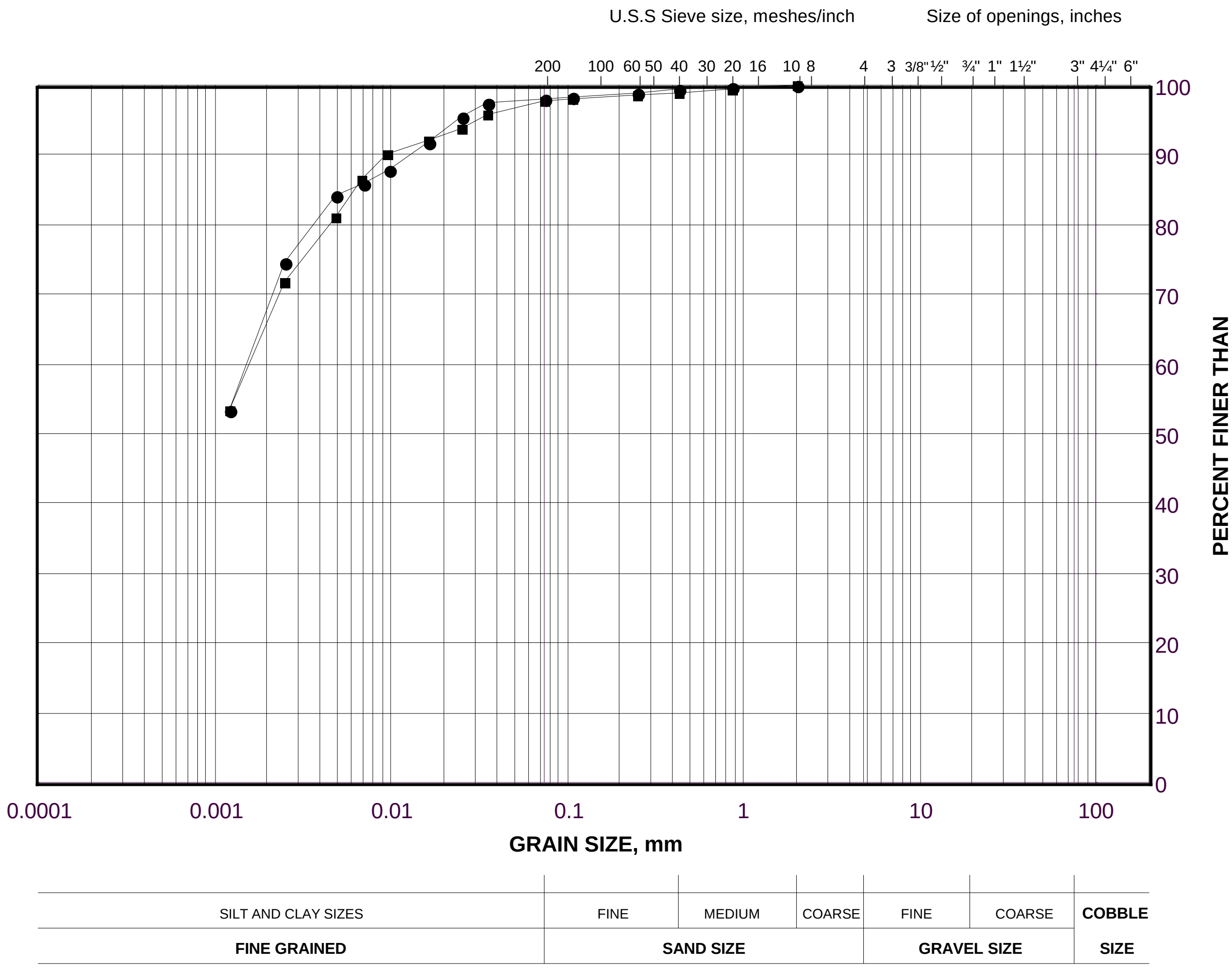


Ministry of Transportation Ontario	PLASTICITY CHART Silty Clay to Clay / Weathered Crust	Figure No. B3 Project No. 05-1111-031 Checked By:

