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DRAFT

REPORT ON

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
DETROIT RIVER INTERNATIONAL CROSSING
BRIDGE APPROACH CORRIDOR

Submitted to:

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PART A

PRELIMINARY FOUNDATION INVESTIGATION REPORT
DETROIT RIVER INTERNATIONAL CROSSING
BRIDGE APPROACH CORRIDOR

1.0 INTRODUCTION

This report presents the results of geotechnical explorations and testing related to the highway access route portion of the Area of Continued Analysis (ACA) associated with the Detroit River International Crossing (DRIC) between Windsor, Ontario, and Detroit, Michigan. This work was undertaken by Golder Associates Ltd. (Golder) working under a subcontract to URS Canada Inc. (URS) as part of an on-going study for a joint partnership between the Ministry of Transportation Ontario (MTO), Transport Canada, the Michigan Department of Transportation (MDOT), and the US Federal Highway Administration (FHWA). Part A of this report (Sections 1 to 5) provides all data collected during field and laboratory work completed during this most recent phase of study for the bridge approach corridor aspects of the DRIC project. Part B of this report (Sections 5 through 10) provides geotechnical evaluations and recommendations for conceptual and preliminary design.

The terms of reference for the original scope of work issued during the proposal period are outlined in the MTO's Request for Proposal (RFP), dated September 2003, and in the scope of geotechnical work prepared by Golder included in the revised URS proposal dated January 2005. Scope changes related to completing borehole exploration and testing work are outlined in a Golder letter to the MTO dated January 26, 2006.

2.0 SITE DESCRIPTION

The highway access route portion of the Area of Continued Analysis (ACA) associated with the DRIC between Windsor, Ontario, and Detroit, Michigan begins near the existing western terminus of Highway 401 and generally follows the alignments of Highway 3, Huron Church Road and E.C. Row Expressway to one of several potential border crossing plaza sites near the Detroit River as illustrated on Figure 1. Conceptual alignments of the new highway access follow the horizontal alignments of these roadways with some alignments within the existing rights-of-way and some parallel to and west/south of the existing roadways. Where the new access highway will depart from the area of Huron Church Road and the alignment of E.C. Row Expressway, the new highway will parallel the south side of the existing expressway.

The site character near the existing terminus of Highway 401 to near the E.C. Row Expressway is generally residential or commercial with low-rise buildings and urban street rights-of-way. The topography in the area is relatively flat with the ground surface elevation gently undulating between about 187 m and 182 m with a general decline from southeast to northwest (toward the river). Within the Highway 3 and Huron Church Road portion of the ACA, the alignment crosses several municipal drains including the Cahill Drain, Lennon Drain, and Grand Marais Drain. Of these, the Grand Marais Drain (channelized section of Turkey Creek) is the most significant watercourse with an invert about 5 m to 6 m below the ground surface of the surrounding area. Side slopes of this drain are about 2 horizontal to 1 vertical where the section is channelized, and flatter in some areas where the creek takes its natural course.

Between Huron Church Road and Ojibway Parkway, along the south side of E.C. Row Expressway, the site is characterised by relatively low-lying and flat areas. The topography gently undulates with a topographic relief generally less than 5 m, between the approximate elevations of 179 m and 184 m, with a general decline from east to west toward the Detroit River. The ground surface is covered with a mixture of low vegetation and trees.

3.0 EXPLORATION AND TESTING PROCEDURES

Subsurface explorations were carried out for the ACA for the potential bridge approach roadways between October 2 and November 14, 2006. During this time, a total of four sampled boreholes (BH-1, BH-7, BH-14, and BH-23) and twenty-three cone penetration tests (CPT-1 to CPT-23) were advanced within the ACA (i.e. from the existing terminus of Highway 401 along Highway 3, Huron Church Road, and E.C. Row Expressway to Ojibway Parkway). An additional cone penetration test, CPT-24 was completed near the intersection of Provincial Road and Highway 401 to allow correlation of cone penetration test results with field performance at a location where a “back-analysis” of bridge structure and embankment settlements had been carried out (Golder 2006, GWP 64-00-00). Field vane shear testing using a push-in vane device was also completed adjacent to each of the borehole locations. Boreholes were numbered to be consistent with the full range of CPT test locations (i.e. BH-1 was located adjacent to CPT-1 and FV-1) and there are no boreholes with intermediate numbers. Locations of all boreholes, field vane shear tests, and cone penetration tests are shown on Figures 1A and 1B.

Field work was supervised on a full-time basis by a member of Golder's staff who located the boreholes and CPTs, directed the drilling, sampling, and in situ testing operations, and logged the boreholes. The soil and bedrock samples were identified in the field, placed in labelled containers and transported to Golder’s laboratories in Windsor and Mississauga for further examination and testing. All borehole and testing locations were determined by Golder relative to points staked in the field by Golder using GPS systems and measured references to local landmarks or features. Ground surface elevations were estimated using a digital terrain map (DTM) provided by URS Canada Inc. The borehole/CPT locations from the current investigation, including MTM NAD83 northing and easting coordinates and ground surface elevations, referenced to geodetic datum, are summarized in Table 3.1 and shown on Figures 2A and 2B. Boreholes had not been abandoned at the time of this report completion but must be abandoned in accordance with O. Reg. 128 (amendment to O. Reg. 903), or current applicable regulations, at such time the MTO concludes the work associated with this project.

The boreholes were advanced using both hollow stem augers and mud-rotary drilling using an all-terrain vehicle-mounted drill rig, supplied and operated by Lantech Drilling Services of Sharon, Ontario. The four sampled boreholes were advanced to depths (including rock coring) ranging from 25.4 m to 38.2 m below the existing ground surface. Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth, using either 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure, or thin-walled tube samplers. In situ vane shear strength testing within the boreholes was carried out using standard MTO vanes at regular intervals of depth, where appropriate, in clayey strata. In general, the sampling routine consisted of a repeating sequence of a split-spoon sample, a field vane shear test, and a thin-wall tube sample. In some areas, deviations from this sampling routine were necessary to assure recovery of sufficient thin-wall tube samples from critical depths. In the event that the soil strength was sufficient to prohibit completion of field vane shear tests, split-spoon samples were obtained instead. Samples of bedrock were obtained using an 'NQ'-size rock core barrel.

Water levels in the open boreholes were observed throughout the drilling operations and one 32 mm diameter standpipe observation well was installed in each of the boreholes BH-1, BH-7 and BH-14 to monitor the groundwater level(s) at the site. The screened portion of each standpipe was installed below the overburden-bedrock interface to measure groundwater pressures within the bedrock. Additional porous-tip piezometers were installed in unsampled boreholes immediately adjacent to the sampled boreholes to measure groundwater pressures within the overburden soils. These piezometers consisted of 13 mm diameter rigid CPVC pipe with a 300 mm long porous tip section, installed within a silica sand filter pack. Each piezometer or standpipe was sealed within specific geologic units using bentonite pellet backfill. The remainder of the annular space within all boreholes was filled with cement-bentonite grout. During drilling of Borehole BH-23, flowing artesian groundwater conditions were encountered. The borehole was sealed by filling it with cement-bentonite grout upon completion of coring; a separate piezometer was installed in an adjacent unsampled borehole that did not fully penetrate the clayey silt soil.

Twenty-four cone penetration tests (CPTs) were conducted along the ACA. Where necessary, shallow boreholes on the order of 3 m to 5 m in depth were advanced through the surface soils using solid stem augers in order to facilitate the start of the CPTs. The CPT is an in situ testing technique for site characterisation studies. No sampling or removal of soils took place during this drilling. The CPT consists of a special cone tip equipped with electronic sensing elements to continuously measure tip resistance, local side friction on a steel sleeve behind the conical tip and porewater pressure. It is pushed at a constant rate into the ground using a drill rig (ASTM D5778-95). A continuous stratigraphic profile together with engineering properties, such as strength, stress history and density, can be interpreted from the results of the CPT.

The CPT equipment was advanced using the hydraulic ram system on the drill rig. All CPTs were advanced to refusal, which was encountered at depths ranging from about 20.8 m to 29.3 m below ground surface. Record of Cone Penetration Test sheets are included in Appendix B following the text of this report. Profiles of tip resistance, porewater pressure during pushing and sleeve-friction are presented together with interpreted profiles of the classification index (I_c) used to infer soil type (stratigraphy), are included in Appendix B following the text of this report.

Push-in field vane shear testing was also conducted adjacent to Boreholes BH-1, BH-7, BH-14, and BH-23. This testing was carried out using the Nilcon Vane Borer device that provides a record of torque using a circular paper graph. The Nilcon Vane Borer also permits a closely controlled rate of rotational strain during the test using a geared drive such that close conformance with ASTM D 2573 can be achieved. Profiles of field vane shear strength were developed with tests completed at approximately 1 m intervals following advancement of a shallow borehole (on the order of 3 m to 5 m) through the stiff to hard upper overburden soils. No sampling or removal of soils took place during this testing. Profiles were obtained until refusal to penetration of the vane borer equipment was reached. Results from the Nilcon field vane shear tests are included in Appendix C of this report.

Laboratory testing, including water content determinations, Atterberg limits testing and grain size distribution analyses, was carried out on selected soil samples. Laboratory consolidation (oedometer) tests and consolidated isotropically, undrained, compression (CIUC) triaxial tests were also completed on twelve selected samples. All laboratory testing was carried out in accordance with applicable ASTM standards. The calcium carbonate and dolomite (magnesium carbonate) content of selected soil samples was tested using the "Chittick" test (Dreimanis 1962).

A simplified subsurface stratigraphy based on these exploration and testing methods is presented on Figure 3. The results of the field and laboratory testing are summarized on Figures 4 through 12. Figure 4 provides a comparison of field and laboratory tests conducted to determine undrained shear strength. Figures 5 through 9 summarize test results for each of the four sampled borehole locations. A simplified profile of estimated undrained shear strength is provided in Figure 10, and laboratory test results are summarized on Figures 11 and 12, with a summary table in Appendix A. Further discussion of the test results and subsurface conditions are provided in subsequent sections of this report.

TABLE 3.1 COORDINATES AND ELEVATIONS OF EXPLORATION LOCATIONS

TESTING LOCATION NUMBER	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	GROUND SURFACE ELEVATION (m)
BH-1	4677738	335500	186.70
CPT-1	4677739	335502	186.69
FV-1	4677744	335493	186.72
CPT-2	4677841	335185	186.35
CPT-3	4678022	334957	185.91
CPT-4	4678208	334516	185.09
CPT-5	4678413	334220	184.69
CPT-6	4678621	333844	184.08
BH-7	4678848	333325	183.17
CPT-7	4678844	333327	183.18
FV-7	4678842	333329	183.17
CPT-8	4678967	333109	182.48
CPT-9	4679105	332828	182.32

TESTING LOCATION NUMBER	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	GROUND SURFACE ELEVATION (m)
CPT-10	4679264	332533	181.81
CPT-11	4679634	332110	180.91
CPT-12	4680072	331924	181.61
CPT-13	4680350	331749	182.08
BH-14	4680648	331648	182.06
FV-14	4680652	331649	182.04
CPT-14	4680652	331651	182.06
CPT-15	4681049	331480	182.13
CPT-16	4681417	331376	181.93
CPT-17	4681625	331208	182.05
CPT-18	4681547	330938	180.65
CPT-19	4681906	330413	181.23
CPT-20	4681775	329868	179.76
CPT-21	4682147	329759	179.89
CPT-22	4682412	328986	178.89
CPT-23	4682325	328523	178.93
BH-23	4682323	328529	178.92
FV-23	4682322	328525	178.86
CPT-24	4679216	338376	190.20

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geological Conditions

The study area is located in the physiographic region of Southwestern Ontario known as the St. Clair Clay Plains. Within this region, Essex County and the southwestern part of Kent County are normally discussed as a subregion known as the Essex Clay Plain. The clay plain was deposited during the retreat of ice sheets (late Pleistocene Era) when a series of glacial lakes inundated the area. In general, the ice sheets deposited till in the area of Windsor and Detroit. Depending on the locations of the glacial ice sheets and depths of water in the ice-contact glacial lakes, the till may have been directly deposited at the contact between the ice sheet and the bedrock or, as the lake levels rose and the ice sheets retreated and floated, the soil and rock debris within and at the base of the ice were deposited through the lake water (lacustrine depositional environment). Glacial till, in its common usage, often indicates a very dense or hard composition resulting from consolidation and densification under the weight of the ice sheet. The mineral soil particles typically have a distribution of grain sizes ranging from cobbles to clay. In many areas of Windsor and Detroit, the soils described as “glacial till” were deposited through water and have a soft to firm consistency as a result.

The major soil stratum in the study area, consisting primarily of silty clay and clayey silt, typically ranging in thickness from about 20 m to 35 m, exhibits a till-like structure exemplified by a random distribution of coarser particles within the primarily fine-grained silt and clay deposit (also called “diamict”). In most of the eastern and northern parts of the Windsor metropolitan area below frost depth, the near-surface clay is generally stiff to hard and brown. Underlying this stiff to hard “crust”, the silty clay becomes grey-brown, and firm to stiff in consistency. Below the groundwater level, the silty clay becomes soft to firm, particularly in the western and southern areas of metropolitan Windsor.

Surficial layers or pockets of more typical layered lacustrine (lake-deposited) silty clay, silt, or sand may be encountered overlying the extensive stratum of “till-like” silty clay. Silt and sand deposits, on the order of 2 m in thickness, can often be found near the ground surface in areas near the western side of Windsor and the southwestern limits of the study area. A relatively thin stratum, on the order of 1 m to 6 m in thickness, of very dense or hard basal glacial till or dense silty sand may be found directly overlying the bedrock surface.

4.2 Site Stratigraphy

The detailed subsurface soil, bedrock and groundwater conditions encountered in the boreholes and inferred from the CPT's, together with the results of laboratory testing carried out on selected soil samples, are given on the attached Record of Borehole and Cone Penetration Test Sheets following the text of this report. The results of the laboratory testing are provided in Appendix A.

The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests (SPTs). These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Furthermore, subsurface conditions will vary between and beyond the borehole and CPT test locations.

In summary, the stratigraphy at the site of the borehole locations consists of relatively thin surficial layers of topsoil and fill, generally less than 1 m thick, overlying a thick deposit of clayey silt to silty clay. This clayey silt to silty clay deposit ranges in thickness between about 22 m and 33 m, based on the boreholes completed for this study. A dense to very dense layer of silty sand and gravel is found in some areas beneath the silty clay to clayey silt deposit and immediately overlying bedrock. Bedrock of the Hamilton Group (Dundee Formation) or Detroit River Group (Lucas Formation) was encountered at depths ranging from about 22.5 m to 33.5 m below the ground surface. The interpreted stratigraphy is illustrated on Figure 3.

4.2.1 Topsoil/Organics

A 0.2 m to 0.3 m thick layer of topsoil was encountered in Boreholes BH-1, BH-7, and BH-23. At the location of Borehole BH-23 the topsoil was covered by a layer of fill. Classification of this material was based solely on visual and textural evidence; testing of organic content or other constituents was not carried out.

4.2.2 Fill

Fill materials were encountered beneath the topsoil (where present) in Boreholes BH-14 and BH-23. The fill materials encountered at Borehole BH-14 extended from the surface to a depth of about 1.4 m. In Borehole BH-23, fill materials were found between depths of about 0.3 m and 0.6 m. At some of the cone penetration test locations, fill was encountered near the surface, preventing pushing of the instrument. Pre-drilling was carried out at these locations but no samples were taken as the drilling was used only to disturb and break up the soil above the start of the CPT.

4.2.3 Silty Sand to Sandy Silt

Silty sand to sandy silt was encountered at the location of Borehole BH-23 between the depths of about 0.6 m and 1.7 m. Classification of this material was based on auger cuttings and visual and textural evidence. The first sample interval penetrated only part of this layer and, therefore, the density, consistency, or composition were not tested. Soil index types inferred from the cone penetration results indicated relatively thin layers of sand on the order of 0.5 m to 1 m in thickness at the locations of CPT-16, CPT-21, and CPT-23. Measured tip resistance values within the silty sand to sandy silt were typically about 2 MPa to 3 MPa indicating a loose to medium relative density. Other CPT test results indicated minor seams (less than 0.2 m in thickness) of silty sand to sandy silt are present along the route, embedded within the silty clay to clayey silt deposit as illustrated on the records included in Appendix B.

4.2.4 Clayey Silt to Silty Clay

A thick deposit of clayey silt to silty clay was found in all boreholes completed for this project. The deposit is generally mottled grey and brown within and near the frost-depth (upper 1 m to 2 m), brown below this level, and grey below the static water level.

In general, the deposit consists mainly of low to medium plasticity clayey silt. The measured clay-size particle content of this deposit ranged between 25 and 40 percent (by weight); gravel-sized particles constituted between 1 and 9 percent; and the remainder of the samples were composed of sand and silt with silt being the larger component (37 to 45 percent). Results of Atterberg Limits determinations are illustrated on Figures A1a, A7a, A8b, A13a, A13b, A19a, and A19b in Appendix A. The plasticity index ranged between about 7 and 31, though most values fell between 10 and 20. Of the 33 samples subjected to Atterberg Limits testing, four exhibited liquid limits greater than 35 percent. The natural water content measured on select samples of this deposit ranged between 7 and 37 percent but was typically between 15 and 25 percent. The higher water contents are typically associated with the middle portion of the deposit. Total carbonate content ranged between about 19 and 32 percent.

Within the soft to stiff mottled brown and grey soils in these holes, the SPT “N” values ranged between about 4 and 14 blows per 0.3 m of penetration. The stiff to hard brown clayey silt and silty clay was encountered in Boreholes BH-1 and BH-7 in which SPT “N” values ranged from about 15 to 59 blows per 0.3 m of penetration. Boreholes BH-14 and BH-23 did not encounter the stiff to hard soils near the ground surface. Standard Penetration Test “N” values typically ranged between about 9 and 15 blows per 0.3 m of penetration in the grey silty clay below the groundwater level in Boreholes BH-1, BH-7, and BH-14 and between about 1 and 10 blows per 0.3 m in penetration in Borehole BH-23. In Boreholes BH-1, BH-7 and BH-14, the deposits became stiff to hard near the bedrock surface at depths ranging from about 22 m in Borehole BH-1 to about 29 m in Borehole BH-14. Within the stiff to hard lower part of the silty clay deposits, SPT “N” values ranged between 19 and 43 blows per 0.3 m of penetration. Very stiff soils were encountered for the last 2 m of drilling in Borehole BH-23, for which an SPT “N” value of 20 blows per 0.3 m of penetration was recorded.

In situ field vane testing carried out within this stratum measured undrained shear strengths ranging from 21 kPa to greater than 100 kPa. In general, the strength of the clayey silt was found to be greatest within the weathered “crust” (upper 2 m to 5 m), and decreased to values on the order of 40 kPa to 80 kPa within the middle of the stratum in Boreholes BH-1, BH-7 and BH-14, and then increased somewhat near the base of the profile. Within Borehole BH-23, the undrained shear strength profile was generally softer than in the other boreholes, with the measured strengths within the middle part of the profile ranging between 20 kPa and 40 kPa. The sensitivity, defined as the ratio of undisturbed field vane shear strength to remoulded field vane shear strength, ranged from about 1.3 to 7.2 (in the softest soil in Borehole BH-23) but was typically about 2.0. Consistency within the deposit ranges from soft to hard, depending on location and elevation. Figures 5 to 9 summarize the results of field testing of the undrained shear strength of the clayey silt to silty clay deposit, and a profile of measured and interpolated undrained shear strength is presented on Figure 10.

A total of 12 isotropically-consolidated triaxial compression tests with pore-water pressure measurements were completed on samples obtained within this deposit. The results of these tests, all of which are included in Appendix A, are summarized on a table in Appendix A, and on Figures 5 through 9, 11, and 12. Measured undrained shear strengths from the triaxial tests (consolidated to pressures generally consistent with in-situ conditions) were similar to the field vane shear test results. The measured effective-stress angle of internal friction and effective cohesion intercept were consistently between 25 and 26 degrees, and 9 kPa and 10 kPa, respectively for the four sets of three tests.

Twelve one-dimensional consolidation (oedometer) tests were completed on samples obtained from thin-wall tube samples. The locations/depths of the oedometer tests were selected to be consistent with the samples selected for triaxial compression testing. Values of interpreted “preconsolidation” pressure are summarized in a table preceding the laboratory test results in Appendix A, and are also shown graphically on Figures 5 through 9. The geometric average coefficient of consolidation, c_v, was found to be about 0.012 cm^2/s, and the geometric average coefficient of volume change, m_v, was found to be about 0.00015 m^2/kN. The results of an evaluation of the oedometer data provided the following correlations (see Figure 11):

$$C_c = 0.007w_n + 0.025$$
$$C_r = 0.11C_c$$

Cone penetration testing was completed along the ACA corridor at twenty-three locations with a twenty-fourth location near Provincial Road and Highway 401. The records of tip penetration resistance are included in Appendix B. All cone penetration tests met refusal at depths interpreted to be above the bedrock surface. Based on the borehole data, it is considered that the refusal depth indicated the transition from the firm to stiff silty clay to the stiff to hard silty clay. Table 4.1, below, summarizes the CPT refusal depths and elevations.

TABLE 4.1 CONE PENETRATION TEST REFUSAL DEPTHS

CPT LOCATION	GROUND SURFACE ELEVATION (m)	DEPTH TO CPT REFUSAL (m)	CPT REFUSAL ELEVATION (m)
CPT-01	186.69	24.90	161.79
CPT-02	186.35	24.12	162.23
CPT-03	185.91	25.00	160.91
CPT-04	185.09	23.16	161.93
CPT-05	184.69	23.92	160.77
CPT-06	184.08	27.04	157.04
CPT-07	183.18	27.96	155.22
CPT-08	182.48	28.80	153.68
CPT-09	182.32	25.16	157.16
CPT-10	181.81	27.14	154.67
CPT-11	180.91	25.74	155.17
CPT-12	181.61	29.30	152.31
CPT-13	182.08	26.92	155.16
CPT-14	182.06	27.98	154.08
CPT-15	182.13	28.18	153.95
CPT-16	181.93	22.24	159.69
CPT-17	182.05	25.78	156.27
CPT-18	180.65	26.22	154.43
CPT-19	181.23	26.86	154.37
CPT-20	179.76	27.98	151.78
CPT-21	179.89	23.24	156.65
CPT-22	178.89	21.24	157.65
CPT-23	178.93	20.84	158.09
CPT-24	190.20	24.02	166.18

A site-specific correlation using the CPT tip resistance (q_c) was developed and applied for interpreting undrained shear strength values at all CPT locations. The correlation considered the results of in situ vane shear testing and laboratory triaxial compression tests. The undrained shear strength estimated from the relevant CPT data was based on the following equation:

$$s_u = q_c / N_c;$$

where: s_u = undrained shear strength (kPa)
q_c = tip resistance (kPa)
N_c = cone factor of 16

4.2.5 Silty Sand and Gravel

Very dense silty sand and gravel was encountered beneath the silty clay to clayey silt in Borehole BH-14. This deposit exhibited SPT “N” values of 51 and 52 blows per 0.3 m of penetration. This deposit was not encountered in other boreholes completed for this project, though it is known from other work in the area that this deposit can be found throughout the ACA corridor immediately overlying the bedrock.

4.2.6 Bedrock

Limestone and dolostone bedrock of the Hamilton Group (Dundee Formation) or Detroit River Group (Lucas Formation) was encountered in the boreholes at depths ranging from about 22.6 m to 33.5 m below the ground surface, as shown in the table below. Based on the cores recovered from the boreholes, there may be a transition in bedrock formation between Boreholes BH-7 and BH-23. Such transitions in the bedrock formations encountered at the rock-soil interface may be expected in the general vicinity based on available mapping. The rock encountered in borehole BH-1 consisted of a light grey limestone, and in Borehole BH-14, the bedrock was composed of brown dolostone. Grey to brown limestone was encountered in Borehole BH-23, with some portions of the rock exhibiting a hydrocarbon odour. It is unknown whether the hydrocarbon odour is from natural sources, though some of the expected formations are known to contain natural bitumen. The rock encountered ranged from slightly weathered to fresh. Testing of one sample from each borehole measured unconfined compression strengths of 49.2 MPa (BH-1), 33.3 MPa (BH-7), 36.4 MPa (BH-14) and 55.4 MPa (BH-23). Rock quality designation (RQD) values ranged between 10 and 100 percent and were typically above 80 percent below the upper 2 m of rock. A description of some of the terms used in the description of the bedrock samples from this site is provided on the *Lithological and Geotechnical Rock Description Terminology* sheet that precedes the Record of Borehole sheets included with this report.

TABLE 4.2 DEPTH TO BEDROCK

BOREHOLE NUMBER	GROUND SURFACE ELEVATION (m)	DEPTH TO BEDROCK (m)	BEDROCK SURFACE ELEVATION (m)
BH-1	186.70	32.46	154.24
BH-7	183.17	33.15	150.02
BH-14	182.06	33.53	148.53
BH-23	178.92	22.56	156.36

4.3 Groundwater Conditions

Groundwater level measurements were obtained during the field work and these are summarized in the table below. Boreholes BH-1, BH-7 and BH-14 each included two piezometers installed as described in Section 3 and as shown on the Record of Borehole sheets. The upper piezometer was installed within the soil profile and the lower piezometer was installed within the bedrock or near the soil-bedrock interface. Borehole BH-23 included only one piezometer installed within the soil profile. Artesian groundwater conditions were observed at and below the soil-bedrock interface at Borehole BH-23, and a piezometer was not installed in the bedrock in this borehole to avoid the potential for long-term groundwater flow through or around the piezometer to the ground surface. For those boreholes in which groundwater was encountered below the ground surface during drilling (BH-1, BH-7, BH-14) the measured groundwater level during drilling will not necessarily be representative of actual groundwater conditions due to the low permeability of the soils, and the action of cutting and removal of soils. The final readings for each piezometer may be most reflective of static groundwater levels. The groundwater pressure elevations may be different within the overburden soils and bedrock. Groundwater levels are expected to fluctuate seasonally and are expected to rise during wet periods of the year.

The groundwater in the project area contains dissolved hydrogen sulphide that is liberated from the water on exposure to atmospheric pressure. Hydrogen sulphide gas was noted by its characteristic odour during drilling of Borehole BH-23 when the bedrock and artesian water pressures were encountered. Concentrations did not exceed the health and safety trigger levels of on-site monitoring equipment.

TABLE 4.3 MEASURED GROUNDWATER LEVELS

		OPEN BOREHOLE DURING DRILLING		UPPER (SOIL)		LOWER (BEDROCK)	
	DATE	**Depth (m)**	**Elevation (m)**	**Depth (m)**	**Elevation (m)**	**Depth (m)**	**Elevation (m)**
BH-1	04-Oct-06	10.1	176.6				
	06-Oct-06			Dry	Dry	10.0	176.7
	13-Oct-06			11.6	175.1	9.8	176.9
	20-Oct-06			8.5	178.2	9.6	177.1
	25-Oct-06			6.7	180.0	9.6	177.1
	27-Oct-06			6.0	180.7	9.5	177.2
	06-Nov-06			2.7	184.0	9.5	177.2
	14-Nov-06			2.3	184.4	9.3	177.4
BH-7	16-Oct-06	6.3	176.8				
	20-Oct-06			2.4	180.8	5.9	177.3
	25-Oct-06			4.2	179.0	5.9	177.3
	27-Oct-06			3.6	179.6	5.8	177.4
	07-Nov-06			3.4	179.8	5.7	177.5
	14-Nov-06			3.1	180.1	5.6	177.6
BH-14	24-Oct-06	2.9	179.2				
	24-Oct-06			5.2	176.8	3.1	179.0
	25-Oct-06			3.0	179.0	3.0	179.0
	27-Oct-06			2.9	179.2	3.0	179.0
	07-Nov-06			2.9	179.1	2.8	179.3
	14-Nov-06			2.7	179.3	2.8	179.2
BH-23	26-Oct-06	-1.7*	180.6*				
	27-Oct-06			dry			
	07-Nov-06			0.1	178.8	NA	NA
	14-Nov-06			0.0	178.9	NA	NA

* artesian water pressures observed during drilling.

5.0 CLOSURE

The technician supervising the field work was Mr. Chris Collins from the Golder Windsor office. The drilling company was Lantech Drilling Ltd. of Sharon, Ontario. The laboratory testing was performed by Golder in Mississauga, Ontario. The Foundation Investigation Report was prepared by Dr. Storer Boone, P.Eng., an Associate with Golder. Mr. Fintan Heffernan, P.Eng., Golder's Designated MTO Contact for this project, conducted an independent quality review of the report.

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PART B

PRELIMINARY FOUNDATION DESIGN REPORT
DETROIT RIVER INTERNATIONAL CROSSING
BRIDGE APPROACH CORRIDOR

6.0 CONCEPTUAL/PRELIMINARY DESIGN CONCEPTS

6.1 Introduction

Sections 6 through 9 of the report present geotechnical evaluations and recommendations related to the highway access route portion of the Area of Continued Analysis (ACA) associated with the Detroit River International Crossing (DRIC) between Windsor, Ontario, and Detroit, Michigan. This work was completed under a subcontract to URS as part of an on-going study for a joint partnership between the MTO, Transport Canada, the Michigan Department of Transportation (MDOT), and the US Federal Highway Administration (FHWA). This is the second part of a two part report (Part A and Part B). Part A provides all data collected to the date of this report during field and laboratory work completed for the bridge approach corridor aspects of the DRIC project. Part B is intended for evaluation of conceptual corridor alternatives and includes recommendations suitable for alternatives evaluation and preliminary design only. Further investigations and analyses will be required for final design activities or changes in conceptual design different from those described within this report.

The interpretation and recommendations provided are intended to provide the designers with sufficient information to assess the feasible project alternatives and to assist with conceptual and preliminary design. Where comments are made on construction they are provided only in order to highlight those aspects that could affect the concepts or design of the project. Those requiring information on aspects of construction should make their own interpretation of the factual information provided as it may affect equipment selection, proposed construction methods, scheduling and the like.

6.2 Project Description

The corridor is approximately 14 km in length between North Talbot Road and Ojibway Parkway and passes through several urban residential and commercial areas. Highway 401 may be extended from its current terminus at Highway 3 (Talbot Road East) northwest along Highway 3 to Huron Church Road, along Huron Church Road to the intersection with E.C. Row Expressway, and then adjacent to the E.C. Row Expressway to its intersection with Ojibway Parkway, as shown on Figure 1. A number of conceptual designs are or have been under consideration, typically involving six-lane urban freeway sections. The concepts revolve around three main methods of freeway construction through the corridor: at-grade, below-grade roadway, and below-grade tunnel constructed as a cut-and-cover structure. Variations on these include a new freeway aligned generally along the existing Highway 3 and Huron Church Road with service roads on both sides or with a freeway constructed adjacent to and south/west of these existing road allowances. The various options for each method of constructing the urban freeway sections are summarized as follows:

- At-grade with retaining walls, noise berms/walls, and service roads and with below-grade sections at each of the major cross-street locations where the new freeway will go beneath these streets.
- Below-grade roadway in an open excavation with sloped sides or retaining walls and service roads.
- Below-grade roadway within a cut and cover (or top-down) tunnel section (built in two halves) with service roads and parking lanes above the tunnel roof.

Figure 13 illustrates a number of conceptual roadway cross-sections. Each of the three major options described above may include several of the conceptual cross-sections or variations of these. Service roads may be located on both sides as two-lane roadways or together as a four-lane roadway on one side or the other of these conceptual cross-sections, depending on the outcome of further studies. Final concepts, including definition of the proportions of the freeway that may be constructed using these cross-sections, had not been completed at the time this report was prepared.