GRAIN SIZE DISTRIBUTION

Silty Clay to Clay

FIGURE B4



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	C4	6	78.6
■	C2	6	77.9

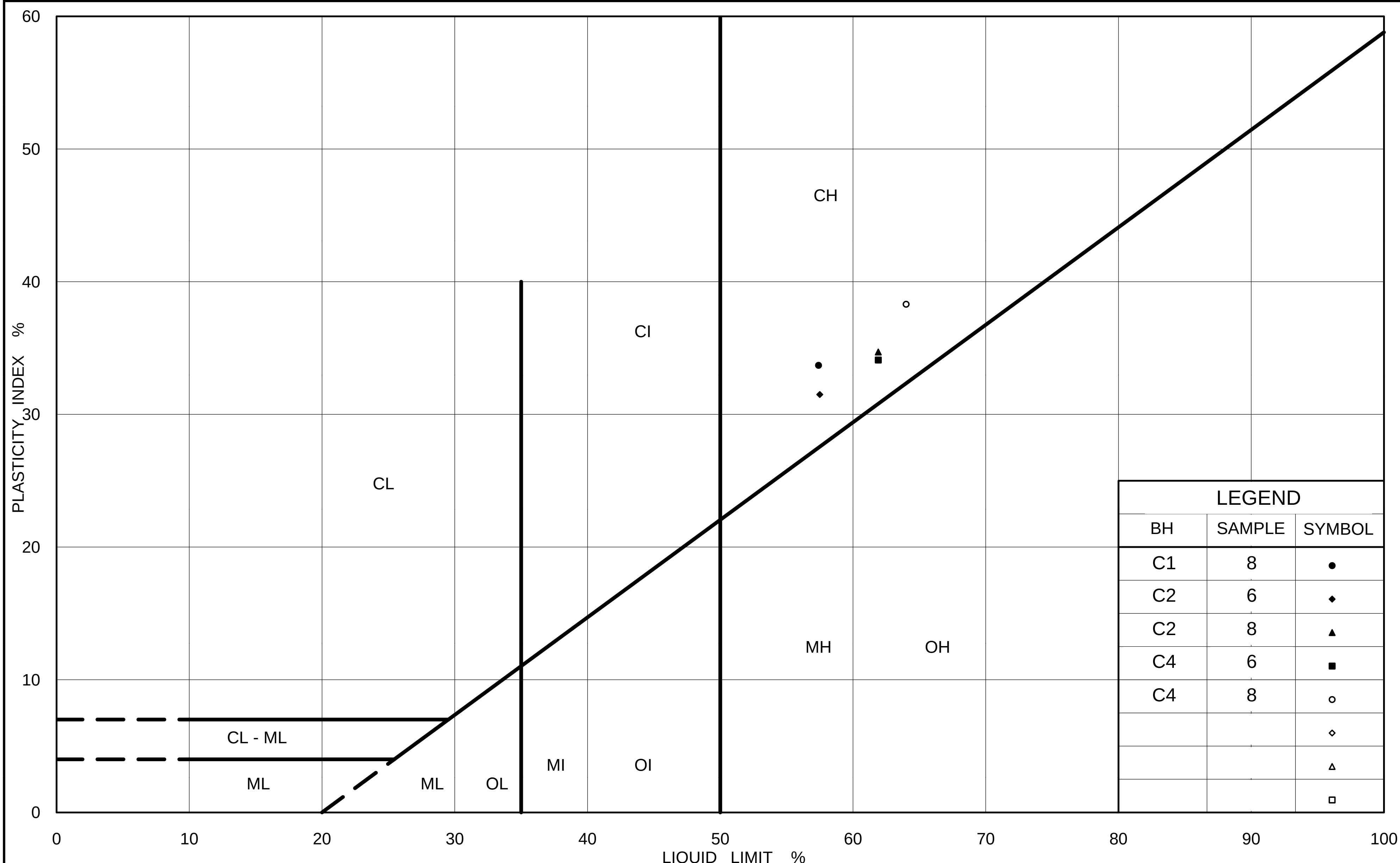
Project Number: 05-1111-031

Checked By: ______________________

Golder Associates

Date: 22-Mar-10

Oct 75, FF-S-21



BH	SAMPLE	SYMBOL
C1	8	●
C2	6	◆
C2	8	▲
C4	6	■
C4	8	○
		◇
		△
		□

Ministry of Transportation

Ontario

PLASTICITY CHART
Silty Clay to Clay

Figure No. B5

Project No. 05-1111-031

Checked By:

CONSOLIDATION TEST SUMMARY

Silty Clay to Clay (Borehole C-1)

SAMPLE IDENTIFICATION

Project Number	05-1111-031	Sample Number	7
Borehole Number	C-1	Sample Depth, m	6.1-6.6

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	12		
Date Started	3/3/2010		
Date Completed	3/24/2010		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.55	Unit Weight, kN/m^3	18.11
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m^3	12.97
Area, cm^2	31.47	Specific Gravity, measured	2.79
Volume, cm^3	80.25	Solids Height, cm	1.209
Water Content, %	39.68	Volume of Solids, cm^3	38.04
Wet Mass, g	148.23	Volume of Voids, cm^3	42.21
Dry Mass, g	106.12	Degree of Saturation, %	99.8

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	2.550	1.110	2.550				
4.85	2.550	1.110	2.550	4	3.45E-01	8.09E-06	2.73E-07
9.56	2.548	1.108	2.549	8	1.72E-01	1.91E-04	3.23E-06
19.37	2.547	1.107	2.547	83	1.66E-02	4.40E-05	7.14E-08
38.93	2.541	1.102	2.544	86	1.59E-02	1.14E-04	1.79E-07
77.92	2.537	1.099	2.539	227	6.02E-03	3.52E-05	2.08E-08
155.63	2.527	1.090	2.532	179	7.59E-03	5.45E-05	4.06E-08
250.79	2.497	1.066	2.512	349	3.83E-03	1.21E-04	4.55E-08
77.92	2.506	1.073	2.501				
19.20	2.518	1.084	2.512				
77.92	2.513	1.079	2.516	240	5.59E-03	3.27E-05	1.79E-08
155.62	2.509	1.075	2.511	244	5.48E-03	2.42E-05	1.30E-08
250.84	2.492	1.062	2.500	228	5.81E-03	6.75E-05	3.85E-08
313.09	2.474	1.047	2.483	390	3.35E-03	1.15E-04	3.79E-08
624.21	2.362	0.955	2.418	747	1.66E-03	1.41E-04	2.29E-08
1244.66	2.174	0.798	2.268	670	1.63E-03	1.19E-04	1.90E-08
2489.94	1.999	0.654	2.086	332	2.78E-03	5.50E-05	1.50E-08
624.21	2.036	0.685	2.018				
250.84	2.072	0.714	2.054				
77.92	2.124	0.757	2.098				
19.37	2.164	0.791	2.144				
4.85	2.188	0.810	2.176				