7.0 INTERPRETED SUBSURFACE CONDITIONS

A subsurface exploration program was carried out to supplement existing subsurface data in the project area for preparation of this report. All subsurface data generated through explorations conducted for this project are presented in Part A of this report. Existing information used to supplement the new data was gathered from MTO files (through the GEOCRES system), Golder project files, the Ontario Ministry of Natural Resources (MNR), Ontario Ministry of the Environment (MOE), and published papers and books. Data resources and references are listed at the conclusion of this report. Figures 2A and 2B illustrate the investigation locations and Figure 3 illustrates the simplified subsurface stratigraphy along the potential highway route. Figures 4 through 8 present summaries of the geotechnical properties at each of the borehole locations. The data from these locations were used in combination with the other CPT data to develop interpreted geotechnical properties along the entire route.

7.1 Stratigraphic Interpretation

The general physical characteristics and geology of the soils and rock encountered along the ACA are described within Part A of this report. For preliminary and conceptual design purposes, however, additional interpretation was required between the exploration location points. The amount and type of data collected using the cone penetration test facilitates some degree of quantitative interpretation rather than reliance solely on broad stratigraphic and behaviour indications provided by the boreholes. To supplement conventional borehole-to-borehole interpretations, geologic and environmental visualization software was used to assist with interpolation of engineering parameters and soil types between the cone penetration test locations.

For this project, the undrained shear strength of the silty clay and clayey silt soils and potential for water-charged granular layers are of critical importance as they relate to excavation stability. The environmental visualization software "EVS" (CTech 2006) was used to develop quantitatively-interpolated subsurface models of both soil type and undrained shear strength. The EVS software interpolates by "kriging"[1] data within a defined grid. This process incorporates uncertainty and error, considering the full data set, when estimating values in localized areas between data points with known values. The kriging process incorporates the principle of "semi-variance" which is a measure of the degree of spatial dependence between samples. The magnitude of the semi-variance between points depends on the distance between the points. The EVS software respects the validity of each known data point and defines the interpolation grid accordingly. In this case, the parameters of soil index type, I_c (defined by correlation between the CPT data and laboratory tests), and undrained shear strength (defined by correlation between CPT

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[1] "Kriging" is the term applied to statistical methods used for interpolation of value of a variable at an unobserved location based on observations of its value at nearby locations and is based on the work of Georges Matheron, *Traité de géostatistique appliqué, Editions Technip (1962)*, and Daniel Gerhardus Krige, *A statistical approach to some basic mine valuation problems on the Witwatersrand, J. of the Chem., Metal. and Mining Soc. of South Africa 52 (6): 119-139, (1951).*

and field vane shear tests), were used as the data set, based on all the available information. Use of the EVS software was complicated, however, by the variation in data resolution between boreholes and CPT locations. For instance, the boreholes and CPT locations were completed at horizontal spacings of about 190 m to 760 m, whereas the vertical resolution of data points at the CPT locations included data points nearly every 20 mm. The disparity between vertical and horizontal sample spacing was accounted for in the interpolation process by applying an anisotropy factor giving more weight to interpolation horizontally than vertically. While the interpreted subsurface conditions are suitable for conceptual and preliminary design purposes, additional investigation and analyses must be completed during the final design phase of this project.

7.2 Groundwater and Hydrogeologic Conditions

According to the Essex Region/Chatham-Kent (ECK) Regional Groundwater Study (MOE 2003), groundwater is not widely utilized for public water supply within the study region. Further, it is anticipated that within the Windsor metropolitan area, groundwater is likely not used for public water supplies and may only be used on limited bases for private water supplies, though a detailed survey of existing wells currently in use was beyond the scope of work for this project. Based on the MOE water well database, there are about eight mapped wells in the immediate vicinity of the proposed project between Highway 401 and the potential plaza locations. The MOE waterwell database (MOE 2005) indicates that these wells may not be currently in use. A more detailed survey of existing or pre-existing wells is beyond the scope of work for this project.

Section 4.3 (Groundwater Conditions) summarizes measured groundwater levels in the four boreholes completed as part of the work described in this report. Measured groundwater levels indicate that in the eastern part of the project area, near Boreholes BH-1 and BH-7, the groundwater exhibits a downward gradient. In this general area, pressure levels within the clayey silt to silty clay overburden do not exhibit hydrostatic pressures throughout the soil and rock profile. This condition is consistent with the generally low-permeability clayey silt to silty clay soils that will inhibit downward seepage of water from the ground surface to the static groundwater level. It is considered that the upper soils within the “crust” are fissured and likely of higher permeability than the native soils below the groundwater level. Within the weathered crust, there will be transitions in soil saturation from near-surface soils that become saturated with stormwater, down through the fissured, unsaturated soils (that exhibit mottled colouring), to the fully saturated soils below (grey in colour). Near-surface clayey silt and silty clay soils may also tend to pool stormwater in local surface depressions. Within the overburden soil, groundwater levels were measured near Elevations 180 m to about 184.5 m (or 2 m to 3 m below the ground surface), with the level to the north and west (BH-7) being lower than the level to the south and east (BH-1), as shown on Figure 3. In these same boreholes, however, measured groundwater levels within the bedrock were close to about Elevation 177.5 m, though there appears to be a trend of increasing levels from south and east to north and west, opposite the trend that may be indicated for those piezometers within the overburden. Between Borehole BH-7 and BH-14, water levels within the overburden soils drop slightly from about Elevation 180 m to about Elevation 179.5 m, while levels within the bedrock increase from about Elevation 177.5 to about Elevation 179.5 m. Further to the west, near Borehole BH-23, the groundwater levels within the overburden remains relatively consistent with that of Borehole BH-14 near about Elevation 179 m to 179.5 m and, in this area, close to the ground surface. Within the bedrock, however, the groundwater level rises, such that at this location, the groundwater within and near the bedrock surface is artesian with respect to the ground surface, with a pressure head at about Elevation 180.5 (or about 1.5 m above ground surface). Two interpreted groundwater pressure elevation lines are illustrated on Figure 3, showing the conditions expected near the top of the saturated soils and near the soil/bedrock interface.

The observation well data indicate, therefore, that there may be a general trend along the potential project alignment of groundwater levels within the overburden soils decreasing from southeast to northwest while bedrock groundwater levels exhibit the opposite trend. It is considered that the trend of decreasing groundwater levels within the overburden is generally reflective of the weathering profile and inhibited infiltration of surface water through the low-permeability clayey silt and silty clay soils, combined with generally declining ground surface elevations from southeast to northwest along the ACA. The trend in groundwater elevation within the bedrock is also considered generally consistent with groundwater flow patterns between Lake St. Clair, the Detroit River, and areas to the northwest flowing southeast, toward the Lake Erie basin. Though there is evidence supporting these general conclusions, project-specific hydrogeological conditions within the overburden and bedrock will be dependent upon local variations in soil permeability, surface watercourses (or municipal drains such as the Lennon, Cahill and Grand Marais Drains), surface topography and bedrock topography. Additional explorations and testing will be required during final design to refine these general conclusions.

Hydraulic conductivity (permeability) of the overburden soils was measured using the oedometer tests and is discussed further in Section 7.3.5. Transmissivity of the bedrock was not measured as part of the scope of work completed for this study. It is noted, however, that the bedrock in the area is considered to be a “good” water-yielding hydrogeologic unit (MOE 2003) and that for wells installed in this formation, the mean transmissivity was estimated to be on the order of 30 m^2/day with specific capacities on the order of 5 to 50 litres per minute per metre (L/min/m) for individual water wells drawing from the Detroit River Group or Dundee hydrogeologic units.

7.3 Interpreted Engineering Parameters

Part A of this report provides a summary of the test results completed during the supplementary investigation carried out for this project. For conceptual and preliminary design, however, test data interpretations and correlations were developed to assist with estimating both the feasibility and potential field performance of construction options. The interpretations and correlations are described in greater detail below.

7.3.1 Undrained Shear Strength

As discussed in Part A of this report, two types of field vane shear tests were completed at this site. The conventional vane shear testing device was used as well as the Nilcon Field Vane Borer testing device (see Appendix C). The conventional vane was used in each borehole and the Nilcon device was used adjacent to each borehole. The Nilcon field vane provided useful additional measurements of field undrained shear strength for soft soils. This device also provides a continuous record of angular rotation and torque to interpret rod friction, peak, post-peak, and remoulded shear strengths while allowing close control of shear rates. The Nilcon vane was advanced without drilling through much of the soil profile at the project site, except where the surface crust was too strong to allow direct pushing of the device. In general, the standard vane indicated strengths about 20 percent greater than the Nilcon device as illustrated on Figure 4. This behaviour is consistent with past work by others (e.g. De Lory and Salvas 1969) and other work by Golder in southwestern Ontario. The differences in undrained shear strength indicated by the two tests are considered to be the result of differing strain rates during testing in slightly to moderately overconsolidated soils where excess negative porewater pressures from relatively rapid strain rates influences the measured shear strengths. The standard vane data, however, provide a useful comparison to other projects and investigations in which this device was used to gauge in situ undrained shear strength. The plasticity index of the silty clay soils along the ACA corridor typically ranged between about 7 and 31, though most values fell between 10 and 20. Based on these values, the correction factor to be applied to field vane shear tests (Bjerrum 1954) ranges between 1.0 and 1.1. Therefore, the field vane shear tests were not corrected for plasticity.

A site-specific correlation between the CPT tip resistance (q_c) and undrained shear strength was developed considering the field vane shear test results as well as the laboratory CIUC testing. The undrained shear strength from the relevant CPTs was interpreted using the following equation:

$$s_u = q_c / N_c;$$

where: s_u = undrained shear strength (kPa)
q_c = tip resistance (kPa)
N_c = cone factor

The “cone factor” was chosen such that the calculated undrained shear strength was in reasonable agreement with the typical range of the Nilcon field vane shear tests, with consideration also given to the results of the standard MTO vane and laboratory test results. Based on the field vane shear tests and such comparisons, the cone factor was chosen to be $N_c = 16$. This value is within typical ranges for similar soils (Kulhawy and Mayne 1990). Figure 4 illustrates a comparison between undrained shear strength values determined using the above relationship and each of the other testing methods.

Laboratory tests were also carried out to estimate the undrained shear strength of the overburden soils. A total of 12 tests were completed using consolidated isotropically, undrained-compression tests with porewater pressure measurements (CIUC). All samples were consolidated to an all-around confining pressure on the order of one-third to one-half the existing vertical effective stress so as not to compress the soils past their one-dimensional yield stress point (“preconsolidation pressure”) or destroy the sample structure.

Peak undrained shear strength from these tests are presented on Figures 5 through 9. The results of the CIUC tests are consistent with both the Nilcon field vane shear tests as well as the interpreted CPT tests.

For the purposes of conceptual and preliminary design, a design line was chosen to represent the variable undrained shear strength profiles at each of the borehole locations. These design lines are illustrated on Figures 5 through 9. Within the upper silty clay “crust”, where field tests indicate relatively high undrained shear strength values, the design line departs from the test results. It has been shown, for construction of embankments on soft ground in particular, that the operative shear strength of the ground mass in such crusts is less than measured peak strengths yet greater than remoulded strengths. Therefore, the approaches of Lefebvre et al. (1980) and Tavenas and Leroueil (1987) as well as the measured post-peak values were considered in defining a design shear strength in the weathered silty clay crust.

7.3.2 “Preconsolidation” Pressure

The “preconsolidation” pressure is a critical parameter for use in settlement calculations and represents the change from small to large strain compression behaviour under one-dimensional loading. While typically referred to as the “preconsolidation” pressure, this point in the stress-strain curve can be the result of many factors other than mechanical preloading as the term implies. This yield stress can be influenced by weathering (wetting and drying cycles) and cementation and it is known that the soils in southwestern Ontario can be lightly cemented (e.g. Brown 1970, De Lory and Salvas 1970, Boone and Lutenegger 1996). Interpretation of oedometer tests in the till-like soft soils in southwestern Ontario can be problematic as the nature of the soils tends to produce curves that do not have a distinct change in behaviour that clearly demarcates the “preconsolidation” pressure. Settlement calculations based on such ambiguous determinations of “preconsolidation pressure” typically overestimate field settlements. The oedometer tests completed for this project were interpreted using conventional Casagrande methods, the work per unit volume method (Becker et al 1987), and a simplified approach whereby:

- the conventional "virgin" compression index based on the maximum slope of the oedometer compression curve was used to extrapolate a tangent line from the maximum oedometer compression stress to the in situ vertical effective stress;
- the conventional "recompression" index from an unload-reload cycle straight line approximation, was used to extrapolate a line from the in situ vertical effective stress point on the oedometer compression curve to the maximum oedometer compression stress; and
- the stress corresponding to the intersection of these lines was selected as an unambiguous effective vertical one-dimensional yield stress ("preconsolidation pressure").

For this project, the simplified approach described above was selected as the method for defining the vertical effective yield stress because the method and the parameters derived from the testing program provided an excellent correlation with the measured settlements at Structure 6-074 that is discussed in more detail in Section 8.3.2.

In addition to the oedometer tests, the CPT tests were also used to infer "preconsolidation" pressure profiles. To define such a profile using the CPT, the undrained shear strength values, as determined from calibration to the Nilcon vane shear test results as described above, were used as follows (after Mesri 1975):

$$s_u = 0.22\sigma_p\text{'} \text{ or for the preconsolidation pressure, } \sigma_p\text{'} = s_u/0.22$$

where: s_u = average mobilized undrained shear strength (kPa)
σ_p' = preconsolidation pressure

The interpreted effective vertical yield stresses ("preconsolidation pressure") from the oedometer tests are illustrated on Figures 5 through 9 with respect to the existing in situ effective soil stresses and porewater pressures A comparison of estimated preconsolidation pressure using the CPT and oedometer tests is provided on Figure 4. A comparison of preconsolidation pressures derived using the Schmertman (1955) method (after Soderman and Kim, 1970), the simplified method as described above, correlation with the CPT-derived undrained shear strength, and a back-analysis of settlement is illustrated on Figure 9. As described herein, the back-analysis utilized a simplified subsurface profile with uniform consolidation parameters. Use of the preconsolidation pressure profile as determined for this report compared well with measured settlements. Therefore, the correlated CPT-derived preconsolidation pressure-defined profiles are considered most suitable for preliminary design purposes.

7.3.3 Stress-Strain Properties

Determination of the stress-strain properties of the soils is critical to assess potential displacements of the ground and associated retaining structures under the loads that may be imposed by the new construction. Determination of these parameters was accomplished through the laboratory tests conducted for this project, examination of other laboratory test results from Golder files, and comparison to published correlations and theoretical relationships. Correlations developed for this project are illustrated in Figures 11A and 11B.

One-dimensional consolidation properties were determined based on the results of oedometer tests completed both for the DRIC project and other projects in the vicinity. “Virgin” compression index C_c values were defined based on the slope of the oedometer compression curve between the last two loading increments. The “recompression” index C_r was based on a correlation of recompression and virgin compression indexes developed from data within Golder files for projects completed in Windsor. These parameters were found to be readily related to the natural water content of the specimens, w_n, as the water content is indicative of the amount of void space within the sample, and since the nature of the soil particles, mineralogy, and geologic environment are relatively consistent across the site. Oedometer test data was also used to define the coefficient of consolidation, c_v, that is related to the time-rate of settlement. The results of data evaluation provided the following correlations (see Figure 9):

C_c = $0.007w_n + 0.026$
C_r = $0.11C_c$
c_v = $0.0094\ \text{cm}^2/\text{s}$ (based on geometric average of all data)

These correlations are consistent with published correlations for similar soil types (e.g. Holtz and Kovacs 1981, Kulhawy and Mayne 1990).

During triaxial testing (CIUC tests), each sample was subjected to unloading and reloading at a fraction of the failure stress. The subsequent stress-strain data developed from the full tests was evaluated to define non-linear stress strain properties (e.g. Duncan and Chang 1970). Elastic modulii were developed for three positions within the stress strain curve: (1) initial tangent modulus, E_{uit}; (2) secant modulus at 50 percent failure stress, E_{s50}; and (3) unload-reload modulus, E_{ur}. The data evaluation provided the correlations below (also see Figure 11). These values are generally consistent with, though somewhat lower than, published correlations for similar soil types (e.g. Kulhawy and Mayne 1990).

E_{uit} = $420S_u$ or $E_{uit} = 250p_a(\sigma_c/p_a)^n$ where $n = 0.8$
E_{ur} = $2E_{uit}$
E_{s50} = $E_{uit}/2$

where: p_a = atmospheric pressure
σ_c = isotropic confining pressure

7.3.4 Effective Stress Strength Parameters

Estimation of the drained strength parameters (internal angle of soil friction) was based on the results of the laboratory triaxial testing. Figure 12 illustrates the peak effective angle of internal friction. Considering an effective cohesion intercept of about 8 kPa, the effective angle of internal friction was determined to be about 26 degrees, based on a best-fit linear regression of the test data. The corresponding effective angle of internal friction for an assumption of an effective cohesion intercept of zero was determined to be about 30 degrees. These values are generally consistent with published correlations for similar soil types (e.g. Kulhawy and Mayne 1990).

7.3.5 Permeability

Permeability of the clayey silt to silty clay was measured during oedometer testing. Values of permeability, measured at stresses less than the preconsolidation pressure, generally ranged from 2.4×10^{-8} to 1.2×10^{-6} cm/s with a geometric average of 2.4×10^{-7} cm/s. These measurements of permeability, however, are only considered appropriate for small specimens of the clayey silt to silty clay deposit. In situ permeability of this deposit below the groundwater level may be as much as one-half to one order of magnitude greater than these values. Above the groundwater level, permeability values may be as much as one to two orders of magnitude greater due to the effects of weathering and fissuring of the overall soil mass. Horizontal permeability may be between one half and one order of magnitude greater than vertical permeability.

8.0 RECOMMENDATIONS FOR PRELIMINARY DESIGN

Recommendations for conceptual and preliminary design of the planned approach roadway structures are provided within this section of the report. These recommendations address the following:

- overview and definition of different retaining structure technologies;
- selection and design of retaining systems for the DRIC project;
- temporary and permanent cut slopes;
- factors of safety as related to excavations within the clayey silt and silty clay soils;
- construction and subsequent settlements of high fill embankments;
- bridge foundations for cross-street overpasses;
- seismic design considerations;
- dewatering;
- use of excavated materials and fills;
- instrumentation and monitoring; and
- investigations related to final design.

These recommendations are intended to assist with conceptual and preliminary design and should not be used for final design or construction. Refinements to these recommendations may be necessary for final design pending further investigations, testing, exploration and analysis in comparison to the selected design concept. Indeed, it is expected that a final investigation and geotechnical design report will prepared for each of the major structures along the ACA.

8.1 Earth Retaining Systems

Earth retaining systems will be required for at-grade sections that incorporate noise barrier berms integral with retaining walls, for below-grade roadway sections constructed in cuts, and for the approaches to any tunnel section where cut slopes cannot be utilized. Selection of the appropriate temporary and permanent retaining systems for the corridor depends on the cost, final surface finish requirements, available subsurface easements, available surface easements for open excavation, lateral earth pressures, groundwater control requirements, and vertical roadway clearance available for horizontal braces between the walls.

Earth retaining systems typically can be grouped into two categories based on the means by which they are constructed. Gravity walls are constructed “bottom up” from the base of a cut, and then backfilled. In situ walls are constructed by building a wall face in the ground either before or while the ground is excavated to create the grade difference. In situ retaining systems can also be categorized considering two main components: the structural wall face; and the lateral restraint system which resists the horizontal earth pressures. Depending on the type of wall constructed, the combination of the embedment of the wall below the excavation bottom and the structural capacity of the wall may be sufficient to resist the horizontal earth load (cantilever walls). If construction of a cantilever wall is not feasible, horizontal displacement of the excavation sidewalls is commonly restrained by inclusion of one or more of the following: internal braces placed between two opposing wall faces; steel rods or wires (strands) drilled into and anchored in the ground behind the wall (tie-backs, ground anchors, or soil nails); or the base friction arising from the weight of either the wall itself or of backfill placed on top of an integrally connected footing.

The following discussion identifies various permanent retaining systems for each option under the two main construction methods. The feasibility of each system has been evaluated on a conceptual level based on technical considerations such as compatibility with ground conditions as understood based on available information; installation and workspace requirements; economic factors such as installation costs; and minimization of traffic disruption. This information may be used by the DRIC project team to assist in analysis or refinement of alternatives.

Below-grade roadway sections can be built either in sloped road cuts or in cuts where the sidewalls are permanently supported. Cut-and-cover tunnel sections can be constructed either in a top-down or bottom-up fashion. In top-down construction, a permanent excavation sidewall is constructed followed by a permanent deck that can be established as a finished permanent roadway prior to undertaking further below-ground work. Excavation to form the tunnel and base then proceeds beneath the deck without further traffic disruptions. In bottom-up construction, construction of the temporary or permanent walls is followed by excavation to the required depth, building the base, sidewalls or wall facing, and finally constructing the deck/roof section and re-establishing the surface roadway.

The following table summarizes the types of retaining systems that were considered for each method of construction:

OPTION	PERMANENT RETAINING SYSTEMS	GENERAL WALL TYPE
At-Grade Noise Berm Retention	Cast-in-Place Reinforced Concrete Wall	Gravity
	Pre-Cast Cantilever or Counterfort Wall	Gravity
	Crib or Bin Wall	Gravity
	Mechanically Stabilized Earth (MSE) Wall	Gravity
	Soldier Piles and Lagging Wall (with permanent facing)	In Situ
Below-grade Roadway: Open Cut	Cast-in-Place Reinforced Concrete Wall	Gravity
	Pre-Cast Cantilever or Counterfort Wall	Gravity
	Crib or Bin Wall	Gravity
	MSE Wall	Gravity
	Soil Nail Wall	In Situ
	Soldier Pile and Lagging Wall	In Situ
	Secant or Tangent Pile (Caisson) Wall	In Situ
	Driven Sheet or Interlocking Pipe Pile Wall	In Situ
	Concrete Diaphragm (Slurry) Wall	In Situ
	Soil-Cement Mix Wall	In Situ

OPTION	PERMANENT RETAINING SYSTEMS	GENERAL WALL TYPE
Below-grade Roadway: Covered Cut, Top-Down Construction	Secant or Tangent Pile (Caisson) Wall	In Situ
	Concrete Diaphragm (Slurry) Wall	In Situ
	Soldier Pile and Lagging Wall	In Situ
Below-grade Roadway: Covered Cut, Bottom-Up Construction	Soldier Pile and Lagging Wall	In Situ
	Soil Nail Wall	In Situ
	Driven Sheet or Interlocking Pipe Pile Wall	In Situ
	Concrete Diaphragm (Slurry) Wall	In Situ
	Secant or Tangent Pile (Caisson) Wall	In Situ
	Soil Cement Mix Wall	In Situ

8.1.1 Gravity Wall Systems

Gravity wall systems are generally constructed in such a manner that the weight of the wall resists the lateral loads and consequent overturning forces from the ground behind the wall. The weight of the wall structure can be provided by the following:

- heavy stone masonry (little used for modern walls);
- cast-in-place structural concrete;
- soil integrally mated with a reinforced wall facing material (mechanically stabilized earth);
- earth or stone fill within structural systems (e.g., bins or cribs); and
- soil resting on structural concrete members of the wall (cast-in-place concrete cantilever walls).

These mechanisms may be used separately or combined in a variety of forms. Some of the general wall types are described in greater detail below. Gravity wall systems are generally backfilled with free-draining granular soils so as to control water and frost pressures. It should be noted, however, that obtaining granular backfill in the Windsor area is generally more costly than in other regions of Ontario.

8.1.1.1 Cast-in-Place Concrete Walls

Until the advent of pre-cast concrete wall systems, cast-in-place concrete walls were the most common retaining wall constructed. In the simplest form, a large mass of formed concrete can be cast-in-place with the resistance to the lateral loads from the retained earth provided by the dead-weight of the concrete, the friction at the wall base, and the resistance offered by the soil at the wall toe. Cast-in-place concrete cantilever walls are constructed for many projects for a number of reasons:

- once the structure is in place, the backfill behind the wall can sometimes be excavated without destabilizing the wall;
- design and construction methods are well established; and
- their shape can be made to fit complex grading and site conditions.

In general, concrete cantilever walls are constructed in the shape of an inverted ‘T’ or in the shape of an ‘L’, where soil is placed on top of the horizontal wall “footing” to effectively increase the wall mass. Typically, cast-in-place concrete walls achieve support of the retained earth through the following mechanisms:

- overturning moment is resisted by the counteracting direction of the soil weight on the footing;
- sliding of the wall is resisted by friction along the wall base and any soil in front of the wall; and
- the integrity of the wall structure itself is maintained by the structural capacity of the wall face and the footing connection.

Cast-in-place concrete wall systems can be constructed within a temporary excavation support system or an open cut if space permits. For wall heights in excess of 6 m to 8 m, structural support can be achieved by internal or external counterforts (buttresses) as well as structural connections with the base slab or footing. Walls of this type generally require either an open or shored excavation with a base width approximately equal to about one-third to one-half of the final wall face height. Though conventional cast-in-place concrete walls are highly adaptable and common in their design and construction methods, their cost can exceed the cost of other available walls for similar project conditions, especially if shoring is required to support the existing earth.

8.1.1.2 Mechanically Stabilized Earth (MSE) Walls

The earth behind a wall can be stabilized and included in the mechanism for resisting the lateral loads of the native ground. Such walls are generally identified as "mechanically stabilized earth" (MSE) walls or "retained soil system” (RSS) walls. Typically, mechanical stabilization of wall backfill is achieved by the following:

- placing and compacting a layer of earth backfill (typically 0.3 m to 0.6 m in thickness);
- laying steel straps, steel wire grids, or plastic grids (polypropylene, polyester, polyethylene) on the surface of the backfill layer as reinforcing elements;
- attaching a structural face (typically consisting of interlocking concrete blocks or panels) to the reinforcing elements;
- placing and compacting additional backfill on top of the reinforcing elements; and
- repeating the above sequence until a structural face is provided to the required height, with the mass of earth stabilized with internal “reinforcement” behind the face.

These walls achieve their internal stability by virtue of the friction and interlocking of the reinforcing strips or grids and the backfill. Resistance to sliding and overturning is accomplished in the same manner as for cast-in-place walls. For the wall face to fail, the connection to the strips/grid must break or the strips/grid must pull out of the backfill. Many systems are available for constructing mechanically stabilized wall systems. These systems are often patented with respect to the method of earth reinforcement, attachment of the reinforcement to the structural face, and/or the structural face finish and interlocking mechanisms.

Some “walls” can be constructed using the principles described above, but utilize a geotextile fabric alone or in combination with a grid to replace the concrete facing. In such walls, the fabric and grid are wrapped over the front edge of each successive layer of backfill, producing a face that is constructed primarily of fabric. Where necessary, such walls can be sprayed with concrete or seeded for landscaping if the wall/stabilized earth face is sloped.

To construct a mechanically stabilized earth system, it is necessary to have an open excavation or to construct the wall as a “fill” wall, whereby the retaining wall and backfill are placed above existing grades. Many highway ramps and grade separations are constructed using such wall systems.

Mechanically stabilized earth walls offer the advantage that they are relatively inexpensive and rapid to construct and, depending on the wall facing units, can be more tolerant of differential settlements than cast-in-place concrete walls. In most cases, MSE walls are not used in areas in which the underlying soil consists of soft silty clay and where measurable differential settlements are anticipated. This restriction on the use of MSE walls arises from a concern that differential settlements could cause breakage at the corners of facing blocks that would ultimately cause premature deterioration of the wall.

Mechanically stabilized earth walls typically require an open excavation that includes a level area from which to build the wall that is approximately equal to 0.6 to 0.75 times the wall face height back from the wall face. Creating such an open area requires extensive earthwork where such walls are used in support of earth “cuts.” In general, for support of cuts, other wall systems are often more economical. “Walls” constructed of wrapped geosynthetic products are generally not suitable where aesthetics are important elements of the project and, depending on their design and construction, can be less durable than other feasible earth support systems. Since mechanically stabilized earth systems rely on both the backfill soil and reinforcement elements for support, excavation into the reinforced zone behind the wall must be restricted, providing a constraint on future infrastructure construction behind such walls.

8.1.1.3 “Crib” and “Bin” Walls

Crib walls derive their generic name by virtue of their construction method. Some crib-wall systems are composed of interlocking timber or pre-cast concrete “stretchers” (similar in size and shape to railroad ties). The stretchers are used to build an interlocking “crib” in which earth backfill is placed. “Bin” walls are typically constructed of relatively thin-walled, pre-cast concrete blocks, open at their top and bottom. As the “bins” are placed, their interior is filled with either compacted earth or stone. The bins and cribs are either constructed from the bottom of an open excavation, or from the ground surface for support of fills. A flat area approximately equal to about one-half of the final wall height is generally required for construction of bin and crib walls. With these wall types, a self-supporting structure is created.

Bin and crib walls offer the advantage that the area required at the base of an open excavation is less than that for mechanically stabilized systems, they are relatively rapid to construct, and can be reasonably tolerant to settlement or deformation, dependent upon the details of their construction. Crib walls are more commonly used where surface appearance is of little importance. Bin and crib walls, however, are generally more expensive than mechanically stabilized earth walls.

8.1.1.4 Pre-Cast Cantilever or Counterfort Walls

A number of pre-cast versions of conventional cantilever or “counterfort” retaining walls are available. In general, the walls are constructed at concrete pre-casting plants to standard panel dimensions. Once at a construction site, the pre-cast panels are then attached to a cast-in-place concrete footing with similar dimensions as for cast-in-place concrete walls. These walls offer the advantages of construction speed and elimination of much of the formwork. The relative cost of pre-cast walls of this type depends on the required variation in panel geometry, local pre-casting facilities, and construction sequencing needs, and may be either greater or less than the cost of cast-in-place concrete walls.

8.1.2 In Situ Wall Systems

In situ walls include a broad range of retaining systems characterized by constructing the face of the wall in-place, as opposed to creating a sloped excavation and building the wall from the bottom to the top. The primary advantage of in situ walls is that they generally do not require excavation behind the wall face. Lateral support is provided by either anchoring back beyond the retained ground during mass excavation in front of the wall, or by providing bracing within the excavation. Such systems do not incorporate free-draining granular backfill behind the wall facing and thus other measures must be taken to resist or control groundwater and frost pressures. Some in situ wall types are suited better for particular soil types, whereas others may be suitable for a broad range of soil types. Uncontrolled fill materials, depending on their local composition, or other subsurface obstructions (e.g. rubble, boulders, cobbles) may be problematic for all in situ wall types. For this project, obstructions such as boulders and cobbles are not anticipated to be present within the top 20 m of the native soil profile.

8.1.2.1 Soil-Nail Wall

Temporary and permanent retaining walls can be constructed using the soil nailing technique whereby the ground is supported by inserting reinforcing steel rods (“nails”) into the ground on a regularly spaced vertical grid, covering the excavation face with steel mesh and spraying on a facing of shotcrete that is structurally connected to the nails. The permanent facing can then be constructed of successive layers of shotcrete, precast panels, or a cast-in-place concrete face that, in northern climates, is placed over an insulating layer. In essence, soil nailing creates the “reinforcing” of the mechanically stabilized earth systems without excavating the native ground behind the wall.

The length of the soil nails is usually 0.6 to 1 times the height of the wall and less than what is used in tie-back or conventional soil anchor construction. The design of a soil nail wall can be readily adapted to fit curved or shaped topographic forms. The equipment is generally portable, requires relatively little space and generates less noise and requires less manpower than other methods.

Soil nailing is most economical in ground that can stand unsupported for at least one day on a vertical or steep slope cut 1 m to 1.8 m height and in which drill holes can remain open for at least several hours. This method of construction is best suited for use in deposits of dense granular and stiff low plasticity clayey soils. Also, groundwater must be well controlled such that seepage does not lead to excavation face instability during the initial construction.