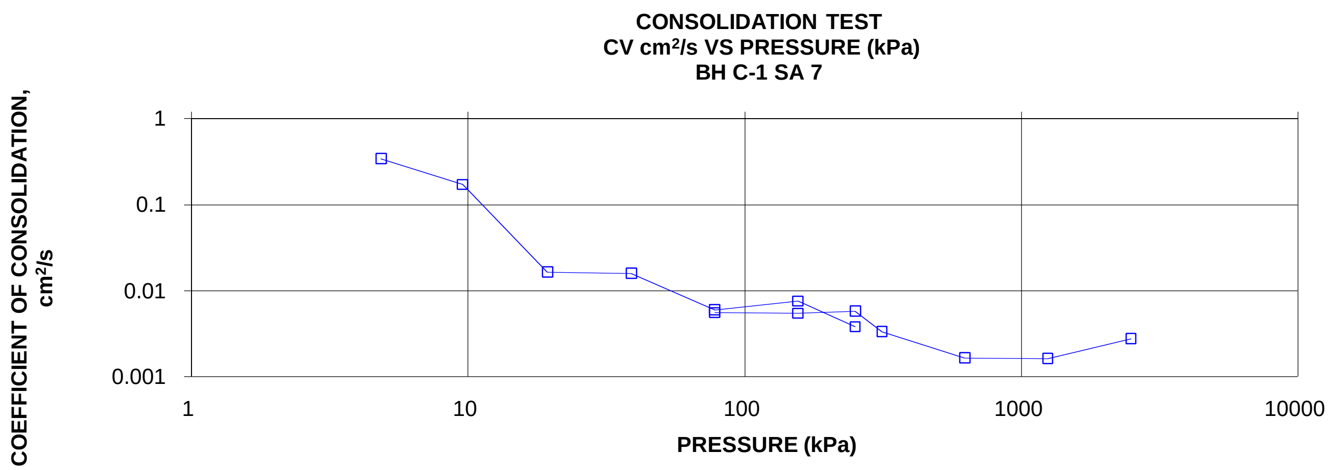
Note:
k calculated using cv based on t_{90} values.

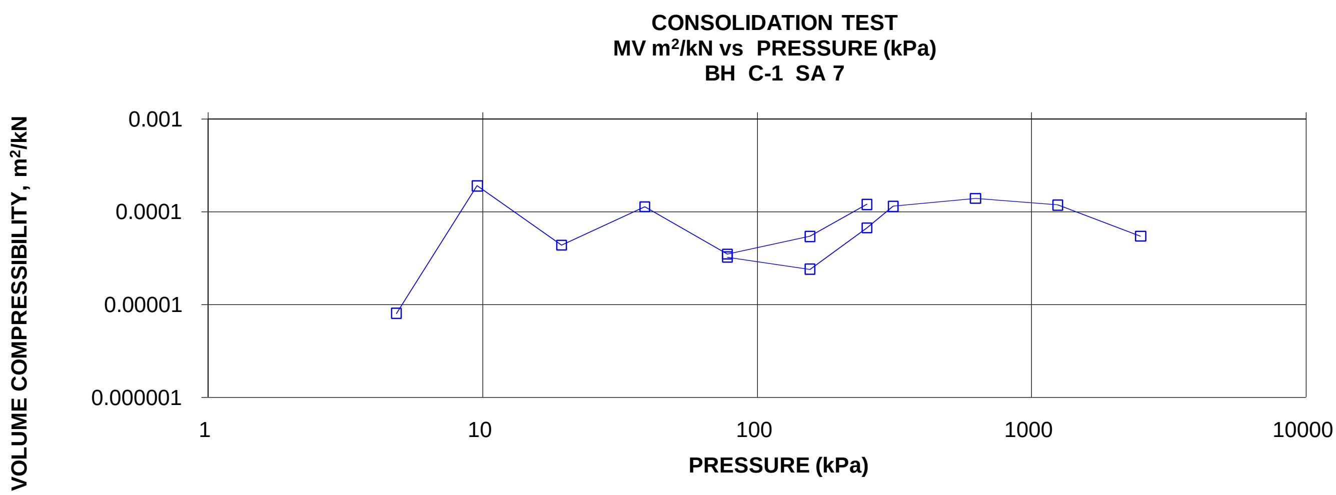
SAMPLE DIMENSIONS AND PROPERTIES - FINAL

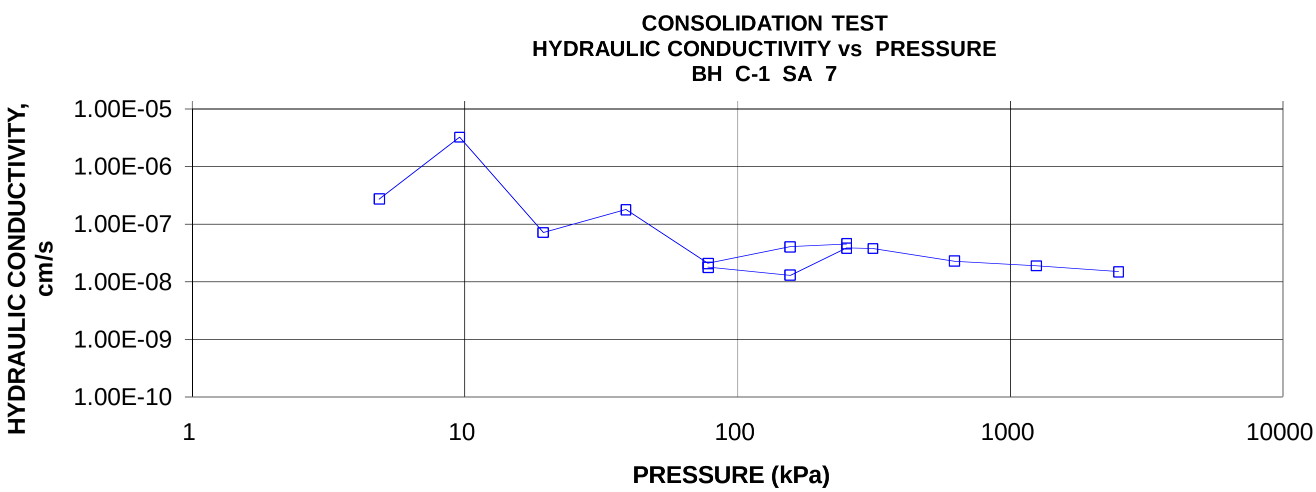
Sample Height, cm	2.19	Unit Weight, kN/m^3	19.64
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m^3	15.12
Area, cm^2	31.47	Specific Gravity, measured	2.79
Volume, cm^3	68.85	Solids Height, cm	1.209
Water Content, %	29.91	Volume of Solids, cm^3	38.04
Wet Mass, g	137.86	Volume of Voids, cm^3	30.81
Dry Mass, g	106.12		