Soil nail walls are constructed from within the area to be excavated and require between about 6 m and 10 m of working space in front of the wall for equipment. The space required for working will depend on the equipment chosen, staging, and routes required for earth moving equipment.

8.1.2.2 Driven Sheet-Piles

For open-cut excavations in loose or soft soils, excavation support and the permanent retaining structure can sometimes be provided by driven sheeting. Such walls can be designed as cantilever walls or with “dead-men” anchors (depending on loading and easements), permanent tie-backs, or internal bracing for lateral support. The choice of a cantilever wall versus one supported by permanent horizontal restraints will depend on the height, soil and groundwater loadings, structural details of the wall, space restrictions, and ground movement tolerances. Such walls are more commonly used where surface appearance is of little importance such as for shipping dock bulkheads or along freight railway corridors. These systems are typically more flexible than thick concrete walls and, therefore, permanent facings, if used, are generally designed to be relatively independent of the more flexible steel sheeting. An insulating layer is also generally required for such walls in areas subjected to freezing conditions.

Driven sheet piles are readily available and effective for the soft ground conditions that will be encountered in the project area. In addition to conventional sheet pile sections, some driven steel walls may consist of interlocking pipe and sheet piles. Such wall systems can typically provide greater bending stiffness than conventional sheeting alone. This method is not suitable for soils that contain substantial obstructions such as boulders, or that are very dense. Installation requires use of sheet pile impact hammers or other vibratory drivers.

Construction equipment for installing a driven sheet pile wall can generally operate within a working zone of about 7 m to 10 m width with the wall at nearly any position within that working zone. Equipment for installation generally consists of mobile cranes suitable for lifting both the steel sheets and operating the vibratory hammer, compressors, and other equipment for delivery of sheets. In many cases, sheeting cannot be installed abutting structures or other features that are vibration intolerant.

8.1.2.3 Secant or Tangent Pile (Caisson) Wall

Secant or tangent pile walls are constructed by drilling holes between 0.9 m and 1.2 m in diameter to the full depth of the wall, inserting steel reinforcement in the form of steel beams or reinforcing bars, and filling the holes with concrete. Tangent pile walls are constructed by having the drilled holes immediately adjacent to one another and secant pile walls are formed by having each pile overlap the adjacent pile. Secant pile walls are preferable where groundwater or soft/loose soils must be controlled. In soft or caving soils, or where the piles are drilled below groundwater levels, the hole sides and bottom must be supported during drilling with fluids, steel casings, or both. Such walls can be constructed as either temporary or permanent walls. Permanent secant or tangent pile walls often have a permanent cast-in-place or precast concrete facing to fill any gaps between piles, provide protection for an insulating layer, and provide a smooth or architecturally appropriate surface finish. These walls can be designed as cantilever walls (up to a site-specific limiting height), with permanent tie-backs, or with internal bracing for lateral support. The choice of a cantilever wall versus one supported by permanent horizontal restraints will depend on the height, soil and groundwater loadings, structural details of the wall, space restrictions, and ground movement tolerances. In some cases, where tie-backs or bracing are not feasible, piles as large as 2 m in diameter can be constructed to allow high cantilever walls. In addition, if carried to a suitable bearing layer or if the wall is of sufficient penetration depth within a relatively competent ground layer, secant pile walls can serve as foundations or vertical load-bearing elements for overlying or attached structures.

The main advantages of secant or tangent pile walls are increased construction alignment flexibility, increased wall stiffness compared to sheet piles, control of groundwater by pile interlock, and the ability to be used in difficult ground containing cobbles or boulders. The main drawbacks are that vertical tolerances may be hard to achieve for deep piles (greater than about 20 m in depth), costs are higher than those for sheet pile walls, and waterproofing may be difficult to achieve at joints. There is also the possibility of ground loss and seepage through any gaps that may be present and that can require remedial work during and after construction.

Construction equipment for installing a secant pile wall can generally operate within a working zone of about 7 m to 10 m width, with the wall at nearly any position within that zone. Equipment for construction of secant or tangent pile walls generally consists of mobile drill rigs (some of which are based on a track-mounted crane platform), cranes suitable for lifting steel reinforcement, and other equipment for delivery of reinforcement and concrete. In some cases, the walls can be installed abutting property limits, buildings, or other features that are intolerant of significant vibration.

8.1.2.4 Soil-Cement Mix Wall

Soil-cement mix walls can be used alone or in conjunction with traditional techniques. In order to reduce steel requirements for temporary shoring, soil-cement mix walls can be constructed as part of a soldier pile and tie-backs system. Soil-cement mix systems can also provide increased stability in deep cuts in ground prone to deep-seated failures when used between the retaining walls.

In general, soil-cement mix walls are constructed by using drilling equipment to produce a hole filled with soil cuttings; cement grout is then injected into the loosened ground, and the grout and soil are mixed with drilling equipment to produce a column of soil-cement slurry. The drilling, injection, and mixing can be accomplished using a variety of equipment configurations including single-flight augers modified with injection points and mixing blades, overlapping continuous-flight augers, or jet grouting equipment. The type of equipment chosen for a particular project typically depends on cost and availability of proprietary systems developed and patented by various contractors. Soil-cement mix walls can be reinforced by inserting steel H or W sections (as soldier piles) into the soil-cement column. As with the secant and tangent pile walls, a final facing is generally required for architectural purposes, drainage, and frost protection and, with soil-cement walls, for surface durability as well.

Soil-cement mix walls have particular application for some soils as the procedure modifies the ground properties so that they are similar to a soft rock or low-strength concrete. This method is not suitable for soils containing more than about 10 percent peat and mixing of soft clay soils must be carefully controlled to avoid large pockets of untreated soils.

Soil-cement mix walls may be of lower relative cost when compared to secant pile or cast-in-place concrete diaphragm walls, as this general wall type has sometimes been used as an alternative wall construction technique on some projects. However, the relative cost depends on many factors particular to the project including soil type, performance requirements, equipment and availability of experienced contractors.

Construction equipment for installing soil mix walls can generally operate within a working zone of about 7 m to 10 m width with the wall at nearly any position within that zone. Equipment for construction of these walls generally consists of mobile drill rigs (some of which are based on a track-mounted crane platform), cranes suitable for lifting steel reinforcement, and other equipment for delivery of reinforcement and concrete. In some cases, the walls can be installed abutting property limits, buildings, or other features that are intolerant of significant vibration.

8.1.2.5 Soldier-Pile and Lagging

Soldier pile and lagging systems are commonly used, can be constructed in a variety of ground conditions and, as with other wall types, can be designed and constructed as either cantilever walls or walls supported by horizontal supports. The choice of a cantilever wall versus one supported by permanent horizontal restraints will depend on the height, soil and groundwater loadings, structural details of the wall, space restrictions, and ground movement tolerances. Soldier pile and lagging systems can be used in place of sheet piling where the soil is bouldery or quite dense. To reduce the noise and vibration usually associated with sheet pile installation and to penetrate through dense or bouldery ground, the piles are typically installed in pre-drilled holes. In soft or caving soils, or where the piles are pre-drilled below groundwater levels, the hole sides and bottom must be supported during drilling with fluids, steel casings, or both. The wall is installed by boring a series of 0.5 m to 1.0 m diameter holes, spaced 2 m to 3 m apart, into which H-piles (soldier beams) are installed; the annular space is then filled with a relatively low strength sand-cement concrete mix. As the excavation proceeds, 50 mm to 100 mm thick boards are inserted behind the front flanges or placed against the piles and clipped to the front flange using fasteners. Concrete lagging, shotcrete or steel sheeting can be used in place of wood. For permanent installations, pre-cast concrete lagging may also be used provided that alignment is closely controlled during installation of the piles in pre-drilled holes. Permanent soldier pile and lagging walls must also include provisions for frost protection and control of any groundwater seepage. The lagging is often installed in lifts of 1 m to 1.5 m, depending on the ground conditions. In addition, if carried to a suitable bearing layer or if the wall is of sufficient penetration depth within a relatively competent ground layer, the soldier piles can serve as foundations or vertical load-bearing elements for overlying or attached structures.

Soldier pile and lagging can be installed at relatively low cost and the installation method can be adapted to poor ground conditions. Excavations will have to be carefully monitored for subsidence and lateral movement particularly when structures or utilities are located nearby.

Since ground loss is more common with this system than sheet piles and some other systems, construction of soldier pile and lagging retaining systems must be carefully controlled, especially in built-up areas.

Construction equipment for installing a soldier-pile and lagging wall can generally operate within a working zone of about 7 m to 10 m width, with the wall at nearly any position within that zone. Equipment for construction of soldier pile walls generally consists of mobile drill rigs (some of which are based on a track-mounted crane platform), cranes suitable for lifting steel reinforcement, and other equipment for delivery of reinforcement and concrete. In some cases, the walls can be installed abutting property limits, buildings, or other features that are intolerant of significant vibration.

8.1.2.6 Cast-in-Place Concrete Diaphragm Wall

Commonly called “slurry walls”, cast-in-place concrete diaphragm walls are constructed by excavating a deep, narrow trench, filling the trench with a viscous slurry (of clay and water or polymers and water) to keep the trench from collapsing, placing reinforcing steel within the trench, and then placing the final concrete from the bottom up, displacing the slurry. Typical trench widths are on the order of 0.6 m to 1 m. These walls offer the advantage that they can serve as both temporary excavation support and the permanent wall, depending on the design details. Concrete diaphragm walls that are constructed as the permanent structural wall often are provided with a cast-in-place or precast facing to improve the architectural finish and address frost protection and/or drainage issues. Such walls can be designed as cantilever walls, with permanent tie-backs or with internal bracing for lateral support. The choice of a cantilevered wall versus one supported by permanent horizontal restraints will depend on the height, soil and groundwater loadings, structural details of the wall, space restrictions, and ground movement tolerances. If necessary, diaphragm walls can be constructed in “T”-shaped sections to permit high cantilever walls without additional internal bracing or permanent tie-backs; however, construction of this type is relatively rare within North America. In addition, if carried to a suitable bearing layer or if the wall is of sufficient penetration depth within a relatively competent ground layer, concrete diaphragm walls can serve as foundations or vertical load-bearing elements for overlying or attached structures.

Slurry walls are suitable for construction of walls in both caving and cohesive soils. They may be necessary for locations where sheet piling or soldier piles and lagging are not applicable or where greater control over ground deformations or groundwater infiltration is required. However, slurry walls can be approximately twice as expensive as these systems. Much of the high cost is attributable to requirements for specialized equipment, more stringent field control, and management and disposal of slurries.

Construction equipment for installing a concrete diaphragm wall can generally operate within a working zone of about 10 m width with the wall at nearly any position within that zone. Equipment for construction of these walls generally consists of mobile cranes suitable for lifting the trench excavating equipment (often a clam-shell bucket), steel reinforcement, slurry processing plants, and other equipment for delivery of reinforcement and concrete. In some cases, the walls can be installed abutting property limits, buildings, or other features that are intolerant of significant vibration.

8.1.3 Horizontal Restraint Systems

8.1.3.1 Internal Struts/Braces

For temporary excavation support, the walls of the excavation can be propped using steel beams or pipe sections placed between the walls as the excavation proceeds. These struts are often removed during construction of the permanent structure if the structure and backfill over the structure will be sufficiently strong to resist the permanent earth loads. Some permanently retained walls for grade separation projects have used permanent internal struts placed near the top of the walls. Permanent struts can be constructed of steel, cast-in-place concrete, or pre-cast concrete depending on the required structural dimensions, design-life performance goals, cost, and construction considerations. Struts present a disadvantage during construction since they obstruct the working space within the excavation. Permanent struts also obstruct the space within the excavated area and are subject to weathering and thermal stresses and require long-term maintenance. However, struts offer the advantage that once in place, the excavation and all the wall systems are contained within the limits of the support walls.

Strut spans are typically limited to about 20 m (when using pipe struts) unless vertical support is provided to inhibit bending or buckling due to the combination of axial and self-weight loads. Larger spans are possible, but installation of supporting piles and multiple strut-to-pile connections can contribute to the complexity of the supports, congestion of the working space, and displacements of the wall and surrounding ground. The horizontal and vertical spacing of the struts will largely depend on the stiffness of the vertical wall elements, the loads that are distributed to the struts, and tolerable displacements of the ground and facilities around the excavation. Typically, the spacing of struts (both vertically and horizontally) is limited to about 5 m, though larger spans can be achieved. In some cases, vertical spans between struts on the order of 8 m to 10 m can be achieved, though the required bending moment capacity of the vertical wall elements must be substantially greater than typical excavation support installations. It may also be necessary to install wales – long structural sections that support the wall horizontally between supports. Wales can consist of steel sections or, in the case of permanent installations, cast-in-place reinforced concrete.

8.1.3.2 Tie-Backs/Ground Anchors

Tie-backs, also called ground anchors, are constructed by drilling cased horizontal or sub-horizontal holes into the ground behind the wall as the excavation proceeds downward. Once a hole is drilled, steel rods or high-strength steel strands are inserted into the hole. An “anchor zone” is then created by filling the annular space around the steel rods or strands with cement grout. Often, the cement grout is injected under pressure in several stages after the initial grout is placed by gravity flow. The anchor zone is typically located beyond the “active” earth zone behind the wall (the mass of earth that deforms and places load on the wall). Once the grout is cured, the anchor is prestressed to its design load and structurally connected to the wall. Tie-backs offer an unrestricted excavation or permanent underground space once they are in place. Tie-backs, however, typically cost more than internal bracing for long and narrow temporary excavations. In addition, subsurface easements are typically required from neighbouring properties if the tie-backs extend beyond existing rights-of-way or property boundaries. Permanent tie-backs can limit future subsurface uses for neighbouring areas since the integrity of the tied-back walls depends on the ground around the tie-backs remaining undisturbed. For planning purposes, it may be assumed that the anchors may extend back from the face of the wall a distance equal to twice the excavation depth.

The horizontal and vertical spacing of the tie-backs will largely depend on the stiffness of the vertical wall elements, the loads that are distributed to the tie-backs and the capacity of the ground in which they are anchored to resist the load, tolerable displacements of the ground and facilities around the excavation, and the cost for installing the tie-backs. Typically, the spacing of tie-backs (both vertically and horizontally) is limited to about 5 m, though larger spans can be achieved. In some cases, vertical spans between tie-backs on the order of 5 m to 8 m can be achieved, although the required bending moment capacity of the vertical wall elements must be substantially greater than typical excavation support installations. It may also be necessary to install wales – long structural sections that support the wall horizontally between supports. Wales can consist of steel sections or, in the case of permanent installations, cast-in-place reinforced concrete.

For this project, the use of tie-backs may largely be limited to temporary installations in the upper firm to stiff clay crust. The deeper soft silty clay is unlikely to be capable of providing adequate resistance for anchoring tie-backs. Should excavations penetrate relatively deep into the silty clay, it may be necessary to extend any ground anchors into bedrock to provide adequate resistance capacity. Depending on the angle at which tie-backs are installed, the vertical component of the tie-back load can be significant and the design of earth retaining systems must take this vertical load into account. Vertical wall members must be capable of supporting the vertical load component while maintaining vertical settlement within tolerable limits. Excessive settlement of the wall can lead to loss of tension in the tie-backs and poor performance of the entire excavation support system. Thus, if tie-backs are used, it may be necessary to extend the vertical wall elements to bedrock.

8.2 Selection and Design Of Retaining Systems for Below-Grade Roads

Depths of excavation and the corresponding heights of retaining structures have been based on vertical profile alternatives provided by URS, as shown on Figures 14 through 16. These include a below-grade road in both a cut and in a cut-and-cover tunnel. In general, the deeper cuts may be required to cross under the significant municipal drains such as the Grand Marais (Turkey Creek) Drain. Depths of excavation and retaining wall heights for selected points along the roadway are provided in Table 8.2.1, below, accounting for approximately 1.2 m or 3.25 m of additional cut below the finished grade for constructing the road base or cut-and-cover tunnel base slab, respectively. The larger of these two cut depths is related to the depth of structure required for long cut-and-cover tunnel ventilation ducts beneath the roadway (see Figure 13). As noted previously, the selection of a suitable retaining system is based on several factors including cost of installation, compatibility with soil conditions, workspace requirements (surface or subsurface easements) and limitation of movements.

A summary table is provided following the text of this report that compares the various advantages and disadvantages of retaining structures as related to this project. Although the summary table provides an overview of retaining wall selection considerations, the relative advantages, disadvantages and costs of these will be highly dependent upon the total quantity of each that may be selected, traffic control and logistics details, and the final conceptual design geometry. The summary table should not be used out of context from this report nor should it be used without thorough consideration of the discussions provided below and other, non-geotechnical requirements of this project.

TABLE 8.2.1 DEPTHS OF EXCAVATION FOR BELOW-GRADE HIGHWAY SECTIONS

INTERSECTION OR PROFILE POINT	GROUND SURFACE ELEVATION (m)	AT-GRADE ROAD		BELOW GRADE ROAD		CUT & COVER TUNNEL	
		Road Elev. (m)	Cut Depth[1] (m)	Road Elev. (m)	Cut Depth (m)	Road Elev. (m)	Cut Depth (m)
Highway 401	186.7			186.7	1.2	187.7	
Howard Ave.	186.0	178.7	8.9	179.4	8.1	173.4	16.2
Low Point 1	186.0			177.4	9.7	172.4	16.7
High Point	185.8			179.4	7.7	174.7	14.4
Montgomery Dr.	185.5			179.0	7.3		
Low Point 2	184.0			176.3	9.6		
High Point (Tunnel)	184.8						8.9
Cousineau Road	184.0	176.4	9.3	177.2	8.1	171.1	16.3
High Point 2	183.8			177.8	7.5		
St. Clair College	183.3	175.3	9.1	176.5	7.9	173.3	13.1

INTERSECTION OR PROFILE POINT	GROUND SURFACE ELEV. (m)	AT-GRADE ROAD		BELOW GRADE ROAD		CUT & COVER TUNNEL	
		Road Elev. (m)	Cut Depth[1] (m)	Road Elev. (m)	Cut Depth (m)	Road Elev. (m)	Cut Depth (m)
Low Point 3	182.0			174.7	8.9	170.7	15.0
High Point 3	182.0			176.1	7.4	171.7	13.9
Huron Church Line	181.9	172.2	10.8	175.1	7.9	171.6	13.5
Todd Lane	182.0	173.0	9.1	172.6	9.5	170.0	14.2
Low Point 4[2]	181.8	172.5	10.8	170.9	12.4	169.5	15.8
High Point 4[2]	181.8			176.9	6.4	170.7	14.7
Turkey Creek High	182.0			180.0	3.3		
Turkey Creek Low	182.0	173.5	9.8	174.9	8.3	165.9	19.4
Low Point 5[2]	182.0			170.6	12.8		
High Point 5[2]	182.0			176.3	11.6	171.1	18.8
Bethlehem Avenue	182.0			173.5	9.6	171.0	14.2
Low Point 7[2]	180.8			173.1	10.1	169.6	15.7
High Point 7[2]	180.8			182.6	0.6		
Basin Drain	180.0			182.0	-0.2	180.6	

NOTES: 1) Depths shown only for crossroads that highway passes beneath.

2) Different high and low points provided for different options depending on whether Turkey Creek is conveyed beneath the highway via an inverted siphon or box culvert or the highway passes beneath Turkey Creek.

8.2.1 Temporary Cut Slopes

For the purpose of a preliminary comparison of retaining systems, it is considered that, generally, temporary cut slopes will be stable at slopes of between 1 and 2 horizontal to 1 vertical. From Highway 401 to E.C. Row Expressway, the soils become progressively softer at the anticipated cut depth, except for a short section between Turkey Creek and E.C. Row Expressway. Therefore, for planning purposes, it may be assumed that temporary cut slopes in the area near the terminus of Highway 401 could be about 1:1 (horizontal:vertical), whereas slopes closer to 2:1 may be necessary along Huron Church Road to the E.C. Row Expressway as the soils become softer and excavation and cut slope stability become more sensitive to depth and slope angle. Temporary construction slopes between these locations may be interpolated between these values based on the distance between the end locations. There are limitations to the maximum depth of temporary cut slopes. Factors of safety for stability of temporary cut slopes in the cohesive silty clay and clayey silt soils will be similar to those associated with deep excavation stability as provided in Table 8.2.2 in the report section below, depending on the duration over which the cuts are made and left open. Temporary cut slopes should be stable at all excavation depths for the below-grade roadway profile illustrated on Figure 15 (excluding cuts for cut-and-cover tunnel) except near Todd Lane where a factor of safety of about 1.1 has been calculated. In this area, temporary cut slopes should not be constructed; rather, excavations in this area will require use of temporary excavation support and additional measures to enhance base stability. Further discussion on base stability enhancements is provided in subsequent sections of this report. During construction, the slopes must be protected from erosion and subsequent surficial failures or formation of erosion channels. Permanent slopes will need to be flatter than temporary slopes and a separate section of this report provides relevant preliminary design recommendations for permanent slopes.

8.2.2 Factors of Safety for Excavation Stability

Three principal conditions affect the stability of deep, vertical, and supported excavations in silty clay and clayey silt soils, as follows:

1. the strength of the soil relative to the depth of the cut, sometimes called “global stability,” or stability against “base heave” (e.g. Peck 1969, NAVFACS 1986, Clough and O’Rourke 1990, CFEM 2006);
2. upward seepage of groundwater carrying fine granular soils leading to loss of ground, or “piping” instability (e.g. NAVFACS 1986, CFEM 1992/2007); and
3. upward groundwater pressures on cohesive soil layers sufficient to overcome the weight and strength of the overlying soil and uplift the bottom of the excavation, or “uplift instability” (e.g. Milligan and Lo 1970, CFEM 1992/2007, Shirlaw 2006).

8.2.2.1 Base Heave (Soil Strength)

The factor of safety for excavation global stability is generally defined by the depth of the excavation and the undrained shear strength of the soil near and below the base of the excavation as follows:

$$\text{Factor of Safety} = \frac{5.5\ S_u}{\gamma_t H}$$

Where Su = undrained shear strength
γ_t = total unit weight of soil (21 kN/m³)
H = depth of excavation or exposed height of retained structure

The excavation plan dimensions (length and width) can also influence the stability factor of safety. For the purposes of this feasibility assessment, the excavations have been considered to be at least twice as long as they are wide. For the cases evaluated, the undrained shear strength was taken as the average strength of the soil between the base of the excavation and an additional depth equal to between 0.5 and 1.0 times the excavation depth. Figures 14 through 16 illustrate the base stability factor of safety for global stability along the proposed approach route, and Table 8.2.2 summarizes calculated factors of safety for several selected locations and corresponding excavation depths. In general, for temporary excavations, it is recommended that the factor of safety for global stability (sometimes called base stability) be at least 1.3. Figures 14 through 16 illustrate the estimated factor of safety along the proposed route. The actual factor of safety will differ depending on the dimensions of the excavation and design of support systems, and will be influenced by conditions above and below the points indicated on the figures. Figures

14 through 16 are provided only as a guide for conceptual and preliminary design to assist in defining where additional stability enhancement works may be required. For permanent cuts, it is recommended that the factor of safety (based on undrained shear strength considerations) be about 2, so as to limit the potential for undesirable long-term creep displacements. The shaded entries in Table 8.2.2 indicate where stability is considered insufficient for the temporary (t) conditions. Table 8.2.2 also indicates where stability is considered insufficient to maintain long-term lateral creep displacements to acceptable values for the permanent (p) conditions. It should be noted that stability for permanent conditions should be satisfactory for the cut-and-cover tunnel options provided that the temporary stability is satisfied. The rigidity of the tunnel structure should limit long-term lateral creep displacements to negligible values.

TABLE 8.2.2 BASE HEAVE FACTORS OF SAFETY FOR EXCAVATIONS

INTERSECTION	AT-GRADE OPTION				BELOW GRADE OPTION				CUT AND COVER TUNNEL OPTION			
	Road Elev (m)	Cut Depth (m)[1]	Su[4] (kPa)	Factor of Safety	Road Elev (m)	Cut Depth (m)[1]	Su[4] (kPa)	Factor of Safety	Road Elev (m)	Cut Depth (m)[1]	Su[4] (kPa)	Factor of Safety
Highway 401					186.7	1.2			187.7	2.2		
Howard Avenue	178.7	8.9	57	1.7 (p)	179.4	8.1	57	1.8 (p)	173.4	16.2	61	1.0 (t)
Low Point 1					177.4	9.7	60	1.6 (p)	172.4	16.7	78	1.2 (t)
High Point					179.4	7.7	60	2.0	174.7	14.4	78	1.4
Montgomery Drive					179.0	7.3	58	2.1				
Low Point 2					176.3	9.6	56	1.5 (p)				
High Point (Tunnel)									175.8	8.9	56	1.7
Cousineau Road	176.0	9.3	63	1.8 (p)	177.2	8.1	65	2.1	171.1	16.3	88	1.4
High Point 2					177.8	7.5	65	2.3				
St. Clair College	175.3	9.1	61	1.8 (p)	176.5	7.9	59	2.0	173.3	13.1	55	1.1 (t)
Low Point 3					174.7	8.9	44	1.3 (p)	170.7	15.0	55	1.0 (t)
High Point 3					176.1	7.4	37	1.3 (p)	171.7	13.9	40	0.8 (t)
Huron Church Line	172.2	10.8	57	1.4 (p)	175.1	7.9	59	2.0	171.6	13.5	65	1.3
Todd Lane	173.0	9.1	50	1.4 (p)	172.6	9.5	51	1.4 (p)	170.0	14.2	58	1.1 (t)
Low Point 4[2]	172.5	10.8	41	1.0 (t &p)	170.9	12.4	42	0.9 (t & p)	169.5	15.8	58	1.0 (t)
High Point 4[2]					176.9	6.4	45	1.8 (p)	170.7	14.7	55	1.0 (t)
Turkey Creek High					180.0	3.3	67	>2.5				
Turkey Creek Low	173.5	9.8	47	1.3 (p)	174.9	8.3	47	1.5 (p)	165.9	19.4	66	0.9 (t)
Low Point 5[2]					170.6	12.8	47	1.0 (t & p)				
High Point 5[2]					176.3	11.6	47	1.1 (t & p)	171.1	18.8	62	0.9 (t)
Bethlehem Avenue					173.5	9.6	56	1.5 (p)	171.0	14.2	50	0.9 (t)
Low Point 7[2]					173.1	10.1	72	1.9 (p)	169.6	15.7	77	1.3

NOTES:
1) Analyses not carried out for intermediate positions or depths;
2) Different high and low points provided where different options pass over Turkey Creek (either in box culvert or siphon) or beneath Turkey Creek;
3) Temporary condition indicated (t), permanent condition indicated (p)
4) Undrained shear strength taken as the average value for the mass of ground between the base of the excavation and an additional depth of 0.5 to 1.0 times the excavation depth.

In Table 8.2.2 and on Figures 14 through 16, it is shown that the soil strength in a number of areas is not sufficient to maintain temporary excavation stability without implementing other construction measures. The areas in which such conditions may be anticipated are illustrated on Figures 14 through and 16 with respect to the various vertical alignment alternatives. A number of construction techniques may be used to permit excavation to the required depths at these locations, such as the following:

- extending the penetration of retaining system walls to well below the base of the excavation into the stiff to hard clayey silt/silty clay near the bedrock interface, or into the bedrock;
- installation of below-grade struts in slurry-filled trenches;
- construction of relatively thick concrete base slabs under slurry or water; or
- improving the ground at the excavation base using techniques such as jet grouting or deep soil mixing; and
- where the factor of safety is insufficient for permanent conditions, using thickened concrete pavements or buried struts may provide adequate long-term displacement control.

Preliminary recommendations related to conceptual selection and design of stability enhancement measures are provided in subsequent sections of this report.

8.2.2.2 Upward Seepage – “Piping” Instability (Groundwater Flow)

Since it is anticipated that the ground conditions will primarily consist of low permeability clayey silt to silty clay from near the ground surface down to near the bedrock surface, base stability for the below grade roadways or cut-and-cover tunnel with the excavation depths as provided in Table 8.2.2 is not expected to be influenced by piping failure mechanisms, except if uplift failure occurs and free flow of groundwater into the excavation subsequently occurs. If deeper excavations are required, however, measures to mitigate piping stability may require additional consideration.

8.2.2.3 Base Uplift Instability (Groundwater Pressure)

The factor of safety for “uplift stability” will be governed by the depth of soil remaining between the bottom of the excavation and the top of the granular soils or bedrock. This factor of safety is calculated as the buoyant forces acting on the base of the low permeability cohesive soil (clayey silt to silty clay) layer and resisted by the weight of the soil above:

$$\text{Factor of Safety} = \frac{21\ \text{kN/m}^3\ (\text{base of excavation elevation – base of cohesive soil elevation})}{9.81\ \text{kN/m}^3\ (\text{groundwater surface elevation – base of cohesive soil elevation})}$$

As these engineering parameters are more certain, a factor of safety of about 1.1 is considered adequate for temporary construction. Frictional forces are sometimes included in calculating the base uplift factor of safety; however, in such cases a higher factor of safety is often used to address the greater uncertainty in frictional parameters. The table below summarizes calculated factors of safety (neglecting soil friction) for uplift stability at selected locations along the ACA route.

TABLE 8.2.3 BASE UPLIFT FACTORS OF SAFETY FOR EXCAVATIONS

INTERSECTION	APPROXIMATE GROUNDWATER PRESSURE ELEV. (m)	FACTOR OF SAFETY[1]		
		At-Grade Option	Below Grade Option	Cut and Cover Tunnel Option
Highway 401	181.7		2.4	
Howard Avenue	182.0	1.8	1.9	1.3
Low Point 1	181.9		1.7	1.1
High Point	181.9		1.9	1.4
Montgomery Drive	181.9		1.9	
Low Point 2	181.9		1.7	
High Point (Tunnel)	181.9			1.7
Cousineau Road	180.1	1.8	1.8	1.3
High Point 2	180.1		1.9	
St. Clair College	178.4	1.8	1.9	1.5
Low Point 3	178.5		1.8	1.3
High Point 3	178.4		1.9	1.4
Huron Church Line	178.4	1.6	1.8	1.4
Todd Lane	178.4	1.7	1.6	1.3
Low Point 4[2]	179.5	1.6	1.5	1.1
High Point 4[2]	179.5		1.9	1.3
Turkey Creek High	177.1		2.3	
Turkey Creek Low	177.1	1.8	1.9	1.0
Low Point 5[2]	179.8		1.4	
High Point 5[2]	179.8		1.8	1.3
Bethlehem Avenue	179.9		1.6	1.1
Low Point 7[2]	179.0		1.6	1.1

NOTES: 1) Analyses not completed for intermediate positions or depths;
2) different high and low points provided where different options pass over Turkey Creek (either in box culvert or siphon) or beneath Turkey Creek

In all cases, except for the Turkey Creek area of the cut-and-cover tunnel option, it is estimated that uplift stability may be maintained without undertaking other groundwater pressure control measures for the main excavations. If excavations should penetrate below the depths as considered for these roadway options, these may require that either groundwater pressures be lowered temporarily (e.g. Conlon et al. 1971), or that alternative construction techniques be employed in order to build a base slab that can resist the upward hydraulic pressures.

Construction of the vertical members of in situ walls (drilled piles, excavated “slurry” trenches, etc.) will require excavation to depths well below the depths noted in the tables above. These localized conditions will be subject to the same principles governing base heave and uplift stability. For all such excavation, it is recommended that, for preliminary planning and design purposes, all drilled pile holes or excavated slurry trenches (for diaphragm walls) be filled with a properly designed drilling slurry to counteract heaving or uplift forces.

8.2.3 Gravity Walls for Support of Grade Cuts

Gravity walls for support of roadway cuts are most economical for shallow excavations or for wall heights up to about 5 to 6 m. Gravity walls for roadway cuts generally require a working space behind the face of the wall in the range of two to three times the wall height to account for the base width and back slope of the cut. Use of MSE, cast-in-place or pre-cast concrete cantilever walls supported on shallow foundations may be suitable for areas in which the assessed bearing capacity is sufficient. The estimated bearing capacities, based on the recent explorations and testing, for selected locations along the potential roadway alignment and at the planned cut depths, are summarized in Table 8.2.4, on the following page.

TABLE 8.2.4 SHALLOW FOUNDATION RESISTANCE AND MAXIMUM HEIGHT FOR GRAVITY WALLS IN CUT SECTIONS

INTERSECTION	AT-GRADE OPTION					BELOW-GRADE OPTION					CUT AND COVER TUNNEL OPTION				
	Cut Depth (m)[1]	Su (kPa)	SLS (kPa)	Factored ULS (kPa)	Max Height (m)	Cut Depth (m)[1]	Su (kPa)	SLS (kPa)	Factored ULS (kPa)	Max Height (m)t	Cut Depth (m)[1]	Su (kPa)	SLS (kPa)	Factored ULS (kPa)	Max Height (m)
Highway 401						1.2					2.2				
Howard Avenue	8.9	60	100	150	4.3	8.1	60	100	150	4.3	16.2	60	100	150	4.3
Low Point 1						9.7	60	100	150	4.3	16.7	80	125	200	5.4
High Point						7.7	60	100	150	4.3	14.4	80	125	200	5.4
Montgomery Drive						7.3	60	100	150	4.3					
Low Point 2						9.6	55	100	150	4.3					
High Point (Tunnel)											8.9	55	100	150	4.3
Cousineau Road	9.3	65	100	150	4.3	8.1	65	100	175	4.3	16.3	90	150	225	6.5
High Point 2						7.5	65	100	175	4.3					
St. Clair College	9.1	60	100	150	4.3	7.9	60	100	150	4.3	13.1	55	100	150	4.3
Low Point 3						8.9	45	75	125	3.3	15.0	55	100	150	4.3
High Point 3						7.4	40	75	100	3.3	13.9	40	75	100	3.3
Huron Church Line	10.8	60	100	150	4.3	7.9	60	100	150	4.3	13.5	65	100	175	4.3
Todd Lane	9.1	50	75	125	3.3	9.5	50	75	125	3.3	14.2	60	100	150	4.3
Low Point 4[2]	10.8	40	75	100	3.3	12.4	40	75	100	3.3	15.8	60	100	150	4.3
High Point 4[2]						6.4	45	75	125	3.3	14.7	55	100	150	4.3
Turkey Creek High						3.3	70	125	175	5.4					
Turkey Creek Low	9.8	50	75	125	3.3	8.3	50	75	125	3.3	19.4	65	125	175	5.4
Low Point 5[2]						12.8	50	75	125	3.3					
High Point 5[2]						11.6	50	75	125	3.3	18.8	60	100	150	4.3
Bethlehem Avenue						9.6	55	100	150	4.3	14.2	50	75	125	3.3
Low Point 7[2]						10.1	70	125	175	5.4	15.7	80	125	200	5.4

NOTES: 1) Analyses not carried out for intermediate positions or depths.
2) Different high and low points provided where different options pass over Turkey Creek (either in box culvert or siphon) or beneath Turkey Creek.
3) Undrained shear strength based on average within depth of foundation influence.

Within Table 8.2.4 it is shown that the maximum wall height is less than the potential cut depth, suggesting that gravity walls supported on shallow spread foundations or mechanically stabilized earth walls are not suitable for the entire cut depth (indicated by the shaded table entries). Conceptual and preliminary design of retaining structures for other transition ramps or roadway facilities may use the heights provided in Table 8.2.4 as guidelines for selecting suitable wall types. In cases for which the cut depth exceeds the maximum values provided in Table 8.2.4, it may be necessary to either construct the wall backfill of lightweight aggregate materials, reduce the wall height by implementing a terraced grading plan (subject to slope stability analyses), or to use retaining structures supported by deep foundations. Overturning moments and the resulting foundation contact stresses may exceed the bearing capacities as noted above and may also further limit the height of gravity wall systems in these areas. Terraced grading plans developed to permit use of gravity retaining structures may require as much space perpendicular to the road alignment as sloped cuts as discussed in subsequent sections of this report.

Although retaining wall total and differential settlements may not be of significant concern, as the cuts will result in a net unloading condition, induced bearing pressures in excess of the Serviceability Limit States (SLS) values provided above may result in long-term creep displacements of the walls. Such long-term creep displacements may affect the alignment of the face (aesthetic concern) and potentially influence drainage or pavement features constructed near the base of the walls.

For below-grade roadway sections, where retaining walls will be built against cut slopes, cast-in-place concrete cantilever walls and MSE walls are often the most economical types of retaining systems with crib and bin walls being the next most expensive. Some crib and bin walls, and MSE walls, however, are not suited for support by deep foundations. For cantilever or counterfort walls, pre-cast sections can be used to speed up construction.

As the depth of excavation extends beyond the maximum heights listed above and into the softer cohesive deposits, flatter temporary side slopes or use of temporary shoring such as soldier piles and lagging, soil nail walls, or sheet piling may be required in order to maintain the stability of the excavation sidewalls and to restrict movement of the surrounding soils. Deformations are expected to become significant for cuts in which the base heave stability factor of safety is less than about 2 (see Table 8.2.2). Displacement of the surrounding ground must be examined in detail during final design since maintaining displacements of adjacent buildings or utilities within acceptable limits may require underpinning or alternative excavation support systems (also see Section 8.2.8 of this report). Based on the available information, groundwater encountered in the shallow surficial granular deposits should be able to be controlled using sump pits and pumps, though groundwater control will need to be examined in more detail near the existing drains and watercourses. The need for temporary shoring and deep foundations increases construction complexity and costs for gravity walls constructed in excavations deeper than the maximum heights provided in Table 8.2.4. For these reasons, in situ retaining systems are preferred for construction of below-grade roadway sections in cuts of greater depth.

8.2.4 In Situ Walls

An advantage of in situ walls is that the road section can be constructed using a vertical cut, resulting in a reduction of earthwork volumes and the required working space compared to gravity walls built within temporary sloped cuts.

8.2.4.1 Soil Nail Walls

Temporary or permanent soil nail walls may be feasible for construction of below-grade roadways in open excavations or covered cuts, depending on the local soil strength and depth of cut. Soil nail walls are best used in stiff to hard cohesive deposits. Due to the potential for basal instability caused by the softer underlying deposits, construction of permanent soil nail walls should be limited to the maximum heights for gravity walls as listed in Table. 8.2.1. It is recommended that permanent soil nail walls be excluded from consideration for cuts west and north of Huron Church Line as the thickness of the stiff to hard silty clay “crust” diminishes at these locations. A subsurface easement extending horizontally a minimum of 0.6 to 1 times the height of the wall is required for installation of the soil nails because of the anticipated soft to firm clays. Where the excavation includes granular soils near the surface, special construction provisions may be required to maintain a stable cut face during installation of the nails and facing materials. In some cases, vertical facing elements or ground reinforcement can be installed in situ in the granular soils prior to construction of the primary soil nail wall system.

8.2.4.2 Sheet Pile Walls

Driven sheet piles should be suitable for temporary support of excavations where below-grade roadway sections are to be constructed. Sheeting of the "Z" shape or interlocking pipe and sheet sections may be best for resisting large bending moments associated with deeper excavations and larger spans between supports. A subsurface easement of approximately 1.5 to 2 times the height of the excavation will be required if dead-men or tie-backs are used. Internal bracing can be used instead of anchors if subsurface easements cannot be obtained or if anchor lengths (extending into bedrock) become prohibitively costly or challenging as may be the case for the DRIC project. Where use of driven sheet piles or interlocking pipe piles is contemplated in close proximity to structures, a specific assessment of potential vibrations and settlement should be carried out in conjunction with an assessment of the structure’s tolerance for vibrations and movements. If driven piles are used adjacent to structures or utilities, it may be advisable to leave them driven into the ground rather than attempting to extract the piles as such extraction in silty clay soils can significantly disturb the area as the soils tend to adhere to the piles as they are extracted. Sheet pile structures for excavations equal to or exceeding the planned depths for this project are not uncommon. Limitations of total pile length and wall height will depend primarily on fabrication (field welding of long sections), the pile structural section properties, vertical spacing of horizontal supports and the factor of safety for base heave stability. Cantilever sheet pile walls using conventional sheeting sections may be limited to less than about 4 m in height, and cantilever heights for interlocking pipe pile walls of 6 m to 8 m may be achievable, depending on displacement criteria, total tip penetration depths, and local conditions; however, the use of cantilever walls must be examined in greater detail during subsequent stages of design before selecting such a system.

8.2.4.3 Soldier Pile and Lagging Walls

Soldier pile and timber lagging shoring systems are commonly used in southern Ontario. Soldier piles and lagging walls are most economical for excavations that extend below the crust to depths of 5 m to 7 m, where the risk of damage to settlement-prone structures or utilities is low. For deeper excavations in soft ground, concrete lagging or shotcrete can be used between the soldier piles to increase the wall stiffness between the piles. Since only the soldier piles will be embedded beneath the base of the excavation, use of soldier piles and lagging is not recommended for excavations deeper than 8 m to 10 m because of the potential for basal heave instability and relatively large ground displacements. Cantilever soldier-pile and lagging systems will likely be limited in height to about 4 m or less depending on displacement criteria. The local applicability of soldier pile excavation support will depend on the pile spacing, diameter or face width of the pile, vertical spacing of horizontal supports, and base heave stability factor of safety.

8.2.4.4 Secant Pile Walls

Secant or tangent (caisson) walls are suitable for deeper excavations and can be used in both open and covered cuts. Carefully constructed caisson walls can form an adequate barrier to inflow of groundwater, though some localised seepage through the wall should also be expected. Since they can be readily extended beneath the base of the excavation and are relatively stiff in bending, they can be used to reduce the deformations which can occur in deep excavations that penetrate softer cohesive deposits. In addition, the stiff section modulus of secant or tangent pile walls can also permit greater vertical spacing between horizontal wall supports. These walls are, however, more expensive to construct than sheet pile or soldier pile and lagging walls. Secant or tangent pile structures for excavations equal to or exceeding the planned depths for this project are common. Limitations of total pile length and wall height will depend primarily on fabrication (steel reinforcement), the composite pile structural section properties, vertical spacing of horizontal supports and the factor of safety for base heave stability. For this project, it is anticipated that secant pile walls may be considered for the following purposes:

- To maintain a relatively dry excavation in the areas of drain crossings, particularly the Grand Marais (Turkey Creek) Drain;

- To support areas very close (within 1.0 times the depth of excavation) to existing settlement-sensitive structures or utilities;
- To assist with maintaining base stability in soft ground;
- To support decking so that traffic can be carried above the excavation, with the decking and traffic loads carried to bedrock (if needed);
- To provide permanent structural walls, in top-down construction between which a roof slab would be constructed with backfill, pavement, and traffic loads transferred to the walls and down to bedrock; and
- To construct relatively high permanent cantilever walls that would have no horizontal supports.

Cantilever heights for secant pile walls of 6 m to 8 m may be achievable in selected areas, depending on displacement criteria, total tip penetration depths, and local conditions; however, the use of cantilever walls must be examined in greater detail during subsequent stages of design prior to selection of such systems.

8.2.4.5 Soil-Cement Mix Walls

Soil mix walls can be used for support of open excavations and covered cut sections built using bottom-up construction. However, for this project, soil mix walls may be comparatively expensive to construct and internal bracing may still be required. Achieving adequate mixing and consistent strength of the soil-cement mix may also be difficult for the anticipated subsurface conditions. It is anticipated that the depth of the relatively soft soils, the base heave factors of safety, and the extent of ground beyond the edges of the excavation required for construction of soil mix walls may all inhibit the use of this wall system.

8.2.4.6 Cast-In-Place Concrete Diaphragm Walls

Slurry or diaphragm walls should be suitable for construction of support of excavation walls in both open-and-cut and cover excavations. Diaphragm wall structures for excavations equal to or exceeding the planned depths for this project are relatively common for large underground construction projects, though they are a type not commonly constructed in southern Ontario. Limitations of total diaphragm panel wall depth and wall height will depend primarily on fabrication (welding of reinforcing steel), the pile structural section properties, vertical spacing of horizontal supports and the factor of safety for base heave stability. If slurry walls are to be incorporated in top-down tunnel construction where the excavation support wall is to sustain roof loads, the structural diaphragm walls must extend to bedrock, either as a continuous wall or with selected panels carrying the vertical loads to bedrock. Specialty equipment may be required for construction of diaphragm walls greater than 30 m in depth. There must be sufficient workroom for both the equipment and storage of both the slurry and spoil material. As with secant pile walls, slurry walls would be best used in areas of soft ground where the depth of excavation is greater than 8 m, a continuous groundwater cut-off is required, base stability enhancement is required, and/or settlement-sensitive structures are located within a distance less than the excavation depth.

8.2.5 Earth and Groundwater Pressures

Earth pressures for the design of gravity walls will likely be governed by the composition of the wall backfill materials. In general, it is recommended that all gravity walls be backfilled with granular soils such that design earth pressures are of typical magnitudes, where the active earth pressure coefficient may range between about 0.25 and 0.30, and groundwater can be drained from the backfill. Should the cut slopes (native soil and backfill interface) made for gravity walls be steeper than about 55^o from horizontal (from the base of the wall to the ground surface), the active earth pressure coefficient may range between about 0.33 and 0.40 due to the loading imposed by the native site soils. Compaction pressures will dominate design conditions near the top of any gravity or cast-in-place cantilever walls.

Earth pressures for the design of in situ walls will be governed by the existing soil and groundwater conditions, and the strength of the in situ silty clay soils. For conceptual and preliminary design purposes, it may be assumed that the walls will be sufficiently stiff for displacement and stability control that an active earth pressure distribution will result. Conventional trapezoidal earth pressure distributions may not be suitable for design of temporary shoring on this project since the factor of safety for base heave and the requirements for displacement control may govern the design. Trapezoidal earth pressure diagrams (apparent earth pressure diagrams) are appropriate only for very flexible wall systems (e.g. Boone and Westland 2005). The active earth pressure may be determined using an effective stress earth pressure coefficient, K_a, value of about 0.33, reflecting the likely length of time the excavations will be open for construction and the consequent changes in pore-water pressure behaviour that will result. Groundwater pressures, as described below, must be added to the earth pressures, as must appropriate allowances for surcharge loads. The passive earth pressure may be determined using an effective stress passive earth pressure coefficient, K_p, of 3.0. The saturated unit weight of the soil, γ, may be taken as 21 kilonewtons per cubic metre (kN/m^3). and the unit weight of water, γ_w, as 9.81 kN/m^3. It is expected that the stiff to hard soils found below the CPT refusal depths may offer substantially greater resistance to displacements of embedded walls. The degree of additional resistance, however, will be highly dependent on the thickness of such soils and their local characteristics that must be defined based on further explorations and testing during final design. It should be noted that along the ACA corridor, groundwater pressures within the excavation should not be taken as hydrostatic from the base of the excavation as there may be either an upward or downward hydraulic gradient. Although the hydraulic gradient may not be observable in the field as the soils are of low permeability, the porewater pressures, u, and their influence on passive restraint must be taken into account. Figure 17 provides a diagram that may be used as a guide to earth and water pressure calculations for preliminary retaining structure design. Note that within Figure 17 there is no consideration for surcharge loads induced by traffic, earth berms, or neighbouring structures. Such surcharge loads must be considered during final design but will depend on the surcharge pressure magnitude, variation, and location with respect to the wall. For conceptual and preliminary design, the surcharge loads induced by traffic on roadways immediately adjacent to the wall (e.g. service roads) can be considered as an equivalent additional height of soil (at the above unit weight) for the wall to support.

8.2.6 Frost Protection

In situ walls will be subjected to freezing ambient temperatures at the wall face during winter. The walls will also be in direct contact with the ground behind the wall without the benefit of free-draining backfill. It is anticipated that the wall materials will serve as a thermal conductor and unless insulation is provided at the wall face, the freezing temperatures may cause ice lenses that will induce frost pressures behind the wall because all native soils behind the walls are considered to be frost-susceptible. It should be noted that there are a number of documented cases of in situ wall distress due to pressures induced by frozen ground (e.g., Broms and Stille 1976, Eigenbrod and Burak 1992). The design and construction of such walls will require that consideration be given to providing the face of the wall with insulation and a protective wall facing. This is consistent with other grade separation projects using permanent in situ walls in Ontario (Golder files) that have been fitted with an insulation layer to prevent such pressures.

8.2.7 Cut Slopes

The stability of cut slopes is dependent upon a number of factors including the local soil type, shear strength, static groundwater level, cut depth, slope angle and the length of time the excavation remains open. A review of slopes cut into similar soils in Welland and Sarnia, Ontario, and Port Huron, Michigan, suggests that excavations with depths of between 15 m and 18 m with side slopes of between 1.5:1 (horizontal:vertical) to 2.5:1 have failed repeatedly (e.g. Conlon et al. 1971, Lo 1971, Dittrich et al. 1997). Final cut slopes in Welland for the canal underpass tunnel cuts, where the depth of cut was on the order of 24 m, required permanent groundwater lowering and side slopes ranging from about 3:1 near the ends of the approach roadways where the cuts were the most shallow, to about 8:1 at the deepest parts of the cut. Stable slopes were achieved in Sarnia with overall slopes of about 3.5:1, though these included 3:1 slopes of limited height with intermediate benches. It is further understood that in Detroit, where the soils may be of somewhat greater strength, cut slopes along the highways are initially cut at 2:1 but continued maintenance is required and some flattening of slopes or buttressing of the slope toes has occurred such that finished cut slopes closer to 2.5:1 are achieved.

A series of preliminary slope stability analyses were conducted to ascertain the long-term stability of cut slopes along the corridor assuming a bulk unit weight of 21 kN/m^3, effective angles of internal friction ranging from 26º to 30º, effective cohesion intercept values ranging from 0 kPa to 8 kPa, and that a long-term factor of safety of 1.3:1.5 would be suitable for final design of cut slopes. Maintaining this long-term factor of safety is predicated on adequate slope drainage where the phreatic water surface level is maintained at least 1 m below the ground surface at all locations. Achieving this condition may require that the slope include a flat 2 m to 3 m wide "bench" at the approximate mid-height of the slope, with a subsurface drain placed along the up-hill edge of the bench. It is understood that earth berms on the order of 2 m to 3 m in height with 3:1 side slopes may be constructed near the crest of the cut slopes to serve as noise barriers. In addition to the parametric variations described above, the analysis included various positions of these berms in relation to the slope crest. The recommended values for cut side slopes and limiting depths provided below also consider that the area being cut, neither undergoes significant slope displacement during construction of the cut nor has experienced slope instability in the past. Final design should include detailed analyses of each area to be constructed with cut slopes to refine this conceptual and preliminary design guidance.

Between Highway 401 and near Todd Lane, the cut depths should be limited to a maximum of 7 m for 2.5:1 slopes, 8 m for 3:1 slopes, and 10 m for 4:1 slopes, all of which will require an intermediate bench and drain as described above for controlling seepage. Permanent cut slopes deeper than about 10 m should not be considered for this area at this conceptual/preliminary engineering stage.

From near Todd Lane to near the intersection of E.C. Row Expressway and Huron Church Road, cut depths should be limited to a maximum of 7 m for side slopes of 3:1. Any cuts deeper than this would require a side slope of 4:1, and possibly flatter, depending on the location, and would require at least one intermediate bench of about 2 m width. Cut depths should be limited to a maximum of about 8 m for these conditions for conceptual and preliminary design. For those areas between Huron Church Road and Ojibway Parkway (parallel to E.C. Row Expressway), the maximum cut depths that should be considered for conceptual and preliminary design diminish such that by Malden Road the maximum cut depth should be less than about 4 m to 5 m, at this conceptual/preliminary engineering stage.

For conceptual and preliminary design, it should be considered that constructing the berms will be equivalent to increasing the cut depth by an amount equivalent to the berm height if they are placed at the slope crest. This effect diminishes with distance such that if the berm toe is three times the cut depth away from the cut slope crest, the effect of the berm will be equivalent to increasing the cut depth by half the berm height, and at a distance of five times the cut depth, the effect will be negligible. The effect of the berm height must be considered as part of the maximum cut depths as noted above.

Drainage and storm water control will be critical in order to maintain the surface integrity of the slopes and ditches may be required near the tops of cuts to redirect surface runoff away from the slope faces. To reduce surface water erosion on the cut slopes placement of topsoil and seeding or pegged sod is recommended. Use of erosion control blankets is also recommended and considered prudent on cut slopes to protect against erosion until the vegetation has been established.

As noted for the Welland and Sarnia cases, stable cut slopes deeper than discussed above have been made through similar soils. Although the Welland cut depths were improved by using permanent groundwater lowering, this technique is not recommended for the DRIC project. It may, however, be possible to achieve deeper permanent cuts or steeper cut slopes for this project than the recommended limits provided above through use of horizontal drainage systems, a combination of slopes, diaphragm walls and cross-walls, or other ground improvement techniques. The potential for achieving greater cut depths would have to be evaluated as part of detailed design when more site-specific data, particularly measurements of effective shear strength parameters, are available.

8.2.8 Displacements Associated With Deep Excavations

Construction of excavations, with the sides either sloped or supported by vertical retaining structures, will cause displacement of the ground to differing degrees. When sloped excavations are made as either permanent cut slopes or for construction of backfilled gravity walls, induced displacements will generally be minimal in magnitude and limited to affecting the ground within a distance back from the top of the cut (slope crest) equal to the depth of the slope cut, provided that the factor of safety for slope stability is satisfactory.

Construction of excavations supported by vertical in situ walls induces localized displacements of the adjacent ground (e.g. Peck 1969, Goldberg et al. 1976, Maria and Clough 1979, Clough and O’Rourke 1990, Boone and Westland 2006). The magnitude and pattern of such displacements varies and depends on factors such as:

- type and structural stiffness of the wall system installed;
- ground conditions (strength and deformation properties);
- depth of cut;
- depth of penetration of the wall below the base of the excavation;
- type, number, and spacing of horizontal supports (tie-backs or struts);
- degree of pre-stressing of the horizontal supports;
- whether or not the horizontal supports are removed during construction; and
- ground conditions (strength and deformation properties).

For cuts typically ranging between 10 m and 12 m in depth, with two to three levels of strut supports below deck beams (if any), without support pre-stressing, and a depth of penetration on the order of 50 percent to 80 percent of the cut depth, historical data suggest that maximum horizontal and vertical displacements of the ground adjacent to the wall could be characterized as:

- on the order of 1 percent of the cut depth for wall systems such as soldier-piles and lagging or sheet piles in soft to firm cohesive soils; and
- on the order of 0.5 percent of the cut depth for wall systems such as contiguous drilled pile or concrete diaphragm walls in similar soils.

Figure 18 illustrates a number of factors that influence the displacement of deep excavations that may be used to estimate performance for different construction scenarios. The initial maximum lateral displacement, δ^*_{hmax}, can be estimated using the upper left graph on Figure 17, based only on knowledge of the relative wall stiffness, S_r (as defined below), and the base heave factor of safety (FS). Having derived the initial displacement estimate, a variety of factors are used to modify this estimate, including:

- stiffness of the soil in unloading and reloading (α_M);
- influence of preloading of the horizontal supports (α_{PL});
- influence of the construction stage, e.g. whether or not the struts or tie-backs are removed, (α_{CS});
- depth to a hard layer (α_D);
- width of the excavation (α_B); and
- stiffness of the struts or horizontal support systems (α_S).

The non-dimensional relative retaining system stiffness, S_r is defined as:

$$S_r = EI/(\gamma h^4)$$

where,

E = modulus of elasticity of the vertical wall structural section;

I = internal moment of intertia for vertical walls structural section distributed on a per unit of wall length;

γ = average wet or saturated unit weight of soil; and

h = average vertical spacing between supports (struts or tie-backs).

A preliminary probabilistic analysis of performance was also completed in which the site-specific soil strength and case history excavation performance data were used to quantify the potential variability in expected settlement. Frequency distribution histograms of soil strength are presented on Figure 19. Using the relationships illustrated on Figure 18, the anticipated variability in soil properties (Figure 19), anticipated variability in potential shoring design stiffness, construction workmanship, and well-known methods of probabilistic simulation (e.g. Monte Carlo simulation), 1,000 trials of excavation support were simulated for a 14 m deep excavation at three locations including near Highway 401 and Highway 3 (vicinity of Howard Avenue), St. Clair College, and near Turkey Creek/Grand Marais Drain. These simulations considered that design stiffness values varied uniformly between S_r = 10 to 20, 50 to 100, and 250 to 500 for soldier pile and lagging (or sheet piles), tangent/secant piles, and diaphragm walls, respectively.

It was further assumed that support pre-stressing would be completed with field performance varying (as a normal distribution) around a target value of 50 percent of the design load and that the extent of the settlement would extend back from the shoring about 1.5 to 2 times the depth of the excavation. Consideration was given to implementation of stability enhancement or settlement control construction in the probabilistic estimates of performance such that if soil strength variably resulted in iterations that exhibited very low factors of safety, the estimated displacements reflected a minimum factor of safety of condition. The results of these evaluations are illustrated on Figures 19 and 20 and are summarized in the table below. Settlements associated with excavations of about 10 m depth should exhibit settlements of approximately half the magnitudes identified in Table 8.2.6 and on Figures 20 and 21.

The estimates suggest that, given the conditions described above for this project, average surface settlement for a 14 m deep excavation, at or near the wall line, may be between 80 mm and 145 mm for soldier-pile and lagging walls. For concrete diaphragm walls, the average surface settlement at or near the wall for a similar excavation may be between 25 mm and 45 mm. Consistent with other experience worldwide (e.g. Long 2000, Boone 2002, Moorman 2004, Boone 2005), there is also an estimated probability of about 5 to 10 percent that excessive settlements will occur, where excessive settlement is defined as double or more the typical values of 1%H and 0.5%H described above for soldier-piles and lagging or sheet piles and concrete diaphragm walls, respectively. It should be noted that these settlement estimates exclude settlement from consolidation of the adjacent ground arising from loss of porewater pressure. Such consolidation settlement may be similar in magnitude to the values provided in Table 8.2.6 and will be additive to the estimated settlement. Consolidation settlements that may occur may be countered by mitigation of seepage through walls, porewater pressure maintenance systems (e.g. Westland et al. 1999), or both.

TABLE 8.2.6 SUMMARY OF PROBABILISTIC ESTIMATES OF MAXIMUM SETTLEMENT ADJACENT TO 14 M DEEP EXCAVATION

PERCENTILE	SETTLEMENT					
	Highway 401/ Highway 3		St. Clair College		Turkey Creek/ Grand Marais Drain	
	(mm)	(%H)	(mm)	(%H)	(mm)	(%H)
	Soldier Pile and Lagging Walls					
20th	65	0.5	75	0.5	105	0.8
50th	80	0.6	105	0.8	145	1.0
80th	105	0.8	160	1.1	205	1.5
98th	230	1.6	310	2.2	320	2.3
	Secant and Tangent Pile Walls					
20th	40	0.3	45	0.3	60	0.4
50th	50	0.4	65	0.5	85	0.6
80th	60	0.4	100	0.7	125	0.9
98th	140	1.0	220	1.6	250	1.8
	Concrete Diaphragm Walls					
20th	20	0.1	25	0.2	30	0.2
50th	25	0.2	35	0.3	45	0.3
80th	35	0.3	55	0.4	70	0.5
98th	90	0.6	150	1.1	170	1.2

A number of measures are available to limit the displacements of such retaining structures including pre-stressing of horizontal supports, stiffening of the vertical wall systems, extending the depth of wall penetration, providing buried struts prior to excavation, or improving the ground at the base of the cut using a variety of grouting or soil mixing techniques (e.g. Shirlaw 2006). Displacements and their effects on nearby facilities should be evaluated in greater detail during further stages of analysis and design (e.g. Boone et al 1998, Boone 2001, ITIG 2006). Implementation of stability enhancement or settlement control works should reduce the settlements estimated above by about one-half. The preliminary evaluation above is intended to facilitate refinement of conceptual alternatives and should be updated as additional project and subsurface information is developed.

8.2.9 Stability and Displacement Control Measures

Due to the relative depth of the excavations as compared to the strength of the soil and groundwater pressures, additional construction measures may be required to permit construction, control displacements, or both. In some cases, it may be sufficient to extend the retaining structures below the base of the excavation such that the walls are embedded in the stiff to hard silty clay, very dense silty sand and gravel, or bedrock well below the excavation base level. In some cases where the base stability factors of safety are less than about 1.3, however, it is anticipated that extending sections of vertical walls beneath the excavation base may not provide sufficient bending stiffness to control stability or displacement, even if the walls fully penetrate to bedrock. In these cases, support between the walls may need to be provided prior to excavating to the base level. Such support can be provided using:

- “buried” struts or braces constructed using diaphragm wall techniques;
- a base slab constructed by placing concrete using tremie methods within a water-filled excavation;
- a base slab constructed using jet-grouting methods; and/or
- reinforcement/replacement of soils at and below the excavation base level using lime columns or soil mixing.

In addition to improving stability related to base heave (soil strength), resistance to groundwater uplift pressure may be required should excavations deeper than those indicated in Table 8.2.3 be required (e.g. bored tunnel portals). If groundwater pressures cannot be lowered, either as a result of potentially large or unmanageable flows, contaminated groundwater, or the adverse effects that changing of groundwater pressures may have on the surroundings, resistance to groundwater uplift forces may be provided by ground anchors (using steel tendons or strands) or tension piles. Both these options require the use of a base slab as well as techniques that permit construction either below and within the ground (e.g. a jet-grouted slab) or below water (e.g. a tremie-concrete slab) prior to draining the excavation and carrying the excavation to its full planned depth. Typically, use of ground anchors with strand tendons is suitable when permanent uplift resistance is required and groundwater pressures can be otherwise controlled for the temporary condition. Achieving adequate structural connection between base slabs and relatively small diameter and flexible tendons or strands can be difficult if not impracticable if working in a sub-aqueous environment or if ground improvement techniques are used to form the base slab. In the case of the DRIC approach project, it is considered that bored tension piles are the more technically and economically suitable approach to providing temporary uplift resistance where required. The bored tension piles may be constructed similarly to mini-piles in which a relatively small diameter (diameter < 200 mm) steel permanent casing is installed during drilling with its base socketed into the bedrock or resisting stratum, steel strands or tendons are installed within the casing to provide load transfer from the base to the casing, and the base and interior are pressure-grouted to form a small diameter pile that functions as an anchorage element. The base slab is then cast around the permanent casing (and any flange plates) and becomes structurally connected to the tension pile.

The techniques described above have been used in many locations around the world (e.g. van Beek et al. 2003, Shirlaw 2006) including the St. Clair River tunnel crossing (Dittrich 2000) where diaphragm cross-walls were used as below-grade braces (see Figure 22). The relative advantages and disadvantages of each of the methods for base stability enhancement will depend on factors such as the following:

- *Retaining Wall Construction Techniques* – For example, it may be more effective to use buried diaphragm cross-walls if extensive diaphragm retaining wall construction is to be used for temporary or permanent cut-and-cover tunnel walls. Alternatively, if secant piles are to be used for the majority of support walls, it may be more cost-effective to use jet-grouting or soil mixing to form a below-grade base slab or cross-supports.