CONSOLIDATION TEST SUMMARY

Silty Clay to Clay (Borehole C-1)

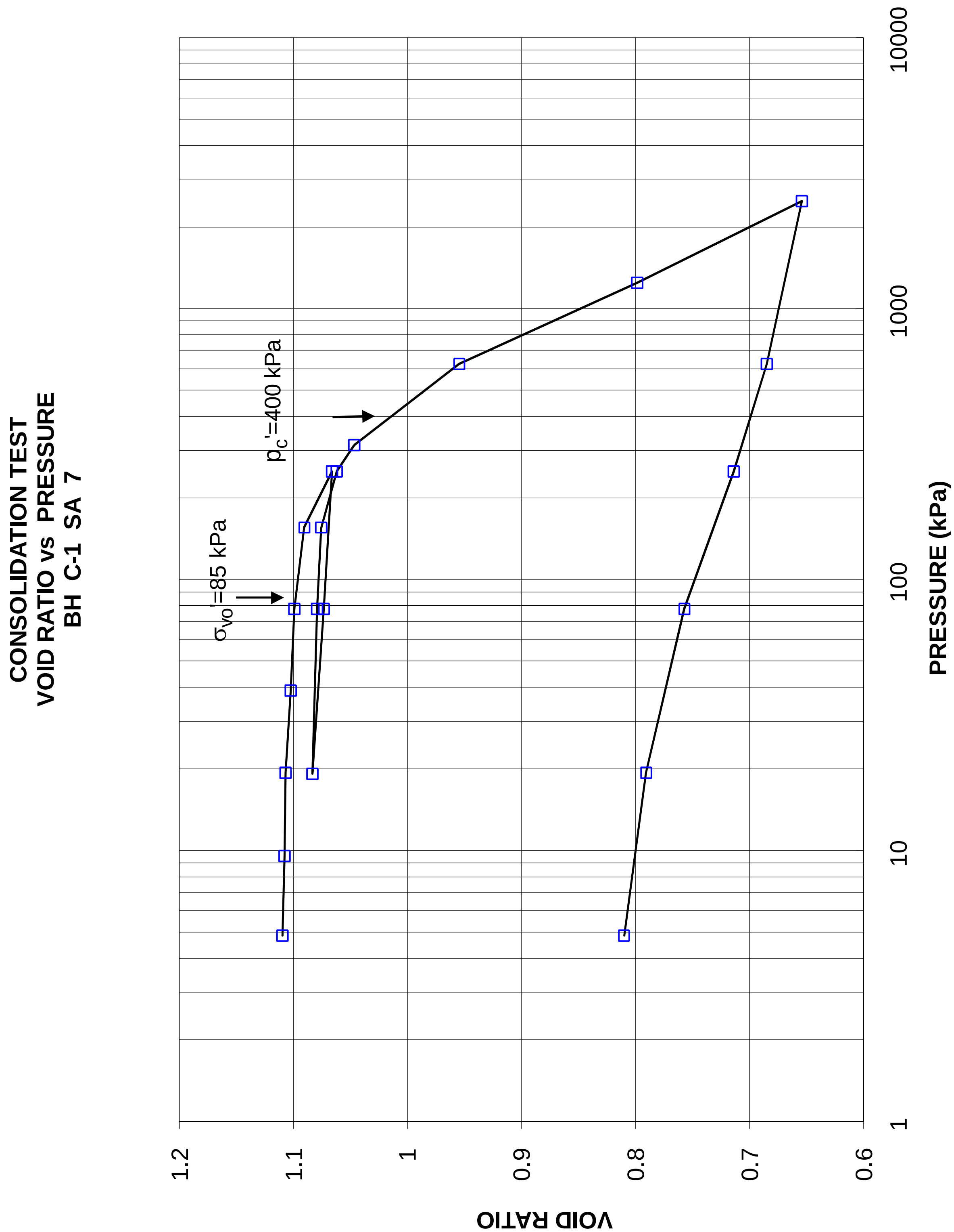






CONSOLIDATION TEST
VOID RATIO VS LOG PRESSURE
Silty Clay to Clay (Borehole C-1)