- *Access Limitations* – Diaphragm walls typically require full and direct vertical access to the depths and locations at which wall or other structural sections are to be constructed. Utility conflicts, traffic management, or the proximity of other facilities may obstruct the use of diaphragm wall techniques, in which case jet grouting may be a more suitable option.
- *Cost* – Each stability enhancement option must be evaluated with respect to the above issues and relative cost as the cumulative extent of the work will have a significant influence on unit costs.

Figures 14 through 16 illustrate sections along the ACA for which stability improvement measures may be required for the different conceptual design options. These figures, however, do not take into account localized areas that may require similar work to limit displacement to values tolerable for adjacent structures. Such areas should be defined during final design when more accurate locations of the various retaining structures and their proximity to settlement-sensitive facilities can be refined.

8.3 Highway Embankments

The conceptual design indicates that new embankments will be required to carry the highway over Malden Road, Machette Road, Ojibway Parkway, and the Essex Terminal Railway (ETR). The embankment leading to the crossing of the Ojibway Parkway and ETR peaks approximately 175 m east of Ojibway Parkway. Embankment heights for conceptual design options are listed in Table 8.3.1, below. Although embankments have not been indicated in other areas according to the conceptual design at the time of this report preparation, discussions regarding embankments in other areas of the ACA are also provided. At the time of this report preparation, there was discussion regarding the need for embankments for potential cross-street overpasses.

TABLE 8.3.1 LOCATIONS AND HEIGHTS OF CONCEPTUAL DESIGN EMBANKMENTS

EMBANKMENT LOCATION	CONCEPTUAL PEAK EMBANKMENT HEIGHT (m)
Malden Road	6 to 7
Machette Road	7 to 8
Ojibway Parkway and ETR	12 to 13

The design and construction of embankments, and other features where relatively thick fills may be required, will be governed by the strength and consolidation properties of the soft to stiff silty clay underlying the project construction sites. The influences of the soft soil conditions and methods to address the influences are described in the report sections below.

8.3.1 Stability

In general, design of embankments on soft soils is completed using strength parameters that are based on averages of the interpreted field or laboratory test results for specific sites. This approach is used recognising that one particularly low value does not represent the strength of the ground mass supporting an embankment. The stability of the overall embankment is judged using a factor of safety defined as the ratio of the available soil strength to the gravitational and groundwater forces acting to destabilise the embankment along a potential failure surface. Typically, a minimum factor of safety of about 1.3 is applied to account for a variety of conditions that may be unknown (untested areas, potential test errors, variation in material properties, etc.). Design calculations for slope stability were completed using the computer software GeoStudio™ (Geoslope International, 2004). Use of this software allowed multiple searches for the most critical potential failure surfaces within the embankment and foundation soils. Recommendations for the conceptual and preliminary design of embankments are provided below.

Figures 23 and 24 illustrate the general relationship between embankment height, side slope, and factor of safety against instability for the conditions during/immediately-following fill placement and for the long-term conditions, respectively. For conceptual and preliminary design of the permanent embankments, it is recommended that a factor of safety of 1.3 should be used as the basis for selection of embankment height and side slope limitations, based on peak soil strengths and conventional analyses as used on other projects. These figures are based on the following assumptions:

- Native soils excavated from other areas or imported earth fills are used for the majority of the embankment fills (see Section 8.11).
- The soil stratigraphy and design parameter profile of Borehole BH-23 is representative of the highest and most extensive embankment areas (parallel to E.C. Row Expressway).
- All organic materials and softened or disturbed materials are removed from the existing ground surface and a 0.5 m thick sand drainage layer (see Section 8.3.2.1) is placed on the prepared grade prior to embankment construction.

Construction and performance of the embankments depends largely on whether the load induced by the filling exceeds the “preconsolidation pressure” of the soil. In areas where the embankments are less than about 3 m to 4 m in height the fill load and in situ stresses will likely be less than the preconsolidation pressure. In areas where the embankments are greater than about 4 m in height, the embankment loads may exceed the preconsolidation pressures, particularly in the areas near and between Turkey Creek, Ojibway Parkway, and the Detroit River shoreline. Where embankment loads exceed the preconsolidation pressures, the strength of the soil will be less than the peak strength measured in the field and may approach the “post-peak” conditions evidenced by the Nilcon field vane shear tests (see Figures 5 through 9). In these areas, Figure 23 indicates that a 5 m embankment height will approach a factor of safety of 1.1 for the post-peak soil strength values.

Figure 23 illustrates the limiting conditions for stability during embankment construction and clearly illustrates a factor of safety of 1.1 or less, based on peak strengths, if an 8 m or higher embankment height was to be constructed in a single rapid stage. This figure also indicates that adequate factors of safety cannot be achieved for embankments greater than 9 m in height if they were to be constructed in a single stage. It is considered feasible, however, to construct a 5 m high embankment in one stage maintaining a factor of safety of greater than 1.1 for the post-peak conditions (see Tavenas and Leroueil 1980, Lefebvre et al 1987) and 1.3 for the conventional peak strength analyses. It is noted that for the immediate construction conditions that the side-slope of the embankment has only a small influence on temporary stability within the range of side slopes considered.

Given the nature of the soils at this site, it is recommended that embankments be planned to be constructed in stages with the first construction stage involving placement of no more than 5 m of fill. The preconsolidation pressure may be exceeded in many areas and the field behaviour should provide indications of both stability and settlement behaviour that could be anticipated for greater embankment heights. Though a higher first stage may be possible without inducing failure, it is inadvisable to construct a higher embankment until field instrumentation (see Section 8.12) indicates that the field performance is consistent with design expectations. The time period of consolidation between stages will depend on whether or not wick drains and surcharging are used to accelerate consolidation (as discussed in Section 8.3.2). Figure 25 illustrates the results of preliminary stability analyses should staged construction be implemented. In this case, as consolidation occurs between stages, the strength of the ground increases and higher embankments may be constructed. Side slope has a more significant influence in this scenario and Figure 25 indicates that to achieve the conceptual maximum embankment height of 13 m, the embankment should be constructed with maximum side slopes of 3 horizontal to 1 vertical. The factor of safety for this condition is also marginal, being about 1.1 considering strength gain from consolidation under the load from the preceding stage. During consolidation, it is anticipated that the factor of safety will improve, transitioning from about 1.1 to about 2 between the immediate and long-term conditions, respectively.

Should right-of way space be available, it may also be possible to use temporary or permanent toe berms to assist with achieving planned embankment heights while maintaining stability. Toe berms consist of additional fill materials, placed at the toe of the embankment such that a berm is formed with a height on the order of ¼ to ½ the embankment height. The top of the berms are typically flat, extending out (perpendicular from the roadway centreline) from the embankment a distance of between 1 and 2 times the embankment height. For temporary construction conditions, these berms may be removed after completion of consolidation settlement.

Based on these preliminary analyses, it is considered that the planned embankment heights should be feasible, provided that adequate instrumentation and monitoring are implemented during construction. During final design analyses, if it is found that areas of lower soil strength are present, or that detailed analyses or risk mitigation considerations suggest that the factor of safety for the immediate construction condition should be greater, it may be necessary to further consider the use of lightweight fills in limited areas within the highest embankment sections.

8.3.2 Settlement

Calculations related to settlement magnitude and the time rate of settlement were carried out using manual and spreadsheet methods, calibrated to experience gained on other soft-ground embankment projects in Ontario, with particular comparisons made to CPT 24 and measured settlements experienced near Highway 401 and Provincial Road (Golder 2006, GWP 64-00-00).

An earlier study completed for MTO by Golder (referenced above) examined settlements of existing Highway 401 bridge structures in southwest Ontario. A precision survey of bridge structures was completed and the resulting data was used with as-built surveys to determine total and differential settlement. Settlement estimates completed using the laboratory and borehole data for each location were also compared to iterative back-analyses as a means to determine the relationships between laboratory-derived parameters, settlement calculation methods, and field behaviour.

As part of the DRIC study, one cone penetration test (CPT-24) was completed in the vicinity of Structure 6-074 at Highway 401 and Provincial Road (see Figure 1). The CPT location was chosen to best represent conditions that likely existed prior to construction of the nearby structure.

Using the engineering parameter correlations developed for the DRIC study for the CPT, Nilcon field vane, and laboratory data, a settlement estimate was completed for a 7.3 m embankment at Structure 6-074. The analysis indicated that about 85 mm of settlement would be expected for the embankment at this site. At the location of the abutment closest to CPT-24, a total settlement of 88 mm was measured. The abutment further to the west experienced a total average of 125 mm of settlement for an 8 m high embankment. Using the correlations developed for this project, approximately 110 mm of settlement would be calculated for the western abutment. It is therefore considered that the settlements estimated as part of this study are reasonably calibrated to field performance.

Settlement of the embankments will occur over a significant period of time as the native soil consolidates, unless additional construction measures are implemented such as surcharging, installation of vertical drains, or use of lightweight fill. For example, preliminary estimates indicate that the proposed 13 m high embankment near the Ojibway Parkway area may undergo maximum settlements on the order of 650 mm along the embankment centreline (see Figure 26). If the embankment is allowed to settle under its own weight it may take up to three years to complete 90 percent of the settlement, with the remaining 10 percent of the total settlement taking an additional three to eight years. By installing vertical drains ("wick drains") beneath the embankment, the time-rate of settlement could be accelerated such that 90 percent of the settlement could be completed within one to two months as shown on Figure 26. The remaining settlement may then be completed over the next two to three months. In addition, pre-loading (or surcharging) could assist in minimising the total time required to achieve the anticipated maximum settlement beneath a given final embankment height, or lightweight fill may be used to create embankments that impose less stress than conventional earth fills. Discussion regarding the configuration of vertical drains, surcharges, and lightweight fill are provided in subsequent sections of this report. The choice of using vertical drains, surcharging, or lightweight fill will depend on the planned construction schedule for both embankments and bridge structures and relative costs and should be evaluated in detail during final design. In general, vertical drains are much less costly than lightweight fill for an equivalent improvement in construction schedule. Use of surcharge loads may be less costly than vertical drains, depending on other earthwork requirements, but will not improve the time-rate of settlement as much (see Figure 26).

Settlement estimates prepared for other sections of the ACA are presented in Table 8.3.2 below. Settlement beneath highway embankments similar to those for the proposed project typically exhibit about 20 percent of the maximum settlement near the toes of the embankments. The embankment will also influence the ground beyond the embankment toe to a distance (measured perpendicular from the embankment centreline) on the order of two to three times the embankment height to where little or no settlement is experienced. As noted in the example above, the embankments will all undergo time-dependent settlement. For the lowest height embankments, particularly near the intersections of Highways 3 and 401, much of the settlement will be "recompression" settlement, or settlement due to loading less than the vertical one-dimensional yield stress ("preconsolidation pressure"). Such recompression settlement will occur as the load is placed. However, for embankments of between 3 m and 5 m in height in all areas north and west of Todd Lane, only about 20 percent of the total settlement will be related to recompression. The remainder of the settlement will be time-dependent and final design must consider the time-rate of settlement for each specific embankment case.

TABLE 8.3.2 CENTRELINE EMBANKMENT SETTLEMENT FOR VARIOUS EMBANKMENT HEIGHTS

TOTAL EMBANKMENT HEIGHT (m)	ESTIMATED SETTLEMENT (mm)			
	HIGHWAY 401 / HIGHWAY 3	ST. CLAIR COLLEGE	TURKEY CREEK	OJIBWAY PARKWAY
2	<20	70	60	85
3	30	100	100	135
4	40	130	150	190
5	60	170	200	250
6	90	210	260	305
7	125	250	315	365
8	160	300	375	415
9	193	340	430	465
10	225	385	485	515
11	260	425	540	560
12	300	475	590	600
13	300	520	640	640

The embankment settlement estimates prepared as part of this report are preliminary in nature and based upon the borehole findings, laboratory testing and cone penetration tests conducted in the areas noted in the table above. In addition, it has been assumed that the embankments will have a width of about 36 m at their top and side slopes of about 3 horizontal to 1 vertical. For final design, it will be necessary to complete additional investigations, testing and analyses, as discussed in a subsequent section of this report.

New fill embankment materials may also compress during placement depending on the material type and placement methods. Assuming that native clayey silt and silty clay cohesive soils are used (see Section 8.11.1) and compacted at moisture contents slightly dry of their optimum compaction moisture content, compression of these materials may on the order of 1 percent of their original placement thickness. The maximum fill compression settlement will generally occur near the base of the embankment where the loads are greatest, and diminish to nominal values near the top of the embankment. Such compression settlement should occur during embankment placement. Native cohesive soils placed at moisture contents dry of their optimum compaction moisture content may compress more than this amount and over a longer period of time, particularly when subjected to saturation after placement. Additional recommendations related to fill placement and compaction are provided in Section 8.11.1. It is anticipated that this fill compression settlement will be substantially less than the consolidation settlement of the underlying native soils and may be compensated for by placement of additional fill required to meet planned grades.

8.3.2.1 Installation of Vertical Drains

The time-rate of embankment settlement could be accelerated by installation of vertical drains so as to decrease the time between embankment and bridge or pavement construction (see Figure

26). The vertical drains should be installed using a steel mandrill to push prefabricated drains into the soft soils.

When vertical drains are used to accelerate consolidation settlements, a pathway for relief of pore-water pressure must be provided. At this site, there is a limited thickness of native sandy silt near the ground surface in some areas. It is expected that this material will not provide a sufficient natural drainage outlet. It is therefore recommended that the first 0.5 m of embankment fill placed following stripping of native organic materials consist of a granular material free of stones greater than 25 mm in maximum dimension and containing less than about 5 percent passing the U.S. Standard No. 200 Sieve. This material should not consist of crushed stone products so that there are no sharp edges or points to tear or damage prefabricated vertical drains. Provided that the vertical drain installation equipment can operate on top of this layer of granular fill, the vertical drains should be installed immediately following granular drainage layer fill placement and prior to placement of the remaining embankment materials.

For preliminary design purposes, vertical drains should be planned for installation on a regular, equilateral triangular grid with a centre-to-centre drain spacing of 1.5 m for all embankments exceeding about 3 m in height along the potential highway corridor parallel to E.C. Row Expressway. This threshold height should be re-evaluated during final design and will depend upon the combination of wick drains, preloading, surcharging, or use of lightweight fills that may best meet the cost and schedule goals of the project. Furthermore, use of wick drains for embankments in other areas either may not be necessary or the threshold embankment height may be greater depending on the final design concept and project scheduling. Although this spacing is relatively close (spacing typically ranges from about 1.5 m to 3 m) the vertical drains are anticipated to be relatively economical to install, will assist in minimising the construction duration requirements, and are considered suitable for this preliminary assessment. The effectiveness of drains decreases rapidly with relatively small changes in drain spacing and, therefore, more distant spacing is not recommended. In some projects, the vertical drains are extended through the compressible layer. At this site, the presence of artesian water pressures must be accounted for and it is considered undesirable to create multiple vertical hydraulic pathways directly between the underlying sand and rock strata so that the artesian conditions do not create excessive drainage water. Therefore, the vertical drains should penetrate to a depth of approximately two-thirds of the depth of the soft and compressible silty clay layer but no deeper than about Elevation 162 m. It is also recommended that the plan area that is to include vertical drains be defined by a 1:1 line projected from the crest of the outermost embankment slopes (excluding shoulder grading). This configuration places wick drains under the full footprint of the full-height embankment and assists in improving the soils in the critical zones where the embankment slopes meet the existing grade (where differential stresses occur). These recommendations related to vertical drain installation are preliminary in nature and are based upon the borehole, laboratory testing and cone penetration tests conducted in the embankment areas. For final design, it will be necessary to complete additional investigations, testing and analyses, as discussed in a subsequent section of this report. In particular, it will be essential to better define the top of rock profile in the areas underlain by artesian groundwater pressures so that the vertical drain design is compatible with the groundwater conditions.

8.3.2.2 Surcharging

By constructing the embankments to a level higher than the planned final grade, the additional load will induce settlements that should match those for a lower embankment but at an earlier time. By placement of additional fill to “surcharge” the soils, the time required to accomplish a given magnitude of settlement can be reduced. For example, the total settlement that a 3 m embankment may experience is estimated to be about 100 mm to 140 mm, with 90 percent of this settlement completed about three years after completion of filling. By placing a 2 m surcharge (total height of 5 m), this same settlement could be completed in about one-quarter to one-third of the time, or about eight to ten months (see Figure 26). If the height of surcharged embankments are such that stability cannot be maintained for additional fill placement, the embankments can also be constructed to their maximum stable height and, at a later date during construction, cut down to some intermediate height then reconstructed with lightweight fill, achieving the same effect as surcharging.

8.3.2.3 Lightweight Fill

Lightweight fill materials may be used to produce embankments of greater stability that settle less than embankments of comparable heights constructed of natural soil or rock materials. Lightweight fill may consist of materials such as environmentally and physically stable blast-furnace slag, expanded shale products, or expanded polystyrene foam. Each of these materials offers advantages and disadvantages in comparison with each other and natural materials. Slag fill may be the least costly but should be examined for its environmental suitability. Some slag fills may also have undesirable crushing characteristics that cause greater settlement within the embankment structure itself and adversely affect performance as a pavement subgrade. Slag fill also has the smallest ratio of fill height to weight of the lightweight fill materials. Expanded shale products (shale passed through a high temperature kiln) have a better height to weight ratio compared to slag but may not be as available in the Windsor area. Crushing characteristics of expanded shale products should also be examined prior to selecting a particular product to evaluate their performance as a pavement subgrade material. Expanded polystyrene foam (EPS) exhibits the highest fill height to weight ratio but may be the most costly of the lightweight fill materials. Crushing characteristics (strength) of EPS must also be evaluated prior to choosing a particular product. All of these materials should be covered by a layer or layers of natural fill to separate them from landscaping or pavement structures. In addition, EPS must be protected from potential hydrocarbon spills as certain petroleum products or solvents will degrade or destroy EPS. In general, lightweight fills are most often and best used to reduce differential settlements, reduce down-drag loads, and improve transitions between high embankments and bridge structures, particularly where stability concerns and schedule constraints make conventional staged construction of embankments problematic.

8.4 Earthen Noise Barrier Berms

Earthen berms may be constructed as either noise barriers or for landscaping along the ACA. It is anticipated that these berms may be up to 5 m in height and generally triangular in cross-section with slopes on both sides and a narrow crest width, on the order of 1 m to 2 m in width. Specific berm shapes or locations have not been defined within the conceptual design for the approach corridor, though some conceptual options may include such berms. Therefore, general recommendations are provided in this report for use in further development of conceptual alternatives.

8.4.1 Stability

Based on the analyses completed for the roadway embankments discussed in Section 8.3.1, it is also anticipated that construction of earthen berms up to 5 m in height may be completed in a single stage while maintaining stability factors of safety of 1.3 for conventional analyses using peak strength and 1.1 for the lower-bound post-peak strength analysis. It is recommended that side slopes no steeper than 2 horizontal to 1 vertical be assumed during conceptual design. For areas between Turkey Creek and the Detroit River, it is considered prudent that, for planning purposes, side slopes of 3 horizontal to 1 vertical be incorporated into the conceptual and preliminary designs.

During conceptual design, consideration has been given to the use of earthen berms to separate a below-grade roadway from service roads or adjacent lands. It is recommended that any berms placed near open-cut sections be limited in height to 5 m or less and be located such that the toe of the berm slope is at least four times the depth of cut from the crest of any cut slope. Earthen berms should not be placed near the crest of any cut slope with a depth of cut greater than about 5 m. For any earthen berms constructed any closer to cut slope crests than this distance or for any berms placed near cuts of 5 m deep or greater, a detailed stability analysis will be required.

8.4.2 Settlement

Settlement considerations for earthen noise barrier berms will be similar to those described in Section 8.3.2 with respect to the time rate of consolidation, patterns of movement at and beyond the berm toe, and measures to accelerate or control settlement. The magnitudes of settlement, however, will generally be less as the width of the berm top will be much less than the assumed 36 m wide highway embankments. Although for the highest of the potential earthen berms the time-rate of consolidation may be of concern, it is generally anticipated that special construction measures for acceleration of settlement may not be necessary. The need for settlement acceleration measures should be examined in greater detail during final design. For the purposes of this preliminary analysis, it was assumed that noise barrier berms would have a top width equal to about 1 m to 3 m, side slopes of about 2 to 3 horizontal to 1 vertical (approximately triangular in cross-section), and a length of at least five times the berm base width. Table 8.4.1, below, summarizes preliminary settlement estimates for noise barrier berms of differing heights up to 5 m in various locations along the ACA.

TABLE 8.4.1 CENTRELINE SETTLEMENT FOR VARIOUS EARTHEN BERM HEIGHTS

TOTAL EMBANKMENT HEIGHT (m)	ESTIMATED SETTLEMENT (mm)			
	HIGHWAY 401 / HIGHWAY 3	ST. CLAIR COLLEGE	TURKEY CREEK	OJIBWAY PARKWAY
2	<20	75	80	65
3	<20	95	105	95
4	25	115	140	135
5	30	140	180	185

8.5 Deep Foundations for Bridges and Heavily Loaded Structures

The soft to stiff silty clay and clayey silt found in the project area is relatively compressible and makes shallow foundations for heavily-loaded bridge structures inappropriate for this site. The proposed bridge piers and abutments, both for the new Highway 401 and cross-streets, should therefore be supported on deep foundations (similar to existing bridge structures) that transfer the foundation loads to more competent bearing materials at depth. In addition, there may be other structures associated with the new highway corridor, such as plaza buildings, overhead signs, and light masts that may induce foundation loads that cannot be carried by shallow spread foundations.

Two options that could be considered for conceptual and preliminary design for these foundations are as follows:

- steel H-piles driven to end-bearing on bedrock; or
- cast-in-place concrete caissons founded on bedrock.

Recommendations for steel H-piles and cast-in-place concrete caissons are presented in the report sections below. Table 1, following the text of this report, provides a brief overview of the advantages and disadvantages of various foundation types for the DRIC approach corridor project.

8.5.1 Steel Pile Foundations

8.5.1.1 Axial Geotechnical Capacity

Steel H-piles driven to found on the limestone or dolostone bedrock may be used for support of heavily-loaded structures. It is anticipated that the pile cap elevations would be sufficiently below the nearest adjacent grade to be protected from frost (see subsequent report section). At the time of this report, conceptual designs and the loading conditions for such structures had not been completed.

Where necessary, pile foundations may be designed with a maximum batter of 1H:3V on the steel H-piles (for seismic or lateral loading considerations). The potential influence of downdrag loads (discussed below) must, however, be considered prior to use of battered piles. In general, it is prudent to plan that all battered piles be equipped with rock points for adequate seating and prevention of slippage along the bedrock surface during driving. Use of rock points must also consider the presence of artesian-water pressure as discussed in Section 8.4.5.

For steel H-piles driven to found on the limestone or dolostone bedrock, Table 8.5.1, below, provides factored axial resistances at Ultimate Limit States (ULS) for various pile sizes that may be assumed for conceptual and preliminary design.

TABLE 8.5.1 PRELIMINARY AXIAL CAPACITY OF DRIVEN STEEL PILES

PILE TYPE/SIZE	PRELIMINARY FACTORED AXIAL RESISTANCE AT ULS (kN)
HP 310 x 110	2,000
HP 310 x 132	2,400
HP 310 x 152	2,750

The values tabulated above take into account the structural capacity limitation of the pile. An SLS value is not provided because the bedrock is considered to be an unyielding material. Under these conditions, the SLS values (for 25 mm of settlement) do not govern design because the SLS value is higher than the ULS value.

During final design, additional recommendations should be developed with respect to final design capacities, construction specification and control, and capacity assurance testing.

In areas where new fill materials will be required to raise the grades, at the locations of Malden and Machette Roads, Ojibway Parkway, and the Essex Terminal Railway, these fills will result in an increase in the vertical stress in the silty clay deposit which underlies adjacent bridge abutments and piers (should pier areas be subject to filling). Compression of this deposit under the stress change will lead to long-term consolidation settlement. If the end-bearing piles are driven prior to new fill placement or before completion of this time-dependent settlement, a small amount of settlement of the clay relative to the pile will result in the development of negative skin friction on the piles. Therefore, negative friction, or downdrag loads, will need to be taken into account during design as additional dead load with appropriate load factors.

The magnitude of the downdrag load acting on the piles is a function of the adhesion between the pile and the cohesive soils or the friction between the pile and cohesionless soils, and the surface area of the pile within the deposits that will undergo settlement following installation. The unit negative friction acting on a unit area along a single vertical pile can be calculated using the equations provided below.

For cohesionless fill soils

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

where

f_{sn} is the unit negative friction (kN)
β is the shaft resistance factor = 0.6
σ_v’ is the effective vertical (overburden) pressure (kPa)

For cohesive soils

$$q_n = \alpha S_u$$

where

q_n is the unit negative friction (kN)
α is the reduction coefficient
S_u is the undrained shear strength (kPa)

For this site σ_v’, can be approximated for conceptual and preliminary design purposes as:

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

where

γ’ is the buoyant unit weight of soil below groundwater table (assume 11 kN/m^3) or the bulk unit weight of soil above the groundwater table (assume 21 kN/m^3); and
z is the depth below pile cap elevation (kPa).

For preliminary design purposes, Table 8.5.2 provides values of S_u and αS_u used to calculate negative friction.

TABLE 8.5.2 PARAMETERS FOR CALCULATION OF DOWNDRAG ON VERTICAL STEEL PILES

SOIL UNIT	AVERAGE S_u	αS_u
Highway 401/Highway 3		
Silty Clay Crust (to Elevation 182 m)	100	30
Soft to Firm Silty Clay (to Elevation 162 m)	65	30
St. Clair College		
Silty Clay Crust (to Elevation 177 m)	95	30
Soft to Firm Silty Clay (to Elevation 156 m)	60	30
Turkey Creek (Grand Marais Drain)		
Silty Clay Crust (to Elevation 177 m)	80	40
Soft to Firm Silty Clay (to Elevation 155 m)	55	25
Ojibway Parkway/E.C. Row Expressway		
Silty Clay Crust (to Elevation 176 m)	100	30
Soft to Firm Silty Clay (to Elevation 158 m)	35	21

Total downdrag loads are a function of the surface areas of the pile within the soil strata and the vertical effective stress or undrained shear strength mobilised from the top of the embedding layer down to a neutral point (Briaud and Tucker, 1994). For conceptual and preliminary design purposes, the neutral plane may be assumed to be at a depth equivalent to about 85 percent of the silty clay soil strata thickness. The load calculated in this manner is a nominal (unfactored) load. The structural engineer must multiply this load by a load factor of 1.25, as defined in Section 6.8.3 of the CHBDC, and include it as part of the dead load acting on the pile as described in the CHBDC.

The downdrag loads estimated using the above methods are based on the assumption that conventional earth fill will be used for grade changes on this project. Downdrag loads may be relatively large in comparison to the axial resistance of the piles, especially at abutments. Furthermore, the downdrag loads as calculated using the above methods are for vertical piles only. Downdrag loads on battered piles may be substantially greater, may induce unacceptable bending, and must be considered in greater detail during final design should battered piles be used in these areas. Downdrag loads could be reduced or eliminated by installing the piles after the consolidation settlements (due to the new grade raises) are completed or by using lightweight fills in the areas of bridge foundations. The magnitude of any downdrag loads on pile foundations, adjustment of construction scheduling, and potential need for vertical drains and/or lightweight fills should be further examined during final design.

8.5.1.2 Lateral Loads Induced by New Embankments

In addition to downdrag loads, the effect of lateral loading on the piles caused by horizontal soil deformations arising from consolidation and lateral spreading of the native silty clay soils under new embankment loading or lateral loads induced on pile-supported wing-walls should also be considered in the design of pile foundations for bridges or any other structural deep foundations in areas of new fills.

Where the clayey foundation soils are not preloaded prior to pile installation, there will be additional lateral loads acting on the piles. The magnitudes of the lateral loads are difficult to quantify given the complex nature of the soil-structure interaction problem and the early stages of conceptual design for this project. The horizontal component of the soil deformations (i.e. lateral spreading due to the approach embankment loading on the compressible clayey silt soils) is anticipated to be on the order of about 15 percent of the vertical settlement (Ladd 1991). The magnitude of this deformation and the effect it could have on the piles and abutments should also be considered in the design. It is anticipated that the distribution of lateral displacement will be similar in profile to the undrained shear strength of the soil (see Figures 5 through 8) whereby the maximum displacements will occur at the depths of minimum soil strength. Numerical modelling of the soil-structure interaction will be necessary during final design to quantify lateral loads and displacements if pile installation is not delayed until after completion of embankment settlement.

Lateral loads on the piles (and horizontal soil deformations) can be reduced or eliminated by constructing the embankment grade raises in the abutment areas as early as possible in the construction and allowing the settlement and lateral movement to occur prior to pile installation.

8.5.1.3 Resistance to Lateral Loads

The design of piles subjected to lateral loads should take into account such factors as the batter of the piles (if any), the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (pile cap level), the structural capacity of the pile to withstand bending moments, the soil resistance that can be mobilized, the tolerable lateral deflections at the head of the pile and pile group effects. For a longer, more flexible pile, the maximum yield moment of the pile may be reached prior to mobilization of the lateral geotechnical resistance. For preliminary design purposes, both the structural and geotechnical resistances should be evaluated to establish the governing case.

Lateral loading could be resisted fully or partially by the use of battered steel H-piles. For vertical piles, the resistance to lateral loading will have to be derived from the soil in front of the piles. Where integral abutments are under consideration, there will also be a requirement for the piles to move sufficiently to accommodate the bridge deck deflections. Use of battered piles must be considered in light of downdrag load considerations as discussed in Section 8.5.1.1.

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

For cohesionless soils:

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

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

For cohesive soils:

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

s_u is the undrained shear strength of the soil (kPa)
b is the pile diameter or width (m)

For the purposes of preliminary design, it is anticipated that the only cohesionless soils that may be used for lateral load resistance may be imported granular fills, in which case n_h can be assumed equal to about 6,500 kPa/m. Values for S_u are provided in Table 8.5.3, below, for the purposes of preliminary design.

TABLE 8.5.3 PARAMETERS FOR CALCULATION OF LATERAL CAPACITY OF DRIVEN PILES

SOIL UNIT	AVERAGE S_u
Highway 401/Highway 3	
Silty Clay Crust (to Elevation 182 m)	100
Soft to Firm Silty Clay (to Elevation 162 m)	65
St. Clair College	
Silty Clay Crust (to Elevation 177 m)	95
Soft to Firm Silty Clay (to Elevation 156 m)	60
Turkey Creek (Grand Marais Drain)	
Silty Clay Crust (to Elevation 177 m)	80
Soft to Firm Silty Clay (to Elevation 155 m)	55
Ojibway Parkway/E.C. Row Expressway	
Silty Clay Crust (to Elevation 176 m)	100
Soft to Firm Silty Clay (to Elevation 158 m)	35

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor, R, as indicated in Table 8.5.4. Subgrade reaction reduction factors for other pile spacing values may be interpolated for pile spacings in between those listed this table.

TABLE 8.5.4 LATERAL LOAD CAPACITY REDUCTION FACTOR FOR PILE GROUPS

PILE SPACING IN DIRECTION OF LOADING d = Pile Diameter	SUBGRADE REACTION REDUCTION FACTOR (R)
8d	1.00
6d	0.70
4d	0.40
3d	0.25

Reference: Foundations and Earth Structures – Design Manual 7.2, NAVFAC DM-7.2. Department of the Navy, Naval Facilities Engineering Command (1986).

8.5.2 Cast-in-Place Concrete Caissons

8.5.2.1 Axial Geotechnical Capacity

Cast-in-place concrete caissons could be used for support of bridge, abutment, and other heavily loaded structures. It is anticipated that the pile cap elevations would be sufficiently below the nearest adjacent grade to be protected from frost (see subsequent report section). At the time of this report, conceptual designs for bridge structures had not been completed.

For caissons socketed nominally (0.3 m) into sound bedrock, preliminary design may be based on an end-bearing factored axial geotechnical resistance at ULS of 6 MPa. SLS resistances do not apply, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

It should be noted that the base of any caisson excavations must be cleaned of loose rock or soil debris prior to concreting. Considering the length/depth of the caissons, a method such as airlifting will need to be employed and tremie concreting will also be necessary for placing concrete.

In areas where new fill materials will be required to raise the grades, at the locations of Malden and Machette Roads, Ojibway Parkway, and the Eastern Terminal Railway, these fills will result in an increase in the vertical stress in the silty clay deposit that underlies adjacent bridge abutments and piers (should pier areas be subject to filling). Compression of this deposit under the stress change will lead to long-term consolidation settlement. If the end-bearing caissons are constructed prior to new fill placement or before completion of this time-dependent settlement, a small amount of settlement of the clay relative to the caisson will result in the development of negative skin friction on the caissons. Therefore, negative skin friction, or downdrag, loads will need to be taken into account during caisson design.

The magnitude of the downdrag load acting on the caissons is a function of the adhesion (skin friction) between the caisson and the cohesive soils or the friction between the caisson and cohesionless soils, and the surface area of the caisson within the deposits that will undergo settlement following installation. The unit negative skin friction acting on a unit area along a single caisson can be calculated using the equations and values provided in Section 8.5.1.1 if the caissons will be provided with a permanent steel casing. However, if the caissons are constructed using temporary casings only, such that the concrete makes direct permanent contact with the native soil, the adhesion factors will be different than for a steel-soil interface. For preliminary design purposes, Table 8.5.5 provides values of S_u and αS_u used to calculate downdrag loads on concrete caissons without permanent steel casings.

TABLE 8.5.5 PARAMETERS FOR CALCULATION OF DOWNDRAG LOADS ON VERTICAL CONCRETE CAISSONS

SOIL UNIT	AVERAGE S_u	αS_u
Highway 401/Highway 3		
Silty Clay Crust (to Elevation 182 m)	100	50
Soft to Firm Silty Clay (to Elevation 162 m)	65	32
St. Clair College		
Silty Clay Crust (to Elevation 177 m)	95	47
Soft to Firm Silty Clay (to Elevation 156 m)	60	35
Turkey Creek (Grand Marais Drain)		
Silty Clay Crust (to Elevation 177 m)	80	40
Soft to Firm Silty Clay (to Elevation 155 m)	55	32
Ojibway Parkway/E.C. Row Expressway		
Silty Clay Crust (to Elevation 176 m)	100	50
Soft to Firm Silty Clay (to Elevation 158 m)	35	22

Total downdrag loads are a function of the surface areas of the caisson within the soil strata and the vertical effective stress or undrained shear strength mobilised from the top of the embedding layer down to a neutral point (Briaud and Tucker, 1994). For conceptual and preliminary design purposes, the neutral plane may be assumed to be at a depth equivalent to about 85 percent of the silty clay soil strata thickness. The load calculated in this manner is a nominal (unfactored) load. The structural engineer must multiply this load by a load factor of 1.25, as defined in Section 6.8.3 of the CHBDC, and include it as part of the dead load acting on the caisson as described in the CHBDC.

The downdrag loads estimated above are based on the assumption that conventional earth fill will be used for grade changes on this project. Downdrag loads may be relatively large in comparison to the axial resistance of the caissons, especially at abutments. Downdrag loads could be reduced or eliminated by installing the caissons after the consolidation settlements (due to the new grade raises) are completed or by using lightweight fills in the areas of the bridge foundations. Alternatively, it may be feasible to construct the caissons with a permanent lining and bentonite slurry “slip” layer; however, such construction may also prove costly; recommendations for this type of construction can be provided if it is determined that caisson foundations will be considered further.

The magnitude of any downdrag loads on caisson foundations, adjustment of construction scheduling, and potential need for lightweight fills should be further examined during final design.

8.5.2.2 Lateral Loads Induced by New Embankments

Lateral loads will be induced by the construction of new embankments if foundation caissons are constructed prior to completion of the time-dependent settlements as discussed in previous sections of this report (see Section 8.5.1.2)

8.5.2.3 Resistance to Lateral Loads

The effects of lateral loading on the caisson, the resistance to lateral loading developed by the soils in front of the caissons, and the reductions due to group effects, may be assessed for preliminary design purposes as per the recommendations in Section 8.5.1.3.

8.5.3 Frost Protection

All pile caps and other structure foundations should be provided with a minimum of 1 m of conventional soil cover for frost protection. Alternatively, rigid insulation could be used to reduce the required thickness of soil cover over the foundation units. For preliminary design, it can be assumed that 25 mm of rigid insulation is equivalent to 0.6 m of conventional soil cover. Rigid insulation installed for this purpose should be installed on the structures extending down from ground surface to the top of the foundation or pile cap and then extending to a distance of 1 m beyond the perimeter of each foundation unit.

8.5.4 Protection of Piles From Influences of Artesian Water Flow

In some areas along the ACA, artesian water pressures exist. In such areas, groundwater may flow to the surface if a pathway is provided between the bedrock or granular soils near the bedrock surface and the ground surface. If such flow occurs, fine soil particles may be eroded along the flow path and removed from the ground. Such conditions have been known to cause loss of pile capacity where additional protection measures have not been included in design and construction. It is anticipated that driven displacement piles and cast-in-place concrete piles may not experience such erosion as these should be in intimate contact with the surrounding soft to stiff silty clay soils that should effectively seal off such flow. However, it is generally recommended that, in areas subjected to artesian groundwater pressures, a bed of select graded granular fill (typically having a gradation similar to fine aggregate for concrete) be placed around the piles immediately against the native soil surface. Water that flows into this graded filter should also be provided with a controlled exit point to the ground surface. The granular filter material and controlled water exit (pipe) should be selected during final design to limit the potential for removal of fine soils while at the same time providing relief of artesian flow and pressure. For the anticipated construction and soil conditions the volume of such flows should be nominal, producing somewhat wet areas at the ground surface near the foundation locations. Additional drainage controls or pipes directing the nominal seepage to stormwater control facilities may be required to limit the surface effects of such seepage. It should be noted, however, that the water may be naturally brackish and corrosive.

It is recommended that any pile tip reinforcement or rock points avoid increasing the perimeter dimensions of the pile. Some prefabricated or site-welded pile tip reinforcement may result in the pile tip being slightly larger than the rest of the pile. During the driving, this may result in a gap or pathway between the soil and pile that could conduct artesian water flow. Such pile points or tips should not be used.

8.6 Foundations for Other Structures

8.6.1 Bearing Resistance of Shallow Foundations

The DRIC project may include foundations for relatively low above-grade retaining walls (for fill sections), low-rise buildings for plaza structures, signs, and other border crossing facilities. Table 2, following the text of this report, provides a brief overview of the advantages and disadvantages of various foundation types for the DRIC approach corridor project.

It is anticipated that all soils encountered during construction of foundations may be sensitive to disturbance from ponded water, construction traffic and frost. In addition, existing fill materials and, near the river or water courses, organic soils may be encountered near foundation level. Preliminary planning and conceptual design and costing should allow for sub-excavation of all shallow foundations by about 0.1 m with the excess excavation replaced with lean mix concrete that will be subsequently used as a pad on which to continue foundation construction.

Based on the investigations completed for preparation of this report, Table 8.6.1, below, provides SLS and factored ULS bearing resistance values for shallow foundations for various locations along the proposed highway corridor. These values are suitable for preliminary design of lightly-loaded structures in which:

- the final grades surrounding the foundations will be at or within 1.5 m of the existing grades;
- adequate frost protection is provided;
- the foundations have a minimum width of 1 m and a maximum width of 3 m;

- the foundations have their base in the brown and stiff to hard silty clay "crust" typically found above the groundwater level and below depths of between 2 and 3 m; and
- the foundations can tolerate total settlements on the order of 25 mm and differential settlements of about half this amount.

Experience in Windsor indicates that the mottled grey and brown silty clay or clayey silt in the upper part of the soil profile is softer than the underlying stiff to hard brown clayey silt to silty clay in some areas. This zone has been subjected to greater weathering and fluctuation of moisture levels and is less consistent in its strength and consequent performance. For planning purposes, therefore, preliminary factored ULS and SLS values for foundations bearing on the mottled grey and brown silty clay found between the ground surface and the brown "crust" may be taken as half the values provided in Table 8.6.1. Shallow foundations that may be planned with their bearing levels below the groundwater level should be examined on a case-by-case basis as the SLS and ULS resistances may be significantly lower. Although the table below provides foundation design parameters suitable for conceptual and preliminary design, final resistance and settlement estimates should be developed during final design pending additional subsurface explorations and testing.

TABLE 8.6.1 PRELIMINARY SHALLOW FOUNDATION BEARING RESISTANCE FOR FOUNDATIONS IN STIFF TO HARD SILTY CLAY (BROWN) CRUST

INTERSECTION	RESISTANCE AT SLS (kPa)	RESISTANCE AT FACTORED ULS (kPa)
Highway 401	250	400
St. Clair College	200	320
Huron Church Line	150	240
Turkey Creek	125	200
E.C. Row Expressway	150	240
Ojibway Parkway	100	160

8.6.2 Frost Protection

All foundations should be provided with a minimum of 1 m of conventional soil cover for frost protection. Alternatively, rigid insulation could be used to reduce the required thickness of soil cover over the foundation units. For preliminary design, it can be assumed that 25 mm of rigid insulation is equivalent to 0.6 m of conventional soil cover. Rigid insulation installed for this purpose should be installed on the structures extending down from ground surface to the top of the foundation or pile cap and then extending to a distance of 1.2 m beyond the perimeter of each foundation unit.

8.7 Lateral Earth Pressures on Above-Grade Retaining Structures

Earth pressures for the design of abutments or wingwalls for bridges, noise walls, or retaining walls for landscaping will depend entirely on the materials used for backfill. In general, it is recommended that all gravity walls be backfilled with granular soils such that design earth pressures are of typical magnitudes, where the active earth pressure coefficient may range between about 0.25 and 0.30, and groundwater can be drained from the backfill. Compaction pressures, however, may dominate design conditions near the top of any gravity or cantilever walls. In some cases, it may be desirable to use the excavated native silty clay soils for large earth fills that may also involve retaining structures. For these cases, it is recommended that the backfill materials consist of granular soils for a distance perpendicular from the back of the wall of at least one-half of the wall height to achieve the range of active earth pressures stated above. If retaining walls are backfilled with cohesive soils, pressures from freezing within the cohesive soils will likely not be acceptable, even if prefabricated or granular drains are provided immediately behind the wall.

8.8 Noise Barrier Wall Foundations

Where walls are to be built up from the existing grade and are to be used to retain noise berm fill or embankments, the wall heights may be limited due to foundation bearing capacity and settlement considerations. Bearing capacity values provided in Table 8.6.1 may be used for conceptual and preliminary design assessment of noise barrier walls supported on shallow foundations. Use of such walls for retaining embankments and fills will depend on further analysis considering overall stability, total settlement, and differential settlement. In some cases, relatively thin and free-standing noise barrier panel walls may be supported using driven or drilled pile foundations as typically used for other projects in Ontario. In such cases, it is anticipated that resistance to lateral loads may be the more critical foundation design case. The methods provided in Sections 8.5.1.3 and 8.5.2.3 may be used for preliminary and conceptual design of driven or drilled foundations for these walls.

8.9 Groundwater Control and Its Influence on Design and Construction

Groundwater conditions within the bedrock or overlying granular soil aquifer that induce uplift pressures will significantly affect the feasibility of constructing deep excavations unless other excavation stability enhancement measures are implemented. Such stability enhancements commonly include dewatering or depressurization of groundwater levels. Significant temporary dewatering and permanent depressurization of groundwater levels was undertaken in similar soil conditions in Welland during construction, and later operation, of the Townline Road and Main Street tunnels beneath the Welland Canal (Farvolden and Nunan 1970, Frind 1970, Olpinski 1970, Golder project files). Dewatering the bedrock aquifer in Welland for these tunnel projects resulted in extracted water volumes on the order of 1,500 to more than 6,000 litres per minute with a zone of influence (where the groundwater levels were drawn down) of many kilometers surrounding each pumping site. The long-term depressurization of the groundwater has created consolidation within the overlying silty clay, though because the area was primarily rural at the time, few surface facilities were affected. If such dewatering is undertaken in Windsor, the potential effects on surface features would be much greater.

Groundwater in the Windsor area contains hydrogen sulphide and managing this dissolved gas, should dewatering be undertaken, will be critical to the safe completion of the project. There is also the potential for groundwater flows through fractures within the upper horizons of the bedrock to be significant and/or prohibitive for temporary construction dewatering of relatively large areas. In addition, depressurization or dewatering of either granular soils near the bedrock interface or the bedrock will induce measurable consolidation settlements within the overlying silty clay soils. Table 8.9.1 provides preliminary estimates of settlement induced by various levels of groundwater drawdown. As with embankment settlement, the settlement caused by dewatering will be time-dependent, with the maximum settlements occurring over a period of many months. Although the time required to induce these settlements may be on the order of six months or up to three years, it is anticipated that some of the larger excavations for which groundwater depressurization or dewatering may be contemplated could be open for such periods of time. Therefore, dewatering or depressurization-induced settlements must be evaluated in detail during final design.

TABLE 8.9.1 PRELIMINARY ESTIMATES OF DEWATERING-INDUCED SETTLEMENTS

GROUNDWATER DRAWDOWN (m)	ESTIMATED SETTLEMENT (mm)			
	HIGHWAY 401 / HIGHWAY 3	St. Clair College	Turkey Creek	Ojibway Parkway
4	<20	<100	140	120
8	55	180	235	225
12	115	255	345	330

Depending on the local strength and compressibility of these soils, such settlements may cause damage to structures or other overlying facilities depending on the zone of influence of depressurization. In some cases, the zone may be laterally extensive with differential movements being inconsequential to overlying structures. In other instances, however, the zone of influence may be more limited and differential settlements could cause damage unless measures such as grout curtains through and within the granular soils and bedrock or re-injection systems are implemented. Such differential movements may be particularly problematic or severe where structures are supported by deep foundations and are connected to grade-supported utilities. Detailed investigations, testing, and analyses will be required during final design to adequately assess the feasibility of dewatering or depressurization of the groundwater levels within the bedrock or the granular soils separating the bedrock from the overlying silty clay deposits.

Creating permanent, open, and below-grade roadways within the native clays using slopes or supported with retaining walls (that do not cut off groundwater pressure gradients from adjacent higher grades) will result in a permanent lowering of the groundwater level within the clay soils. Based on the estimated variation in vertical and horizontal permeability and the analytical approach of Powers et al. (2007), and for preliminary planning purposes, it is anticipated that the zone of influence of such groundwater lowering within the silty clay should be assumed to be a distance up to about 5 times the depth of cut. Such groundwater lowering will induce settlement within the silty clay subsoils within this zone. It is anticipated that if low permeability in situ walls (e.g. contiguous caisson walls or concrete diaphragm walls) are used for excavation support or for permanent below grade structures, the influence of the excavation on near-surface groundwater would be much less. Further refinement of this zone of influence and the magnitude of potential settlement requires additional site-specific investigation and analyses.

8.10 Seismic Design Considerations

Although construction of either a below-grade roadway or a cut-and-cover tunnel represents a large engineering effort, the design of the below-grade or tunnel alternatives may not be as sensitive to seismic loading or hazards. However, both alternatives include the construction of above-grade bridge structures and associated embankments, particularly near the western limit of the project where the alignment crosses the Essex Terminal Railway, Ojibway Parkway, Matchette Road, and Malden Road. These structures will be affected by seismic excitation and will require appropriate seismic analysis and design procedures.

In addition to the above-mentioned highway bridges, it is understood that a structure crossing the Detroit River is planned for this project. Recommendations related to the main river-crossing bridge structure are provided in a separate report.

A preliminary evaluation was carried out using the profile and plan views of the proposed alignments, as well as the interpreted subsurface conditions as summarized in this report.

8.10.1 Background Seismic Analysis and Design Methods

Several different seismic design and analysis guidelines and relevant design codes are discussed below, along with a recommended approach for conceptual and preliminary design. Since the practice and codes related to seismic design are evolving, it will be necessary for the project stakeholders to agree on the seismic design criteria and codes that will be applied to this project.

8.10.1.1 Canadian Highway Bridge Design Code

The 2006 Canadian Highway Bridge Design Code (CHBDC) is the fundamental specification for bridge design in Canada and it is based on seismic hazard as defined in the 1995 National Building Code of Canada (NBCC). The seismic hazard is defined using the Zonal Acceleration Ratio (A), defined as in NBCC 1995. The design earthquake is defined as having a probability of exceedance of 10 percent in 50 years. In the CHBDC bridge design specifications, an Elastic Seismic Response Coefficient (C_{sm}) is used to define the spectral shape. The spectral shape is a function of the Acceleration Coefficient (A), given in the CHBDC, the site coefficient, the importance factor and the period of the bridge. The soil profile types and site coefficients are similar to those in NBCC 1995. The importance factor (I) is used to scale the elastic seismic response coefficient. For lifeline bridges I=3.0, for emergency-route bridges I=1.5 and for other bridges I=1.0.

Using this methodology, the soil profile type for the DRIC ACA would be Type 3 (soft to medium stiff clays) and the site coefficient, S, would be 1.5. The zonal acceleration ratio for Windsor is 0, but a minimum value of 0.05 is used to construct the acceleration spectra as per the CHBDC. Figures 27 and 28 show the acceleration spectra for Windsor with an importance factor of 1.0, and Figures 29 and 30 show the acceleration spectra with an importance factor of 1.5. The structures associated with the Detroit River International Crossing may be considered lifeline or emergency route bridges in which case the design spectra would be multiplied by the corresponding importance factor.

8.10.1.2 National Building Code of Canada 2005

The National Building Code of Canada was published in 2005 with an updated seismic analysis and design methodology. Seismic hazard is now defined by uniform hazard spectra (UHS) at spectral coordinates of 0.2s, 0.5s, 1.0s and 2.0s. The probability of exceedance of the seismic hazard specified by means of the UHS is 2 percent in 50 years. In the 2005 edition of NBCC, the 1994 National Earthquake Hazard Reduction Program (NEHRP) site categories and response factor are adapted to the reference ground condition for Canada. The reference ground condition adopted by the 2005 NBCC is Site Class C. The 2005 NBCC method defines the site class by the shear wave velocity, undrained shear strength or standard penetration resistance in the top 30 meters of soil. There are six site classes from A to F, decreasing in soil strength from A (hard rock) to E (soft soil) with site class F to denote particularly vulnerable soils. The site class is used to obtain soil factors, F_a and F_v, used to modify the UHS to account for the effects of soil conditions in design. The 2005 NBCC uses an importance factor I_e to multiply the base shear for seismic design. Normal structures are assigned an I_e =1.0, high importance category structures are assigned I_e =1.3, and post-disaster structures are assigned an I_e =1.5. The importance factor is not applied directly to the spectral acceleration used in seismic design.

Using the NBCC 2005 methodology, the soil profile type between Highway 401 and near St. Clair College is categorized as site Class D - soft soil with an undrained shear strength of about 50 kPa to 100 kPa. From near St. Clair College to the Detroit River, the soil profile type is categorized as Class E - soft soil with an undrained shear strength of about 50 kPa or less. The F_a and F_v values for the Class D areas are 1.3 and 1.4, respectively. For the Class E areas, the F_a and F_v values are both 2.1. The reference spectral acceleration coordinates for Windsor are $S_a(0.2)=0.18$, $S_a(0.5)=0.086$, $S_a(1.0)=0.04$, $S_a(2.0)=0.011$ and PGA=0.12. Figures 27 through 30 show the spectral acceleration for Windsor site Classes D and E, with importance factors of 1.0 and 1.5 applied.

8.10.1.3 ATC 2003 Recommended LRFD Guidelines for the Seismic Design of Highway Bridges

In 2003 the Applied Technology Council and Multidisciplinary Council for Earthquake Engineering Research (ATC/MCEER) published seismic design guidelines to be used as a supplement to the AASHTO bridge design specifications. It is anticipated that these new guidelines would form the basis for the next revision to both AASHTO and CHBDC seismic design codes. The changes in ATC/MCEER 2003 guidelines include the adoption of new USGS maps, more clear performance objectives, design incentives, new soil factors and new spectral shapes. In 1996 the USGS published new seismic hazard maps to be used in the United States (Frankel et. al., 1996). The new seismic hazard values are presented as contour maps or tabulated values. The PGA values and the spectral acceleration at 0.2, 0.3 and 1 seconds are given. The USGS has presented these values at three probability levels: 10 percent in 50 years; 5 percent in 50 years; and 2 percent in 50 years. The spectral shape recommended in the 2003 guidelines is based on the 0.2 second and 1 second spectral accelerations. The values of the 0.2 second and 1 second spectral acceleration are determined based on the uniform hazard spectra procedure and as such, both values have the same probability of exceedance. The spectral shape used in the 2003 guidelines increases in the short period range to a plateau level and then decreases in the long period range. The 2003 ATC/MCEER guidelines also adopt the site class and site factors recommended by NEHRP in 1994 (also adopted in 2005 NBCC). The reference ground condition considered in the 2003 ATC/MCEER guidelines is site Class B (rock), whereas in the 2005 NBCC the reference ground condition is site Class C (soft rock). Therefore the F_a and F_v values used in design by the procedures of the two codes are slightly different. The 2003 guidelines for the Seismic Design of Highway Bridges uses seismic performance objectives based on life safety or operational criteria; no importance factors are used.

Using the ATC 2003 methodology, the soil profile type would be site Classes D and E similarly for the areas described above for the NBCC approach. The F_a and F_v value would be 1.6 and 2.4, respectively, for the Class D areas, and 2.5 and 3.5, respectively, for the Class E areas. The reference (Class C) spectral acceleration coordinates are $S_a(0.2)=0.12$, $S_a(1.0)=0.04$ and PGA=0.06 for a probability of exceedance of 2 percent in 50 years for Detroit. The USGS and

GSC have not developed a consistent framework for hazard definition, thus GSC defines the reference Class C PGA for 2 percent in 50 years as 0.12. Figures 27 and 28 show the spectral acceleration for site Classes D and E.

8.10.2 Seismic Hazard Assessment

The site location has historically been considered to be in an area of low seismicity, with PGA values of less than 0.05g from an earthquake with a 10 percent probability of exceedance in 50 years. New hazard models and a move to design earthquakes with a 2 percent probability of exceedance in 50 years now define reference (Site Class C) PGA values in the order of 0.12g.

To reflect the actual Site Class conditions of D and E, the site-specific PGA value would be amplified to about 0.25g, which represents a moderate level of ground shaking. Such ground shaking could be reflected in a potential for seismic liquefaction in loose, saturated granular deposits. However, the borehole data does not indicate the presence of such deposits at the site.

Nonetheless, the seismic stability of earthen embankments should be assessed in consideration of the moderate level of ground shaking. In addition, retaining walls will need to consider the lateral pressures induced by such seismic shaking.

8.10.3 Preliminary Seismic Design Summary

Because of recent developments in the quantification of seismic hazard, the CHBDC will likely be updated to adopt the specification of seismic hazard in terms of the UHS at 2 percent in 50 years (Adams et. al., 2003) and the NEHRP 1994 site classification system. However in order to incorporate the new information and practices, the CHBDC 2006 method of seismic analysis and design needs to be modified, much as NBCC had to update their seismic methodology from 1995 to 2005. The recommended LRFD guidelines published in 2003 by ATC/MCEER provide a likely framework to incorporate these changes into the seismic analysis and design methodology of the next generation CHBDC. It is suggested that a design approach using performance-based seismic design with the ATC 2003 performance objectives and the NBCC 2005 seismic hazard definition and site factors be used for the Detroit River International Crossing project. Prior to final design, the seismic design approach needs to be agreed upon by all agencies involved in the funding and design overview of the DRIC project.

8.11 Soil and Groundwater Management

Surplus soils excavated for the proposed approach roadways must be managed and disposed of according to appropriate regulatory guidelines with respect to environmental quality. Analysis of the environmental quality and chemistry of soil and groundwater is beyond the scope of this report but must be undertaken during final design. It is recommended that, during final design of the project, a detailed management and disposal strategy be developed to consider the following:

- land use history along and immediately adjacent to the alignment with respect to the potential for environmental contaminants to be present within the soils or groundwater;
- reuse of excavated soils for construction and landscaping purposes;
- hauling and disposal of large volumes of earth materials that may not be suitable for reuse on this project as a result of their physical consistency or environmental contamination, either as a result of their in-place condition or construction processes;
- management and disposal of water collected in construction sumps that could include potential contaminants from construction processes (e.g. lubricating oils, fuels, etc.); and
- management and disposal of groundwater collected in dewatering or pressure relief systems that is likely to contain hydrogen sulphide.

8.11.1 Reuse of Excavated Soil Materials

Based on conceptual plans, the embankments along the approach corridor parallel to E.C. Row Expressway may require a volume of fill materials on the order of 1,000,000 m^3. The existing soils are predominantly fine-grained in nature (silt and clay) and as a result their physical properties are sensitive to changes in moisture content.

The native silty clay or clayey silt soils below a depth of 1 m and above the groundwater level are likely to be near their optimum water content for compaction, whereas those below the groundwater level will likely be saturated and well above their optimum water content for compaction.

Soils above the groundwater level will also likely be firm to hard in consistency, and the lumps of soil resulting from mass excavation will need to be broken to permit adequate compaction. Therefore, if the soils are to be reused for embankments or other earthworks, it should be anticipated that reworking of the soils will be necessary to facilitate compaction through slight wetting, breaking or drying with discs, and use of sheep’s-foot roller compactors. Inclement weather will produce soft, wet, and muddy areas unless the surface is appropriately graded at the end of each work shift. The native silty clay and clayey silt materials are also frost-susceptible and should be protected from freezing temperatures during placement. In addition, the silty clay, if exposed to hot and dry weather, may form hardened lumps that impede spreading and compaction efforts. Control of moisture content during placement and compaction will also be essential for maintaining adequate performance of the final embankment materials. Clayey silt and silty clay, when it is placed and compacted at moisture contents less than the optimum for compaction, may be subject to additional compression after placement, particularly on subsequent wetting or saturation of these materials. Therefore, if the native materials are to be reused for embankment construction, it is generally recommended that they be placed and compacted at moisture contents within about 10 to 15 percent wet of their optimum compaction water content (for example, if the optimum compaction water content is 20 percent, the fill should be compacted at a moisture content ranging between about 20 and 23 percent by weight). While the soils may be used for construction of large earthworks (embankments, noise berms, etc.), compaction and moisture control may be challenging.

Design of pavements supported by embankments constructed of native clayey silt and silty clay soils must consider the frost-susceptibility of these soils. To minimize the effects of freezing temperature, proper drainage and protection of these soils from saturation will be essential. Pavement design and subgrade control are the subject of a separate report prepared for this project.

As noted above, native silty clay and clayey silt from below the groundwater table (the grey soils) are anticipated to be well above their optimum moisture content for compaction and, therefore, should be considered unsuitable for use as compacted, engineered fills.

8.11.2 Management of Construction Dewatering Flows

Along the ACA, groundwater pressure head levels within the granular soils and bedrock (near the bedrock interface) range from about Elevation 182 m near the intersection of Highway 401 and Highway 3 (about 3.5 to 4 m below ground surface), to about Elevation 179.7 m near E.C. Row Expressway and Ojibway Parkway (about 1.2 m above ground surface). During drilling of Borehole BH-23, artesian groundwater containing hydrogen sulphide was encountered. Similar groundwater conditions were encountered during drilling for the potential bridge crossing sites along and west of Sandwich Street. In one instance, groundwater flowed up and around one of the 440 mm diameter surface casings (not up through the centre) installed to the top of the bedrock at a rate of between 120 and 200 litres per minute (L/min), as documented within a separate report for the DRIC project.

At the time this report was prepared, no detailed dewatering assessments had been completed as the locations and dimensions of the potential areas requiring groundwater control had not been defined. Based on the anticipated condition of the soil and bedrock near the bedrock interface and the likely overall dimensions of construction, it is likely that significant volumes of water would require extraction in order to have measurable effects on the groundwater pressures. The natural groundwater contains hydrogen sulphide that must be managed and may require treatment during any collection process. Disposal of the volumes that might be generated by construction dewatering may be impractical or prohibitively costly and will certainly require that a permit to take water be obtained from the MOE for the project.

8.12 Instrumentation and Construction Monitoring

During final design, it is recommended that detailed strategies and plans be developed for monitoring of field construction performance. Measuring field performance will be critical to the success of many aspects of the construction, but particularly so for deep excavations and construction of highway embankments in soft ground areas. Measuring lateral and vertical displacements adjacent to deep excavations will assist with comparisons of design expectations with field performance, warning of potential problems, documentation with respect to adjacent facility owners, and defence of claims. Furthermore, construction scheduling and the timing of staged embankment construction will depend on the results of monitoring settlement and pore-water pressure dissipation in high embankment areas.

For monitoring of deep excavations, it is generally recommended that an instrumentation program be established that includes a number of instruments and procedures to adequately monitor the performance of the work in comparison with design expectations and protection of neighbouring facilities (roadways, utilities, buildings, etc.). Such a program should include the following:

- Regularly-spaced settlement monitoring points should be installed along the edge of the excavations, close to the back of the retaining structures. These settlement monitoring points should be designed such that they penetrate any local pavement structures (as pavements can mask underlying ground movements) and are not susceptible to frost movements. The spacing of the settlement monitoring points will depend on the final design and the particular construction location; however, the spacing should be on the order of every 20 m to 30 m on both sides of the excavations.
- At intervals of about 150 m to 200 m, it is recommended that more extensive monitoring of the ground conditions be undertaken. At these locations, inclinometers should be installed to measure the lateral displacement of the retaining structures. These inclinometers should be installed prior to construction of the retaining structures and extend to at least 2 m into bedrock to that the full lateral displacement can be captured. Ground monitoring points should be installed in lines (on both sides of the excavation) perpendicular to the face of the retaining structure, at distances approximately equal to 0.1H, 0.2H, 0.5H, 1H, 1.5H and 2H, where H is equal to the excavation depth. Piezometers should also be installed at these locations to monitor groundwater pressures within the bedrock and overburden soils.

Where base stability is marginal, either as related to base heave (soil strength) or uplift (groundwater pressures), or near sensitive facilities, it may be necessary to supplement the general guidelines provided above with additional instrumentation.