FIGURE B6
Page 3 of 4



Project No. 05-1111-031

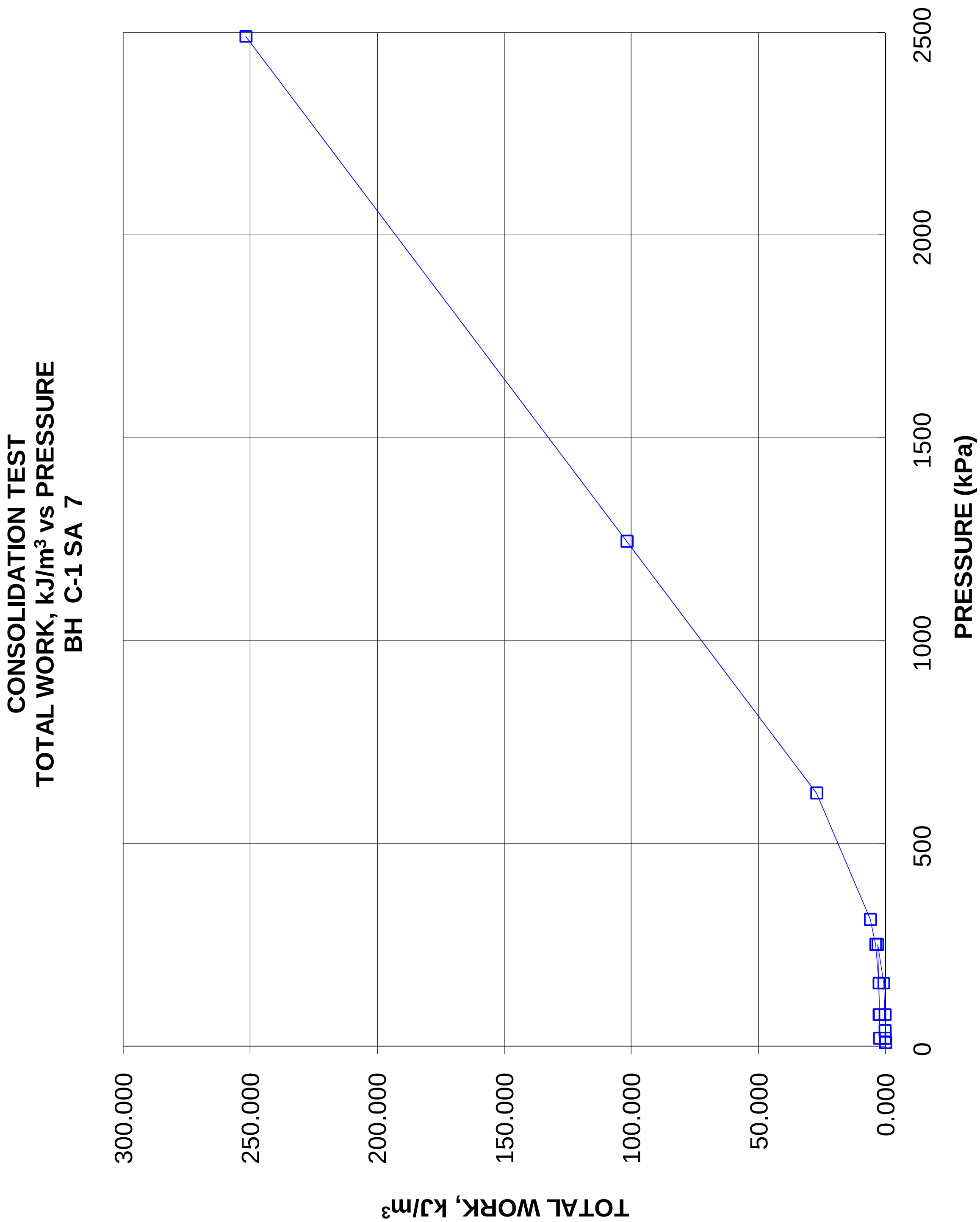
Prepared By: LH

Golder Associates

Checked By: MM

CONSOLIDATION TEST
TOTAL WORK VS PRESSURE
Clayey SIlt to Clay (Borehole C-1)

FIGURE B6
Page 4 of 4



Project No. 05-1111-031

Golder Associates

Prepared By: LH

Checked By: MM

CONSOLIDATION TEST SUMMARY

Silty Clay to Clay (Borehole C-4)

SAMPLE IDENTIFICATION

Project Number	05-1111-031	Sample Number	9
Borehole Number	C-4	Sample Depth, m	6.1-6.6

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	5		
Date Started	3/3/2010		
Date Completed	3/24/2010		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m^3	16.50
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	10.39
Area, cm^2	31.52	Specific Gravity, measured	2.79
Volume, cm^3	59.89	Solids Height, cm	0.722
Water Content, %	58.75	Volume of Solids, cm^3	22.75
Wet Mass, g	100.74	Volume of Voids, cm^3	37.14
Dry Mass, g	63.46	Degree of Saturation, %	100.4

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	1.900	1.633	1.900				
4.78	1.900	1.633	1.900	7	1.09E-01	1.10E-05	1.18E-07
9.59	1.895	1.626	1.897	97	7.87E-03	5.47E-04	4.22E-07
19.10	1.892	1.622	1.893	97	7.83E-03	1.72E-04	1.32E-07
39.08	1.880	1.605	1.886	179	4.21E-03	3.16E-04	1.30E-07
77.79	1.859	1.576	1.869	231	3.21E-03	2.87E-04	9.02E-08
155.56	1.813	1.512	1.836	187	3.82E-03	3.11E-04	1.16E-07
250.08	1.686	1.337	1.749	832	7.80E-04	7.06E-04	5.39E-08
77.79	1.702	1.359	1.694				
18.05	1.731	1.399	1.717				
77.77	1.716	1.378	1.724	217	2.90E-03	1.38E-04	3.94E-08
155.08	1.697	1.351	1.706	240	2.57E-03	1.29E-04	3.24E-08
250.08	1.665	1.307	1.681	148	4.05E-03	1.76E-04	6.97E-08
314.39	1.601	1.218	1.633	1940	2.91E-04	5.27E-04	1.50E-08
624.15	1.432	0.984	1.516	1940	2.51E-04	2.87E-04	7.07E-09
1245.55	1.308	0.812	1.370	519	7.66E-04	1.05E-04	7.89E-09
2487.45	1.209	0.675	1.258	83	4.04E-03	4.20E-05	1.66E-08
624.15	1.236	0.712	1.222				
250.08	1.255	0.739	1.245				
77.77	1.289	0.787	1.272				
19.10	1.333	0.848	1.311				
4.78	1.350	0.871	1.342				

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

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.35	Unit Weight, kN/m^3	19.62
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	14.63
Area, cm^2	31.52	Specific Gravity, measured	2.79
Volume, cm^3	42.55	Solids Height, cm	0.722
Water Content, %	34.15	Volume of Solids, cm^3	22.75
Wet Mass, g	85.13	Volume of Voids, cm^3	19.80
Dry Mass, g	63.46		

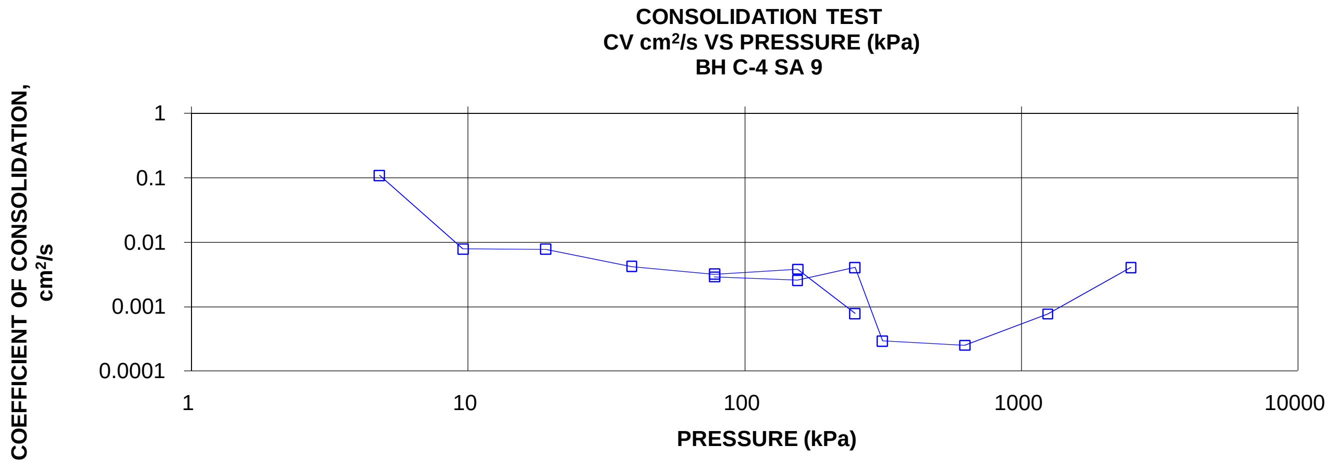