For high fill embankments, monitoring will also be necessary to monitor the stability of the embankments during fill or surcharge placement and to monitor the progression of settlement as related to construction of bridge foundations and final paving. Typically, such monitoring programs should include the following:

- Inclinometers should be installed near the toe of embankments that exceed 5 m to 7 m in height where these embankments are close to bridge foundation structures. The inclinometers will assist with assessing the potential for embankment failure as well as the magnitude of lateral spreading.
- Measurement of porewater pressures will be critical to understanding when stages of filling and surcharging may proceed. During fill placement, the porewater pressures within the underlying clayey silt and silty clay soil will increase and, to maintain stability, the porewater pressures should not exceed specific threshold levels that are based on staging analysis and design. It is recommended that monitoring of porewater pressures be carried out with both vibrating wire piezometers as well as standpipe type piezometers. The vibrating wire piezometers have the advantage of responding more quickly and with greater resolution than standpipe piezometers and, if the connecting wires are broken by construction equipment, they can be reconnected relatively readily. Standpipe piezometers are necessary for calibration of the vibrating wire piezometers during initial installation as well as later during the progress of the work. Piezometers should be installed at several depths (selected based on final design calculations) beneath the centrelines of each of the eastbound and westbound lanes of embankments where these are equal to or greater than about 5 m in height. In general, the piezometers should be installed at the locations where the embankments are highest and near bridge structure locations at approximately 100 m intervals along the freeway centreline. Reference piezometers should be installed at these locations and at depths similar to those beneath the embankment but at about 20 m distant from the toe of slope to measure baseline pore-water pressures at the selected depths.
- Embankment settlement should be measured at each of the piezometer locations described above, on approximately 100 m spacing along the freeway. Settlement should be measured at the crest of each embankment slope, the centrelines of the east and westbound embankments, and at the freeway centreline (for a total of five positions across the highway). Settlement should be measured by a combination of vibrating wire settlement cells as well as conventional settlement platforms, with the mix of instruments selected based on final design and construction planning. Settlement platforms have the advantage of providing a direct physical measurement of embankment settlement but can (and often are) damaged by construction equipment. Vibrating wire settlement cells can be installed at the interface between the existing grade and new embankment with the remote monitoring connections run to a convenient location outside of the construction traffic area. The vibrating wire instruments can provide better resolution than surveying of settlement platforms and do not require a surveying crew, but can be more sensitive to equipment and calibration reliability difficulties. A mix of both types of instruments should be planned for final design and construction. In addition to the fully instrumented locations described above, it is also recommended that single settlement measuring devices be located at approximately 50 m intervals along the length of embankments to gauge the general longitudinal settlement performance of the embankment(s).

Prior to construction, it is recommended that detailed condition surveys be carried out for buildings and other facilities near deep excavations, or areas where structures or utilities may be affected by settlements associated with high embankment fills or dewatering. For conceptual and preliminary design, it is recommended that such surveys be completed for buildings and other facilities within a distance equal to 2.5 times the excavation depth from the excavation edge. Guidelines for condition surveys have been published by Building Research Establishment

(BRE 1989). Following construction, the condition surveys should be updated noting changes, if any, since the preconstruction survey.

It is generally recommended that, for projects of this nature, a detailed monitoring program be undertaken by the owner rather than delegating such work as the sole responsibility of the contractor, regardless of the type of contract(s) undertaken (design-bid-build, design-build, etc.). It is considered in the owner's best interest to have control over a program that allows comparison of actual performance to design expectations, contractual requirements, the influence of the work on third-party properties, and the safety, technical, legal, and insurance implications that unsatisfactory field performance may have.

8.13 Further Investigation

The evaluations described in this report are based on exploration and testing carried out as part of this project, a review of the documented subsurface conditions gathered for other projects, and the highway access route concepts developed and provided to Golder by URS. These evaluations have been provided to allow a general assessment of the different retaining wall and tunnel construction systems that may be used along the proposed Highway 401 extension. It is essential that detailed geotechnical and hydrogeological investigations are conducted as part of final design because the strength of the soils, the groundwater levels, and the hydraulic conductivity of the soils and bedrock will have significant effects on the final design alternatives and costs. The level of investigation effort conducted for this report will not be sufficient for final design.

Preparation of investigation programs suitable for design of the selected alternative should be developed once the final design concept has been chosen. Based on the investigation and testing completed for this report, general guidance on the final investigation program is provided below.

Bridge Structures

A minimum of two boreholes should be completed for each conventional four-lane cross-street bridge foundation location to bedrock with a minimum of 3 m of rock coring completed in each hole. Additional boreholes may be necessary for six-lane bridges or for wider bridge structures. Each of these boreholes should include standard field vane shear testing at 3 m intervals, thin-wall tube sampling at 3 m intervals through the clayey silt and silty clay profile, and Standard Penetration Testing in any granular deposits. A piezometer should be installed and sealed within the bedrock in one of the boreholes, and the second borehole should include a piezometer sealed within the silty clay to clayey silt deposit at approximately 10 m below ground surface. Rising head hydraulic conductivity tests should be completed in all piezometers. Every soil sample should be subjected to natural water content determinations with Atterberg limits determinations completed on approximately 25 percent of the samples. A total of three thin-wall tube samples for each bridge site should be subjected to laboratory triaxial testing (isotropically consolidated, undrained compression tests with porewater pressure measurements) with confining pressures chosen to be equivalent to the estimated in situ horizontal effective stress. The samples selected for the triaxial testing should be selected from three relatively evenly spaced locations within one of the boreholes and below the planned foundation elevation. A minimum of three unconfined compression tests should also be conducted on rock core samples taken from each borehole.

Below-grade Roadway Cut Sections

Along any cut sections, including those sections that may be constructed as cut-and-cover tunnels, exploration locations should be spaced at approximately 75 m intervals. These explorations should extend to a depth at least equal to twice the cut depth. If the cut sections are to be deeper than about 10 m, it is recommended that these explorations be extended to bedrock. Boreholes taken to bedrock should include a minimum of 3 m core into bedrock. Piezocone penetration tests (CPT) should be used for approximately half of these exploration locations with conventional boreholes completed at the remaining locations. Each of the conventional boreholes should include standard field vane shear testing at 3 m intervals, thin-wall tube sampling at 3 m intervals through the clayey silt and silty clay profile, and Standard Penetration Testing immediately following each thin wall tube sample and in any granular deposits. Piezometers should be installed and sealed at approximately 2 m below the bottom of cut elevation in half of the boreholes. Piezometers should also be installed and sealed within the bedrock in all boreholes that include rock coring and rising head hydraulic conductivity tests should be completed in each piezometer. Every sample should be subjected to natural water content determinations with approximately 25 percent of the samples subjected to Atterberg limits determinations. For each borehole that extends to bedrock, a total of three thin wall tube samples should be subjected to laboratory triaxial testing (isotropically consolidated, undrained compression tests with porewater pressure measurements) with confining pressures chosen to be equivalent to the estimated in situ horizontal effective stress. The depths of the samples subjected to this testing should be selected to be in the range of 0.5H, 1H and 1.5H, where H is the depth of the cut.

High Fill Embankments

For high fill embankments, exploration locations should be spaced at a maximum of approximately 75 m intervals along the alignment of the embankment. It is also recommended that these explorations be located near the crest of the proposed embankments on alternating sides of the embankment such that an indication of the transverse variability of the subsurface soil conditions and bedrock surface is obtained. Piezocone penetration tests (CPT) should be used for half of the planned locations with conventional boreholes completed at the remaining locations. These boreholes should be cored at least 1.0 m into bedrock. Each of these holes should include standard field vane shear testing at 3 m intervals, thin-wall tube sampling at 3 m intervals through the clayey silt and silty clay profile, and Standard Penetration Testing immediately following each thin-wall tube sample and in any granular deposits. Piezometers should be installed and sealed at a depth of approximately 10 m below existing grade in half the boreholes. The piezometric pressure head should also be measured for each of the boreholes that include rock coring. It should be noted that groundwater pressures west of Huron Church Road in the ACA near the E.C. Row Expressway will be artesian and the exploration program should anticipate groundwater pressure heads of up to 2 m above ground surface. During piezocone penetration testing, it is recommended that at least nine pore water pressure dissipation tests be carried out at evenly spaced elevations within one test location to assist in assessing the horizontal permeability of the soils. Every soil sample should be subjected to natural water content determinations and approximately half of the samples should be subjected to Atterberg limits determinations. Two boreholes should be selected from each embankment area to include detailed laboratory testing. From each of these boreholes, three thin-wall tube samples should be selected and subjected to laboratory triaxial testing (isotropically consolidated, undrained compression tests with porewater pressure measurements) with confining pressures chosen to be equivalent to the estimated in situ horizontal effective stress. In addition, conventional oedometer (consolidation) tests should be completed on specimens selected from the same thin-wall tube samples as for the triaxial tests.

Other Investigation Considerations

Explorations and testing will also need to be completed for proposed structures such as high mast lighting, large sign structures, and buildings for ventilation equipment, maintenance facilities, or operations facilities. At the time this report was prepared, the conceptual design was not completed to the point at which provision of guideline exploration and testing recommendations was appropriate. If extensive dewatering or depressurization of groundwater is contemplated for final design, pumping tests and packer pressure tests may be required within the bedrock and near the soil-bedrock interface. Furthermore, chemical testing of the groundwater should be completed to ascertain potential treatment and disposal options. Additional exploration, testing, and analysis may also be required to characterise the sites for seismic design depending on the seismic criteria to be used for final design and to complete site-specific evaluations where necessary.

9.0 SUMMARY AND CONCLUSIONS

The available subsurface information and preliminary evaluations completed as part of this study suggest the following:

- Construction of open-cut (below-grade roadway) sections may be made to assist in separating traffic grades with permanent side slopes of approximately 2.5:1 (horizontal:vertical) transitioning to 3:1 between Highway 401 and E.C. Row Expressway or with permanent retaining structures (using a variety of systems), provided that the cut depths are limited to be consistent with the transition in ground strength and groundwater conditions from east to west along the corridor.
- Construction of high embankments on relatively compressible soils along the ACA, particularly in the areas parallel to E.C. Row Expressway and near Ojibway Parkway, will require the use of staged embankment construction and measures to accelerate or mitigate settlement (such as wick drains, surcharging, and/or lightweight fill).
- Cut-and-cover tunnels should be feasible for the entire length of the approaches; however, base stability conditions may require either temporary ground improvement measures or other temporary wall and base stability enhancements during construction of excavations, particularly where these excavations are deeper than about 10 m to 12 m between Highway 401 and E.C. Row Expressway.
- Where in situ retaining walls are constructed for cuts deeper than about 5 to 6 m that are within a distance of about 1.5 times the excavation depth from displacement-sensitive facilities, it is anticipated that a reinforced concrete retaining system such as either secant/tangent pile or concrete diaphragm walls will be required to control ground displacements.
- Foundations for bridges and other heavily-loaded structures should consist of either driven piles or caissons (drilled shafts) bearing on sound bedrock. Shallow spread foundations may be suitable for some lightly- to moderately-loaded structures, but the feasibility of using spread footing foundations will be sensitive to the design loads and location along the ACA corridor.
- The need for controlling groundwater within the bedrock (or near bedrock) aquifers should be minimized as the natural groundwater can produce hydrogen sulphide gas on exposure to atmospheric pressure. Final design should limit the total depth of excavations so that the need to lower groundwater is avoided to the extent practicable. Between Huron Church Road and the Detroit River, groundwater pressures become artesian and these conditions will also require additional consideration during final design and construction.
- It is recommended that a design approach using performance-based seismic design with the ATC 2003 performance objectives and the NBCC 2005 seismic hazard definition and site factors be used for seismic design. This recommended approach, or an alternative, needs to be agreed upon by the various agencies involved with the project prior to final design.

The feasible cut depths for excavations are particularly sensitive to local soil strength values and, along much of the alignment, the potential cut depths are near the threshold of instability; therefore, further investigation and testing will be necessary for evaluating appropriate excavation support systems and any stability enhancement measures.

Given the scope and scale of this project, the geotechnical factors of safety as discussed within this report may have a significant influence on the cost and complexity of the work. Consideration should be given to examining these factors to ensure that they satisfy the Ministry of Transportation Ontario design and economic goals as well as an acceptable balance between cost and risk.

Each of the below-grade construction alternatives and the cuts made for cross-street underpasses for the at-grade option discussed in this preliminary evaluation induce differing degrees of risk to adjacent facilities and overlying roadways; some of these risks may be acceptable, while others may not. The options also carry differing degrees of risks to the project design and the construction cost and schedule. The tolerance of the adjacent and overlying facilities to displacements should be addressed in progressively greater detail as the project evolves, taking into account changes in conceptual and, at later stages, design geometry in combination with future subsurface investigation results. The owner’s tolerance of varying levels of technical, cost, and schedule risks and the risks to third parties should also be examined in detail as the project develops and appropriate risk analysis, mitigation, and management strategies developed appropriate to the stage of design or construction (e.g. ASCE 1997, Westland et al. 1998, Eskessen et al. 2004, ITIG 2006, Boone 2007).

10.0 CLOSURE

This report has been prepared to assist the DRIC project team with evaluating conceptual alternatives associated with the highway access route to the Detroit River crossing. Within this report estimated values are provided for displacement and design stresses. The level of precision and accuracy of the presented results is consistent with conceptual and preliminary design and is considered to provide a reasonable basis for comparison among different design options and to gauge the potential order of magnitude of the associated engineering issues. Additional geotechnical analyses must be completed during final design, however, as it is anticipated that subsequent changes in highway design may have a significant influence on final geotechnical recommendations. As this work was prepared to assist with conceptual alternatives and is based on available data and supplementary explorations and testing, the recommendations provided within this report should be reviewed and revised as necessary as further information is developed with respect to design concepts, more detailed structure locations, and subsurface information.

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RECORD OF BOREHOLE SHEETS

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.) 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, 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	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.

LIST OF SYMBOLS

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

I. General

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

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\ \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
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
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_α	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 = σ'_p/σ'_{vo}

(d) Shear Strength

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

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)

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering.

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

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

GRAIN SIZE

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

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

CORE CONDITION

Total Core Recovery (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

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

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

Description and Notes

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

Abbreviations

B – Bedding
FO - Foliation Schistosity
CL - Cleavage
SH - Shear Plane Zone
VN - Vein
F - Fault
CO - Contact
J - Joint
FR - Fracture
M F - Mechanical Fracture
II - Parallel To
⊥ - Perpendicular To

P - Polished
S - Slickensided
SM - Smooth
R - Ridged / Rough
ST - Stepped
PL - Planar
FL - Flexured
UE - Uneven
W - Wavy
C - Curved

Golder Associates
London, Ontario

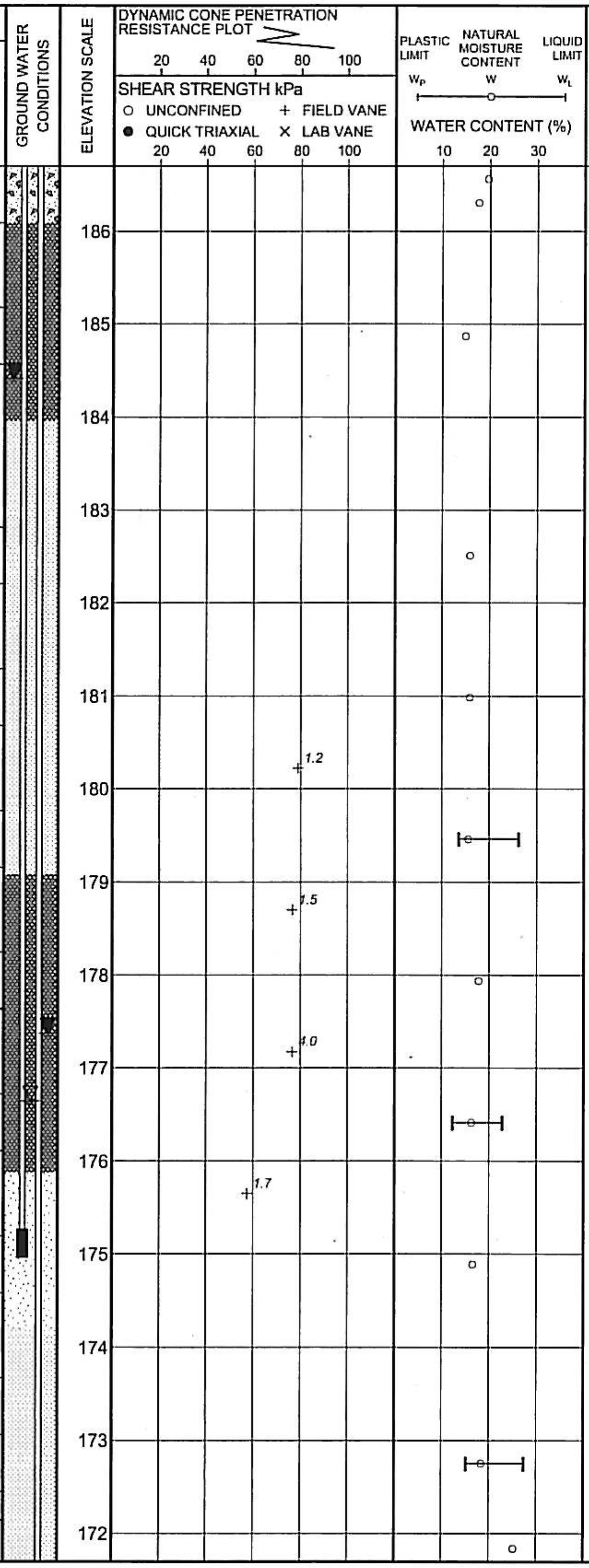
RECORD OF BOREHOLE No 1

1 OF 4 **METRIC**

PROJECT 04-1111-060
G.W.P.
LOCATION 335500E, 4677738N
ORIGINATED BY C.C.
DIST SW Region HWY 401 / 3
BOREHOLE TYPE POWER AUGER,HOLLOW STEM
COMPILED BY T.M.
DATUM Geodetic
DATE November 2, 2006 - November 5, 2006
CHECKED BY SJB

SOIL PROFILE ELEV / DEPTH	DESCRIPTION
186.70	GROUND SURFACE
0.00	TOPSOIL, clayey Firm Black
0.27	SILTY CLAY, some sand, trace gravel, fissured, oxidized along fissures Firm Brown
185.18 1.52	SILTY CLAY, some sand, trace gravel, fissured and oxidized along fissures Very stiff Brown
182.80 3.90	CLAYEY SILT, some sand, trace gravel Stiff Grey

SAMPLES NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL	
1	SS	5						
2	SS	28						
3	SS	15						
4	SS	14						
5	TO	PH	21.1	9	30	36	25	CICU, Oedometer
6	SS	9						
7	TO	PH						
8	SS	11						
9	TO	PH						
10	TO	PH	20.5	2	28	41	29	CICU, Oedometer
11	SS	10						



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

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

Golder Associates
London, Ontario

PROJECT 04-1111-060

RECORD OF BOREHOLE No 1

2 OF 4

METRIC

G.W.P.

LOCATION 335500E, 4677738N

ORIGINATED BY C.C.

DIST SW Region HWY 401 / 3

BOREHOLE TYPE POWER AUGER,HOLLOW STEM

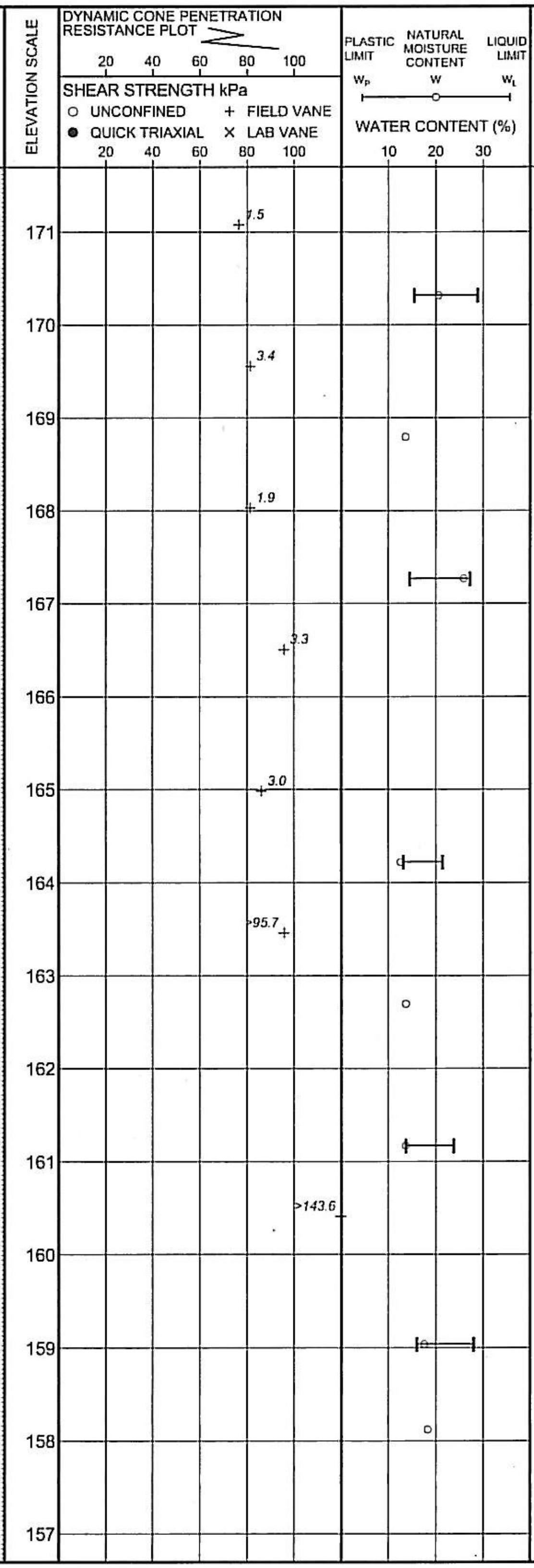
COMPILED BY T.M.

DATUM Geodetic

DATE November 2, 2006 - November 5, 2006

CHECKED BY SSB

SOIL PROFILE		SAMPLES			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES		GR	SA	SI	CL
	CLAYEY SILT, some sand, trace gravel Stiff Grey								
170.61 16.09	CLAYEY SILT, some sand, trace to some gravel Stiff Grey	12	TO	PH					
		13	SS	7					
		14	TO	PH	20.0	3 CICU, Oedometer	23	44	30
166.03 20.67	SILTY CLAY, some sand, trace gravel, laminated Stiff Grey	15	SS	7					
164.51 22.19	CLAYEY SILT, some sand, trace gravel, fine to medium silty sand layers Stiff to hard Grey	16	TO	PH					
		17	SS	43					
		18	TO	PH					
		19	TO	PH					
		20	TO	PH					
		21	SS	19					



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+³, ×³: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

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RECORD OF BOREHOLE No 1

3 OF 4 **METRIC**

PROJECT 04-1111-060
G.W.P.
DIST SW Region HWY 401 / 3
DATUM Geodetic
LOCATION 335500E, 4677738N
BOREHOLE TYPE POWER AUGER,HOLLOW STEM
DATE November 2, 2006 - November 5, 2006
ORIGINATED BY C.C.
COMPILED BY T.M.
CHECKED BY SJB

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)
	CLAYEY SILT, some sand, trace gravel, fine to medium silty sand layers Stiff to hard Grey	22	TO	PH			
		23	TO	PH	156		
		24	SS	PH	155		
154.24 32.46	LIMESTONE, white to light grey					>95.7 +	
153.93 32.77	LIMESTONE, fresh, medium strong, laminated, very fine grained, moderately porous, white to light grey *(FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)*	25	SS	50	154		
		26	NQ RC		153		
		27	NQ RC		152		
		28	NQ RC		151		
		29	NQ RC		150 149		
148.78 37.92	END OF BOREHOLE						

Water level encountered in borehole at about elevation 176.65m during drilling and on completion of drilling October 2, 2006

Lower piezometer 32mm PVC screen and riser pipe. Second (Upper) piezometer installed in immediately adjacent unsampled borehole, 13mm porous tip and CPVC riser pipe.

Water level in Upper Piezometer at about elevation 184.41m on November 14, 2006.

Water level in Lower Piezometer at about elevation 177.37m on November 14, 2006.

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100
PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L
WATER CONTENT (%) 10 20 30
UNIT WEIGHT γ kN/m³
REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

PROJECT: 04-1111-060

RECORD OF DRILLHOLE: 1

SHEET 4 OF 4

LOCATION: 335500E, 4677738N

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 2, 2006 - November 5, 2006

DRILL RIG:

DRILLING CONTRACTOR:

DATUM: Geodetic

JN - Joint
FLT - Fault
SHR - Shear
VN - Vein
CJ - Conjugate
BD - Bedding
FO - Foliation
CO - Contact
OR - Orthogonal
CL - Cleavage
PL - Planar
CU - Curved
UN - Undulating
ST - Stepped
IR - Irregular
PO - Polished
K - Slickensided
SM - Smooth
Ro - Rough
Br - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	PENETRATION RATE (m/min)	COLOUR % RETURN / FLUSH	RECOVERY TOTAL CORE %	RECOVERY SOLID CORE %	R.Q.D. %	FRACT. INDEX PER 0.3	DISCONTINUITY DATA B Angle (Strike)	DISCONTINUITY DATA DIP w.r.t. CORE AXIS	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	ROCK STRENGTH INDEX	WEATHERING INDEX	RMC Q' (avg.)	NOTES WATER LEVELS INSTRUMENTATION
		ROCK SURFACE		154.24														
	MUD ROTARY NW CASING	LIMESTONE, while to light grey		32.40 153.96														
33		LIMESTONE, fresh, medium strong, laminated, very fine grained, moderately porous, whitish grey		32.74 153.48	1									BD,PL,Ro Cl BD,PL,Ro Cl BD,PL,Ro Cl				
	DIAMOND DRILLING NQ ROCK CORE	LIMESTONE, fresh, medium strong, thinly laminated, fine grained, non-faintly porous, bluish grey		33.22 152.79										BD,UN,Ro Cl BD,IR,VR Si BD,CU,SM Cl				
34 35		LIMESTONE, fresh, medium strong, laminated, very fine grained, faintly to moderately porous, whitish grey, fossils present		33.91 151.65	2									,IR,Ro Cl				
36		LIMESTONE, fresh, medium strong, thinly laminated, very fine grained, faintly to moderately porous, light brownish grey highly porous, fossiliferous from 35.7m to 36.2m depth.		35.05 150.05	3													
37		LIMESTONE, fresh, medium strong, thinly laminated, very fine grained, faintly to moderately porous, light brownish grey vuggy, fossiliferous at 37.2m and 37.5m depths		36.65 148.78	4									BD,IR,Ro Cl				
38		END OF BOREHOLE		37.92														
39																		
40																		
41																		
42																		
43																		
44																		
45																		
46																		
47																		

MISS-ROCK-2 04-1111-060.GPJ GAL-CANADA.GDT 6/14/07 DATA INPUT: T.M.

DEPTH SCALE
1 : 75

Golder Associates

LOGGED: C.C.
CHECKED: SJB

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 7

1 OF 4 **METRIC**

PROJECT 04-1111-060
G.W.P.
DIST SW Region HWY 401 / 3
DATUM Geodetic
LOCATION 333325E, 4678848N
BOREHOLE TYPE POWER AUGER,HOLLOW STEM
DATE November 10, 2006 - November 16, 2006
ORIGINATED BY C.C.
COMPILED BY T.M.
CHECKED BY SJB

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
183.17	GROUND SURFACE
0.00	TOPSOIL, clayey Firm Black
0.25	CLAYEY SILT , some sand, trace gravel Stiff Mottled brown and grey
181.65 1.52	CLAYEY SILT , some sand, trace gravel, oxidized fissures Very stiff to hard Brown
179.28 3.89	CLAYEY SILT , some sand, trace gravel, silt and sandy silt pockets and laminated zones Firm to very stiff Grey

SAMPLES

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL	REMARKS
1	SS	10						
2	SS	23						
3	SS	59						
4	SS	32						
5	TO	PH						
6	SS	11						
7	TO	PH	21.4	1	30	39	30	CICU, Oedometer
8	SS	12						
9	TO	PH						
10	SS	8						
11	TO	PH	20.7	3	19	42	36	

GROUND WATER CONDITIONS

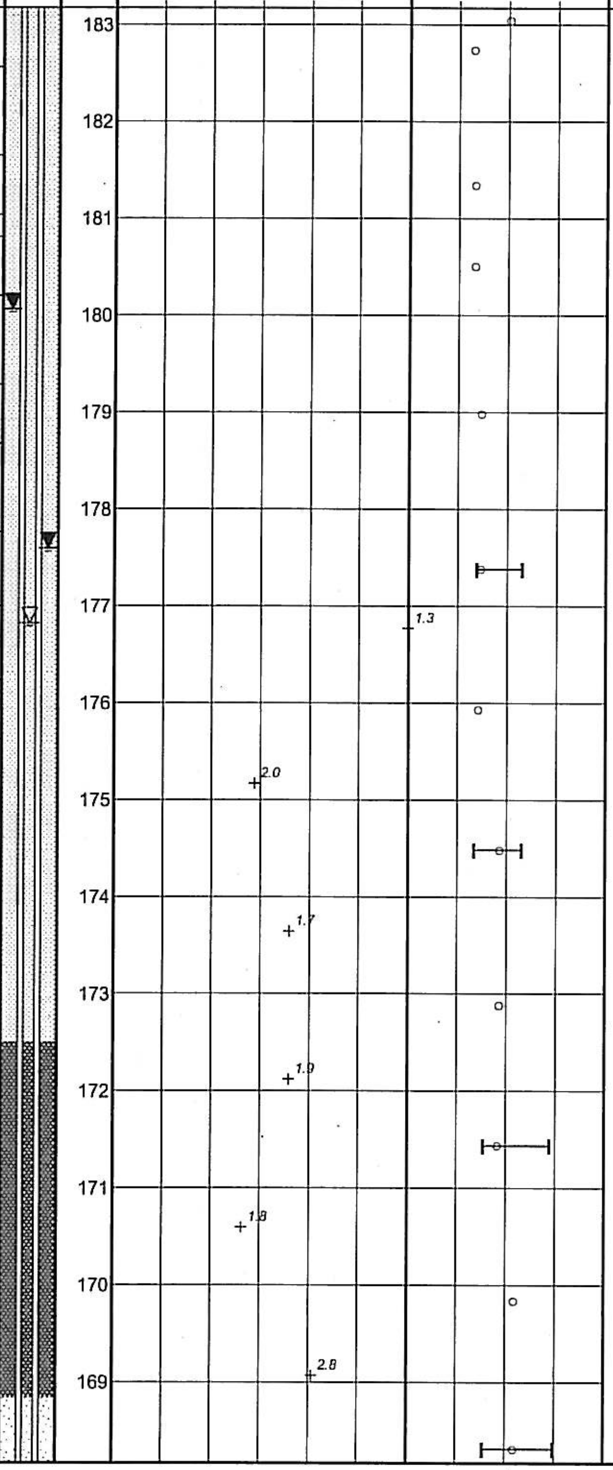
ELEVATION SCALE

DYNAMIC CONE PENETRATION RESISTANCE PLOT
20 40 60 80 100

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

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%)
10 20 30



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

RECORD OF BOREHOLE No 7

2 OF 4

METRIC

PROJECT 04-1111-060
G.W.P.
DIST SW Region HWY 401 / 3
DATUM Geodetic

LOCATION 333325E, 4678848N
BOREHOLE TYPE POWER AUGER,HOLLOW STEM
DATE November 10, 2006 - November 16, 2006

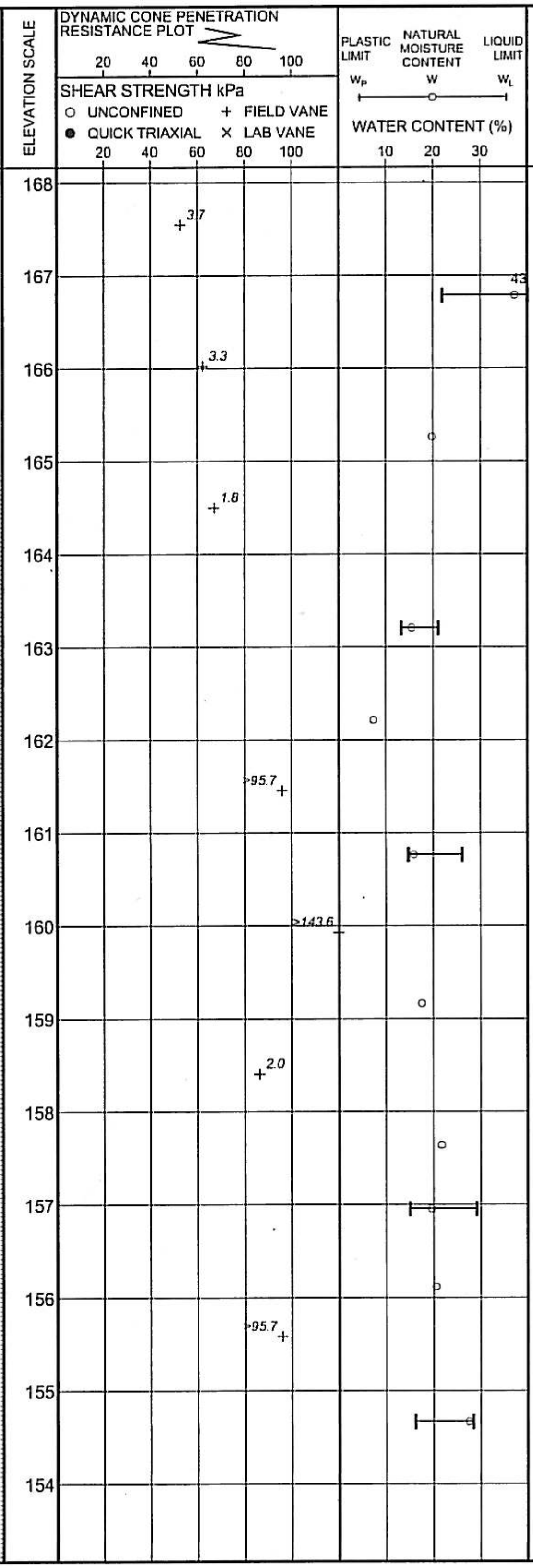
ORIGINATED BY C.C.
COMPILED BY T.M.
CHECKED BY SSB

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
	CLAYEY SILT , some sand, trace gravel, silt and sandy silt pockets and laminated zones Firm to very stiff Grey

SAMPLES

NUMBER	TYPE	"N" VALUES
12	TO	PH
13	SS	12
14	TO	PH
15	TO	PH
16	SS	21
17	SS	PH
18	SS	13
19	SS	12
20	TO	PH
21	SS	9
22	SS	PH



ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
168		CICU, Oedometer
157	21.0	2 19 42 37 CICU, Oedometer

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

○³% STRAIN AT FAILURE

RECORD OF BOREHOLE No 7

3 OF 4 **METRIC**

PROJECT 04-1111-060

G.W.P.