Prepared By: LH

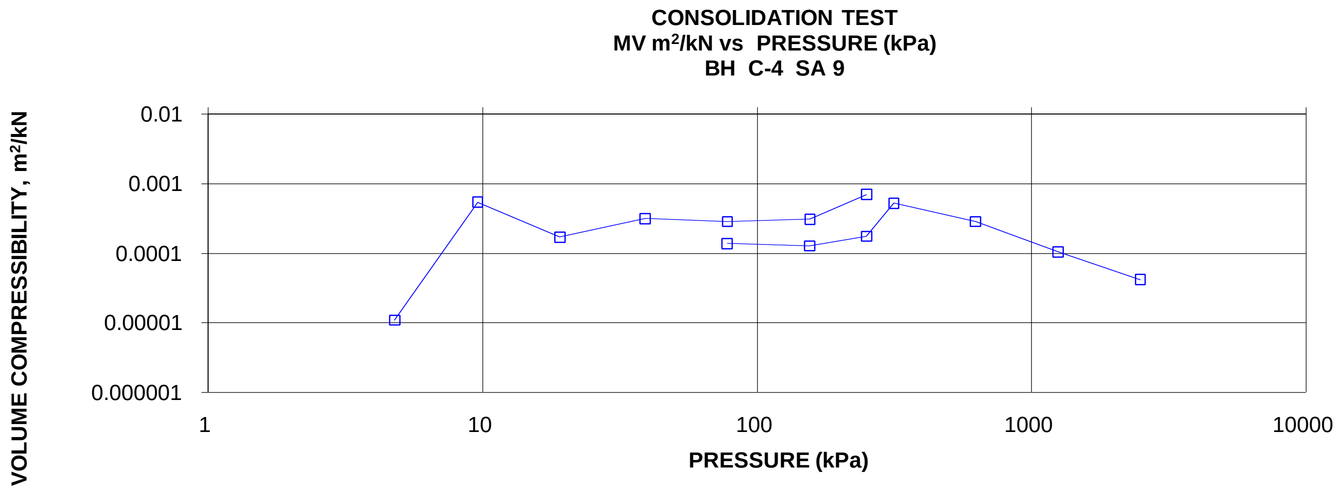
Golder Associates

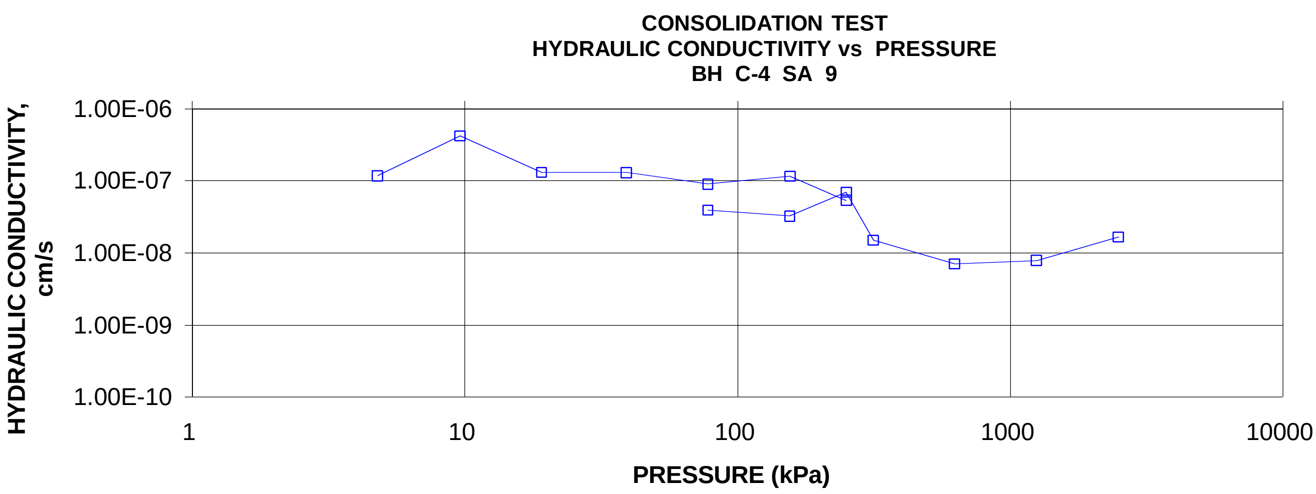
Checked By: MM

CONSOLIDATION TEST SUMMARY
Silty Clay to Clay (Borehole C-4)

FIGURE B7







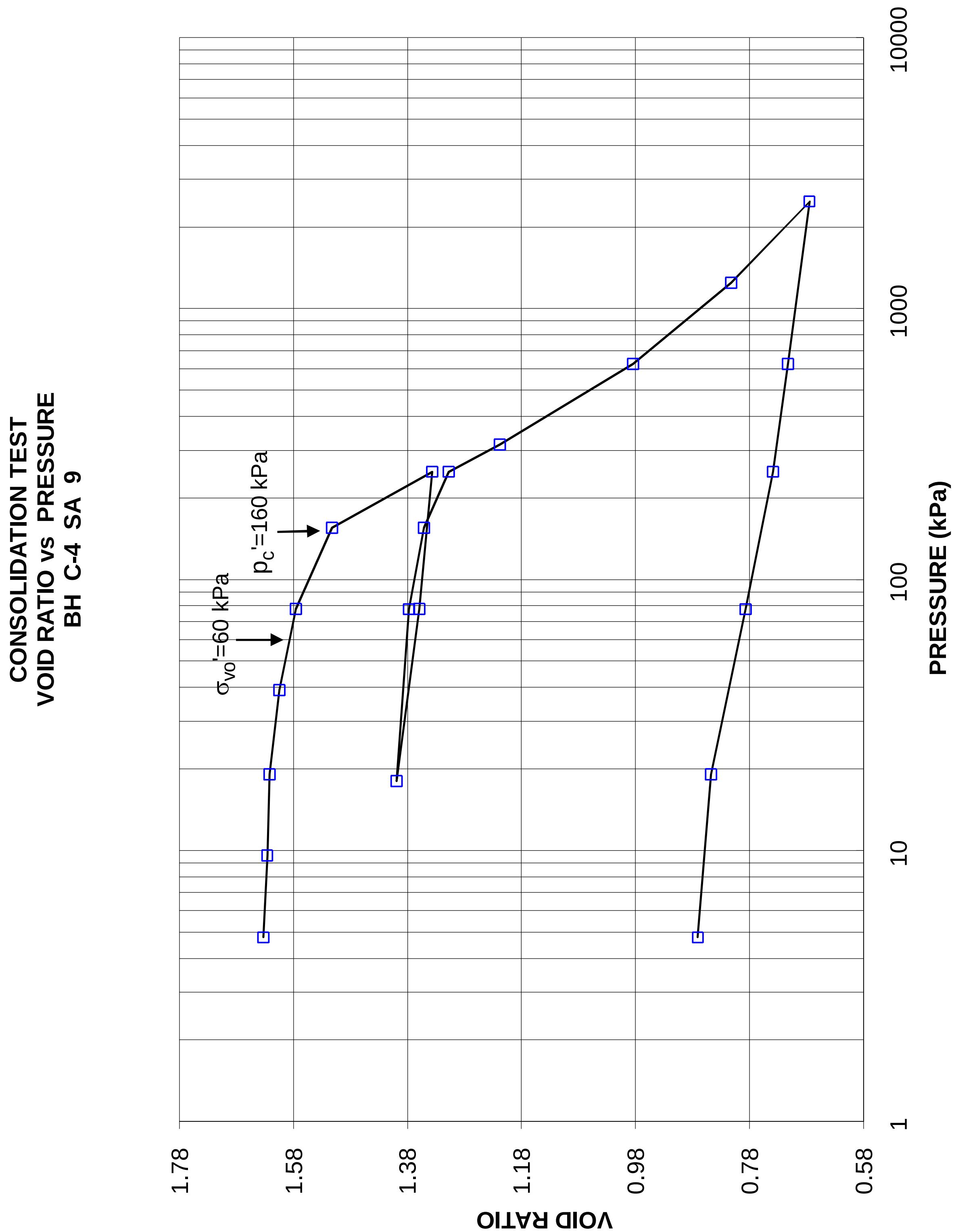
Project No. 05-1111-031
Prepared By: LH

Golder Associates

Checked By: MM

CONSOLIDATION TEST
VOID RATIO VS LOG PRESSURE
Silty Clay to Clay (Borehole C-4)

FIGURE B7
Page 3 of 4



Project No. 05-1111-031

Prepared By: LH

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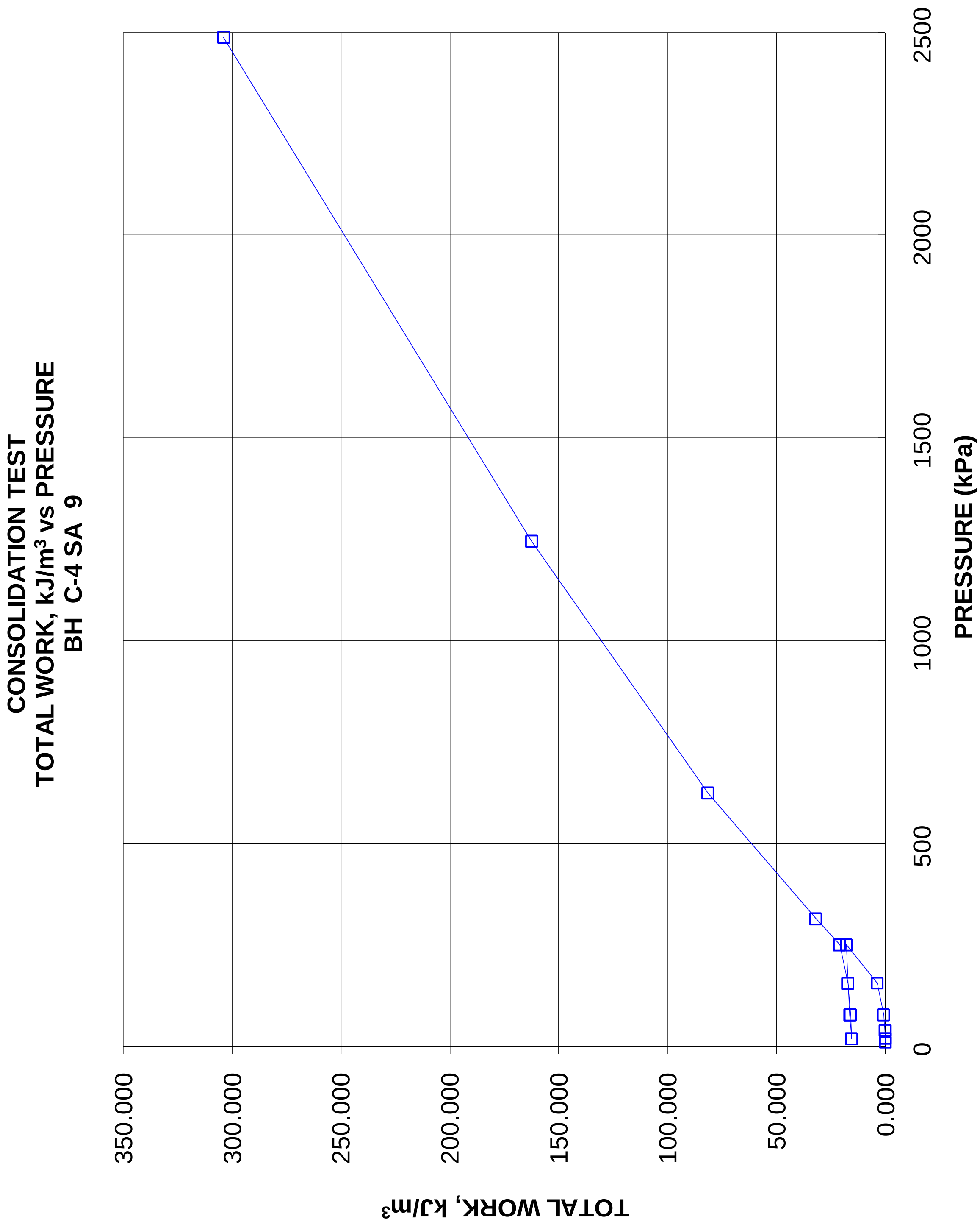
CONSOLIDATION TEST
TOTAL WORK VS PRESSURE
Clayey Silt to Clay (Borehole C-4)



Project No. 05-1111-031

Prepared By: LH

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Checked By: MM

APPENDIX C

Non-Standard Special Provisions

WORKING SLAB, Item No.

Non-Standard Special Provision

SCOPE

This Special Provision covers the requirements for the supply and placement of a concrete working slab under the structure foundations. The purpose of the working slab is to protect the subgrade from disturbance and loosening due to construction traffic and ponded water and also to provide a level working surface.

CONSTRUCTION

Within four hours following inspection and approval of the prepared subgrade, a concrete working slab with a minimum thickness of 100 mm shall be placed on the foundation subgrade as per the contract drawings and documents. The concrete shall have a minimum 28 day compressive strength of 20 MPa.

Unwatering of the excavation for the footing construction, including the construction of the working slab, might be required and is covered under separate Tender Item. The dewatering scheme shall be done in such a manner as to prevent any disturbance to the surrounding original soil.

BASIS OF PAYMENT

Payment at the contract price for this Tender Item shall include full compensation for all labour, equipment and material required to do the work.

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

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