LOCATION 333325E, 4678848N

ORIGINATED BY C.C.

DIST SW Region HWY 401 / 3

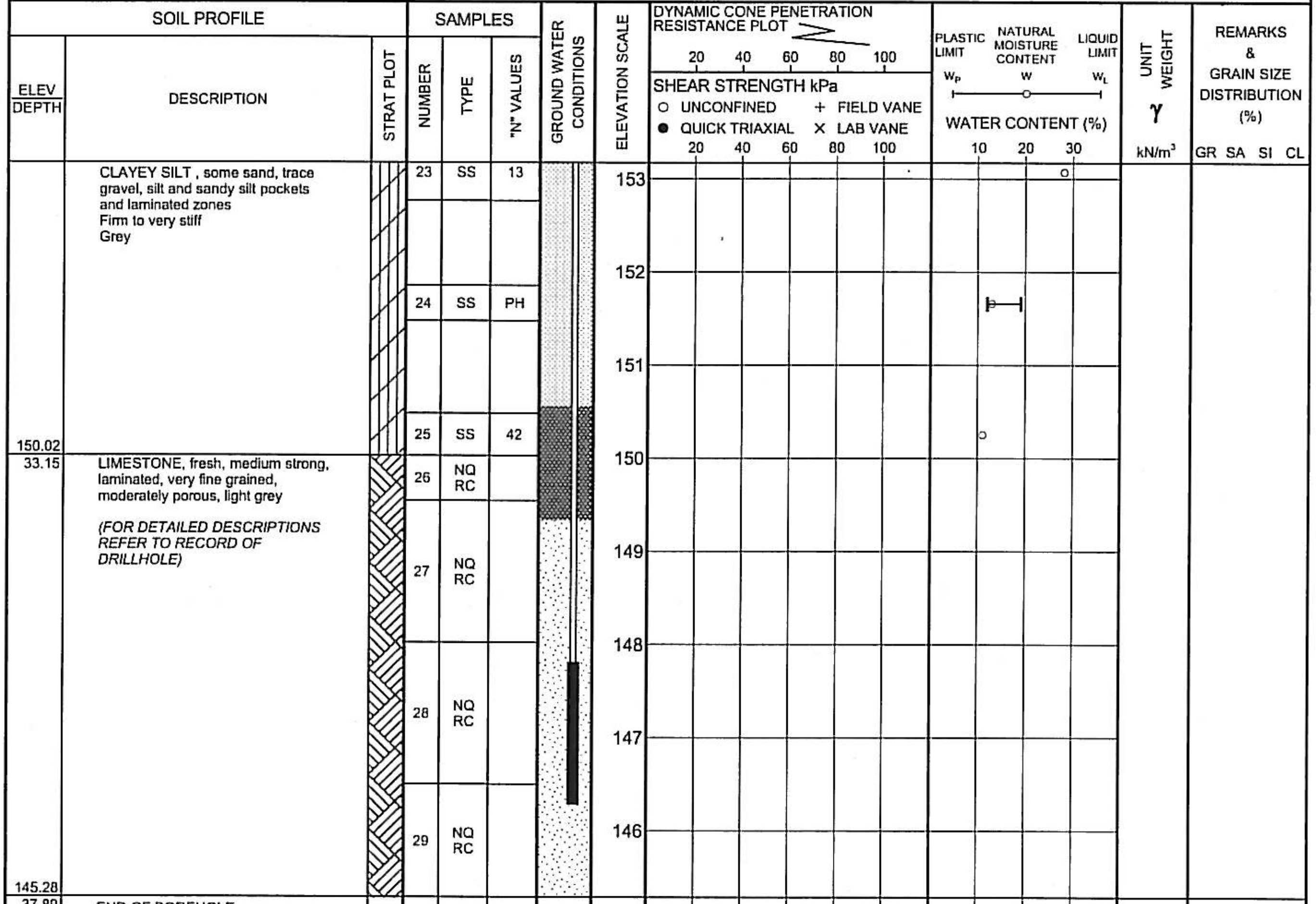
BOREHOLE TYPE POWER AUGER,HOLLOW STEM

COMPILED BY T.M.

DATUM Geodetic

DATE November 10, 2006 - November 16, 2006

CHECKED BY SJB



ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES
	CLAYEY SILT , some sand, trace gravel, silt and sandy silt pockets and laminated zones Firm to very stiff Grey	23	SS	13
		24	SS	PH
		25	SS	42
150.02 33.15	LIMESTONE, fresh, medium strong, laminated, very fine grained, moderately porous, light grey *(FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)*	26	NQ RC	
		27	NQ RC	
		28	NQ RC	
		29	NQ RC	
145.28 37.89	END OF BOREHOLE			

Water level in borehole at about elevation 176.82m on October 16, 2006

Lower piezometer 32mm PVC screen and riser pipe. Second (Upper) piezometer 13mm porous tip and CPVC riser pipe.

Water level in Upper Piezometer at about elevation 180.06m on November 14, 2006.

Water level in Lower Piezometer at about elevation 177.59m on November 14, 2006.

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

PROJECT: 04-1111-060

RECORD OF DRILLHOLE: 7

SHEET 4 OF 4

LOCATION: 333325E, 4678848N

DRILLING DATE: November 10, 2006 - November 16, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

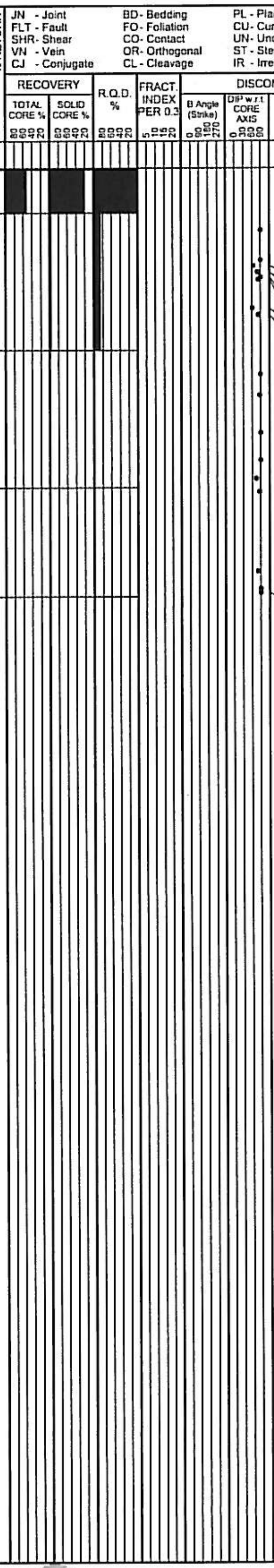
DRILL RIG:

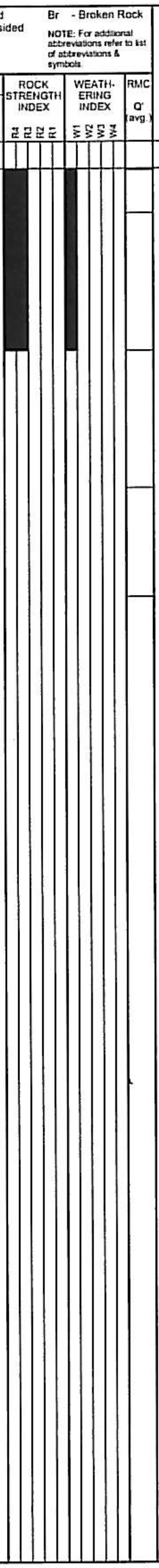
DRILLING CONTRACTOR:

JN - Joint
FLT - Fault
SHR - Shear
VN - Vein
CJ - Conjugate
BD - Bedding
FO - Foliation
CO - Contact
OR - Orthogonal
CL - Cleavage
PL - Planar
CU - Curved
UN - Undulating
ST - Stepped
IR - Irregular
PO - Polished
K - Slickensided
SM - Smooth
Ro - Rough
Br - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION
		ROCK SURFACE	150.04 33.13		
	MUD ROTARY NW CASING	LIMESTONE, fresh, medium strong, laminated, very fine grained, faintly to moderately porous, light grey	149.64 33.53	1	
34 35	DIAMOND DRILLING NQ ROCK CORE	LIMESTONE, fresh, medium strong, laminated to thinly laminated, very fine grained, faintly to moderately porous, light grey	148.02 35.15	2	BD,UN,SM Cl ,IR,SM Cl JN,PL,SM Cl BD,CU,SM Cl BD,IR,VR Cl BD,PL,SM Cl JN,PL,Ro Cl BD,CU,Ro Cl
36 37		LIMESTONE, fresh, medium strong, laminated to thinly laminated, very fine grained, faintly to moderately porous, light grey Moderately yo highly porous from 35.4m to 35.8m and from 36.5m to 36.9m depths.	145.28 37.89	3 4	BD,UN,SM Cl BD,PL,Ro Cl ,PL,Ro Cl BD,CU,SM Cl JN,UN,Ro Cl BD,UN,Ro Cl ,IR,VR Cl BD,IR,SM Cl BD,CU,SM Cl
38		END OF BOREHOLE			
39					
40					
41					
42					
43					
44					
45					
46					
47					
48					





NOTES
WATER LEVELS
INSTRUMENTATION

DEPTH SCALE
1 : 75

Golder Associates

LOGGED: C.C.
CHECKED: SJB

Golder Associates
London, Ontario

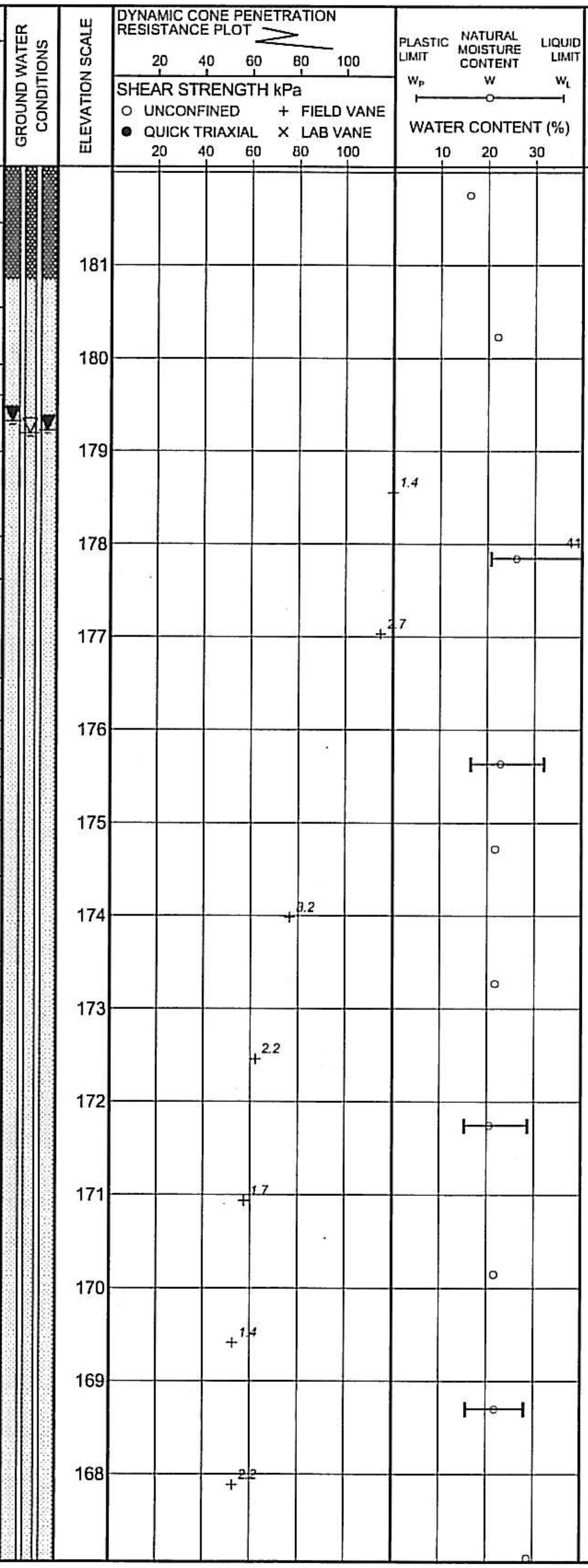
RECORD OF BOREHOLE No 14

1 OF 4 **METRIC**

PROJECT 04-1111-060
G.W.P.
LOCATION 331648E, 4680648N
ORIGINATED BY C.C.
DIST SW Region HWY 401 / 3
BOREHOLE TYPE POWER AUGER,HOLLOW STEM
COMPILED BY T.M.
DATUM Geodetic
DATE November 18, 2006 - November 23, 2006
CHECKED BY SJB

ELEV / DEPTH	DESCRIPTION
182.06	GROUND SURFACE
0.00	FILL, silty clay, pockets of crushed gravel, topsoil, sand partings, glass fragments Stiff Brown to grey
180.99 / 1.07	FILL, silty sand and gravel Brown
180.69 / 1.37	CLAYEY SILT, laminated, some sand Soft to stiff Brown and grey,
	SILTY CLAY, some sand, trace gravel Soft to Stiff Brown and Grey
176.57 / 5.49	CLAYEY SILT, some sand, trace gravel Firm to stiff Grey

SAMPLES NUMBER	TYPE	"N" VALUES
1	SS	13
2	SS	4
3	SS	14
4	TO	PH
5	TO	PH
6	TO	PH
7	SS	7
8	TO	PH
9	TO	PH
10	SS	WR
11	TO	PH



UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
21.0	3	26	41	30
	CICU, Oedometer			
20.2	2	23	39	36
	CICU, Oedometer			

Continued Next Page

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

Golder Associates
London, Ontario

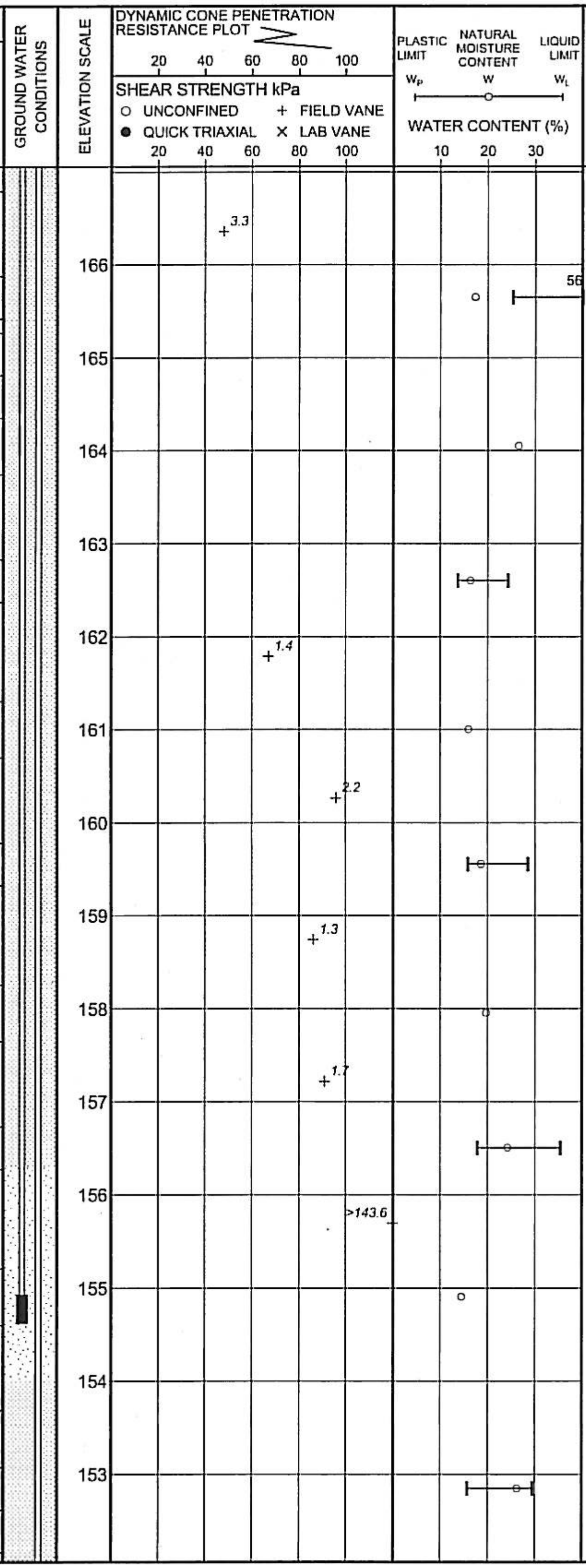
RECORD OF BOREHOLE No 14

2 OF 4 **METRIC**

PROJECT 04-1111-060
G.W.P.
LOCATION 331648E, 4680648N
ORIGINATED BY C.C.
DIST SW Region HWY 401 / 3
BOREHOLE TYPE POWER AUGER,HOLLOW STEM
COMPILED BY T.M.
DATUM Geodetic
DATE November 18, 2006 - November 23, 2006
CHECKED BY SJB

ELEV DEPTH	DESCRIPTION
	SILTY CLAY, some sand, trace gravel Firm to Stiff Grey
	CLAYEY SILT, some sand, trace gravel Firm to Stiff Grey
	SILTY CLAY, some sand, trace gravel Firm to Stiff Grey
155.24 26.82	SILTY SAND, trace gravel Compact Grey
153.10 28.96	CLAYEY SILT, some sand, trace gravel Stiff Grey

NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
12	SS	9		
13	TO	PH	21.4	7 24 44 25 CICU, Oedometer
14	TO	PH		
15	SS			
16	TO	PH		
17	SS	11		
18	TO	PH		
19	SS	5		
20	TO	PH		
21	SS	16		
22	TO	PH		
23	TO	PH		



Continued Next Page

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

LDN_MTO_2006 04-1111-060.GPJ GLDR_LON.GDT 6/14/07

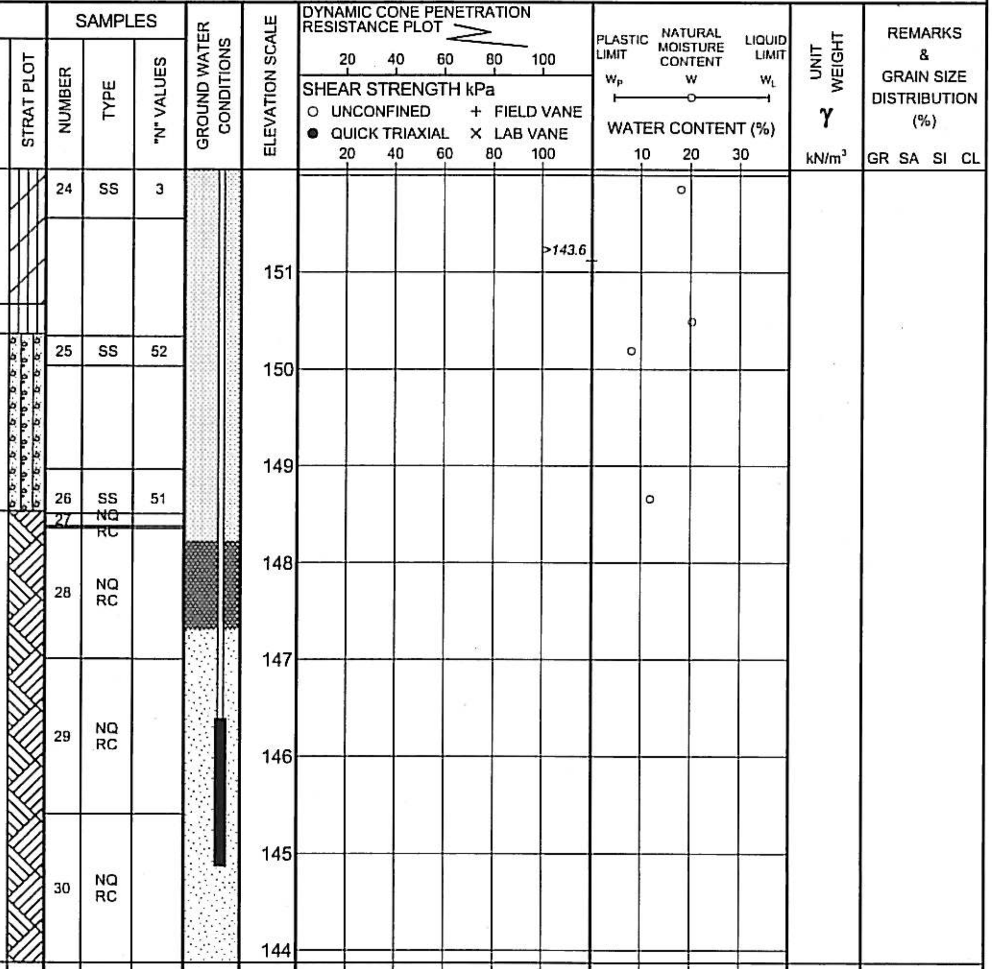
Golder Associates
London, Ontario

RECORD OF BOREHOLE No 14

3 OF 4 **METRIC**

PROJECT 04-1111-060
G.W.P.
DIST SW Region HWY 401 / 3
DATUM Geodetic
LOCATION 331648E, 4680648N
BOREHOLE TYPE POWER AUGER,HOLLOW STEM
DATE November 18, 2006 - November 23, 2006
ORIGINATED BY C.C.
COMPILED BY T.M.
CHECKED BY SJB

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES
	CLAYEY SILT, some sand, trace gravel Stiff Grey	24	SS	3
150.67 31.39	SILT, Very dense Grey			
150.36 31.70	SILTY SAND AND GRAVEL Very dense Grey/ black	25	SS	52
		26	SS	51
148.53 33.53	DOLOSTONE, fresh to slightly weathered, medium strong, thinly laminated, fine to medium grained, moderately porous, interbedded, brown and dark brown changing to DOLOMITIC LIMESTONE, brown and black changing to LIMESTONE, light and dark grey *(FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)*	27	NQ RC	
		28	NQ RC	
		29	NQ RC	
		30	NQ RC	
143.87 38.19	END OF BOREHOLE			



Water level in borehole at about elevation 179.19m during drilling on October 23, 2006

Lower piezometer 32mm PVC screen and riser pipe. Second (Upper) piezometer 13mm porous tip and CPVC riser pipe.

Water level in Upper Piezometer at about elevation 179.32m on November 14, 2006.

Water level in Lower Piezometer at about elevation 179.22m on November 14, 2006.

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

PROJECT: 04-1111-060

RECORD OF DRILLHOLE: 14

SHEET 4 OF 4

LOCATION: 331648E, 4680648N

DRILLING DATE: November 18, 2006 - November 23, 2006

DATUM: Geodetic

DRILL RIG:

INCLINATION: -90° AZIMUTH: —

DRILLING CONTRACTOR:

JN - Joint
FLT - Fault
SHR - Shear
VN - Vein
CJ - Conjugate
BD - Bedding
FO - Foliation
CO - Contact
OR - Orthogonal
CL - Cleavage
PL - Planar
CU - Curved
UN - Undulating
ST - Stepped
IR - Irregular
PO - Polished
K - Slickensided
SM - Smooth
Ro - Rough
Br - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	DISCONTINUITY DATA: TYPE AND SURFACE DESCRIPTION
		ROCK SURFACE	148.53		
34	DIAMOND DRILLING NQ ROCK CORE	DOLOSTONE, fresh to slightly weathered, medium strong, thinly laminated, fine to medium grained, moderately porous, interbedded brown and dark brown	33.53	1	JN,CU,SM Cl BD,PL,Ro Cl BD,PL,SM Cl BD,PL,SM Cl
35		DOLOSTONE/LIMESTONE, fresh to slightly weathered, medium strong, thinly laminated, fine grained, faintly to moderately porous, interbedded light brown and black	147.01 35.05	2	BD,IR,Ro Cl BD,UN,SM Cl ,IR,SM Cl VN,IR,VR Ca
36		LIMESTONE, fresh, medium strong, thinly laminated, fine grained, moderately to highly porous, light and dark grey mottled	145.89 36.17	3	BD,IR,VR Cl BD,IR,Ro Cl JN,UN,Ro Cl BD,UN,SM Cl ,IR,Ro Cl
37		LIMESTONE, fresh to slightly weathered, medium strong, thinly laminated, fine grained, highly to faintly porous, interbedded brown and white	144.70 37.36	4	
38		END OF BOREHOLE	143.87 38.19		

Other column headings on the form: SYMBOLIC LOG; PENETRATION RATE (m/min); COLOUR / % RETURN; FLUSH; RECOVERY (TOTAL CORE %, SOLID CORE %; 80 60 40 20); R.Q.D. % (80 60 40 20); FRACT. INDEX PER 0.3 (5 10 15 20); B Angle (Strike) (0 90 180 270); DIP w.r.t. CORE AXIS (0 30 60 90); ROCK STRENGTH INDEX (R4 R3 R2 R1); WEATH-ERING INDEX (W1 W2 W3 W4); RMC Q' (avg.); NOTES WATER LEVELS INSTRUMENTATION

39
40
41
42
43
44
45
46
47
48

DEPTH SCALE
1 : 75

LOGGED: C.C.
CHECKED: SJB

Golder Associates
London, Ontario

PROJECT 04-1111-060

RECORD OF BOREHOLE No 23

1 OF 3

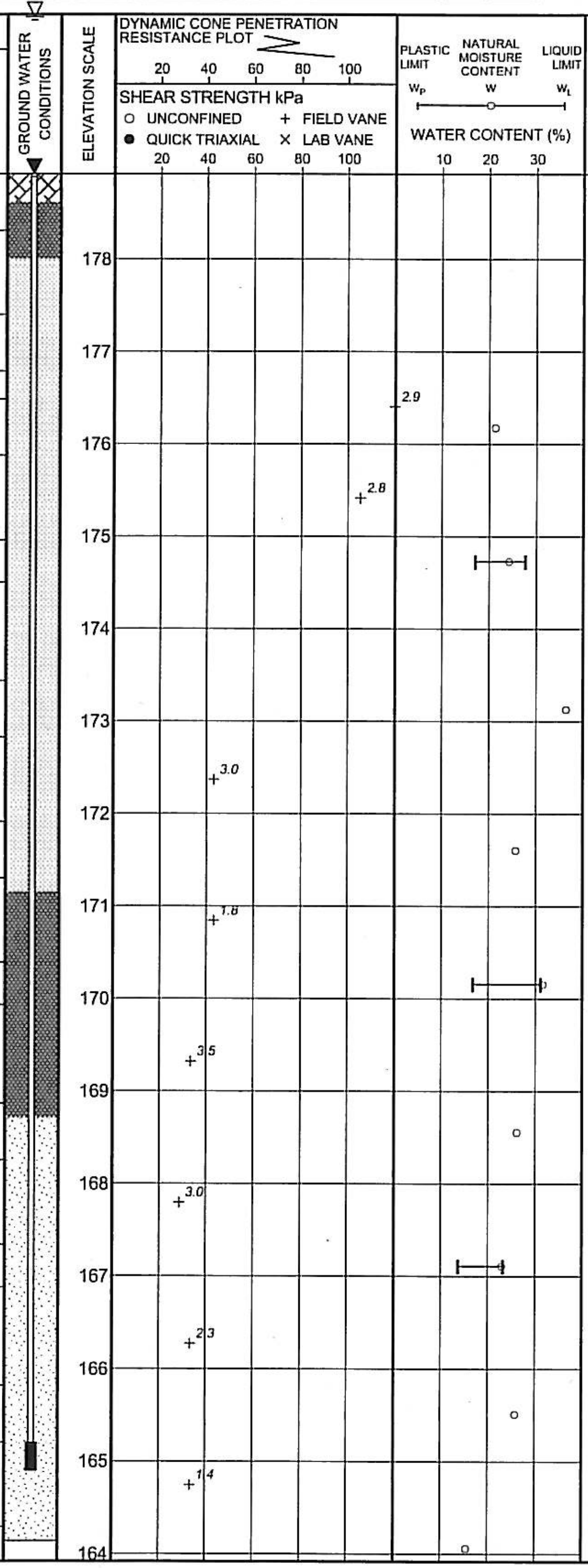
METRIC

G.W.P. ______ LOCATION 328529E, 4682323N ORIGINATED BY C.C.

DIST SW Region HWY 401 / 3 BOREHOLE TYPE POWER AUGER,HOLLOW STEM COMPILED BY T.M.

DATUM Geodetic DATE November 24, 2006 - November 26, 2006 CHECKED BY SJB

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
178.92	GROUND SURFACE				
0.00	FILL, silty sand, with gravel and silt pockets Brown	1	AS	-	
178.62 0.30	TOPSOIL, silty sand Black				
178.31 0.61	SANDY SILT, Compact Brown to grey,				
177.24 1.68	CLAYEY SILT, laminated, some sand, numerous silt and sandy silt partings Stiff to very stiff Grey	2	SS	15	
		3	SS	6	
		4	TO	PH	
174.50 4.42	CLAYEY SILT, some sand, trace gravel, silty sand pockets Soft to very stiff Grey	5	SS	4	
		6	SS	5	
		7	TO	PH	1 23 37 39 CICU, Oedometer
		8	SS	1	
		9	TO	PH	
		10	SS	3	
		11	TO	PH	



Continued Next Page

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

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

Golder Associates
London, Ontario

RECORD OF BOREHOLE No 23

2 OF 3

METRIC

PROJECT 04-1111-060
G.W.P.
DIST SW Region HWY 401 / 3
DATUM Geodetic
LOCATION 328529E, 4682323N
BOREHOLE TYPE POWER AUGER,HOLLOW STEM
DATE November 24, 2006 - November 26, 2006
ORIGINATED BY C.C.
COMPILED BY T.M.
CHECKED BY SJB

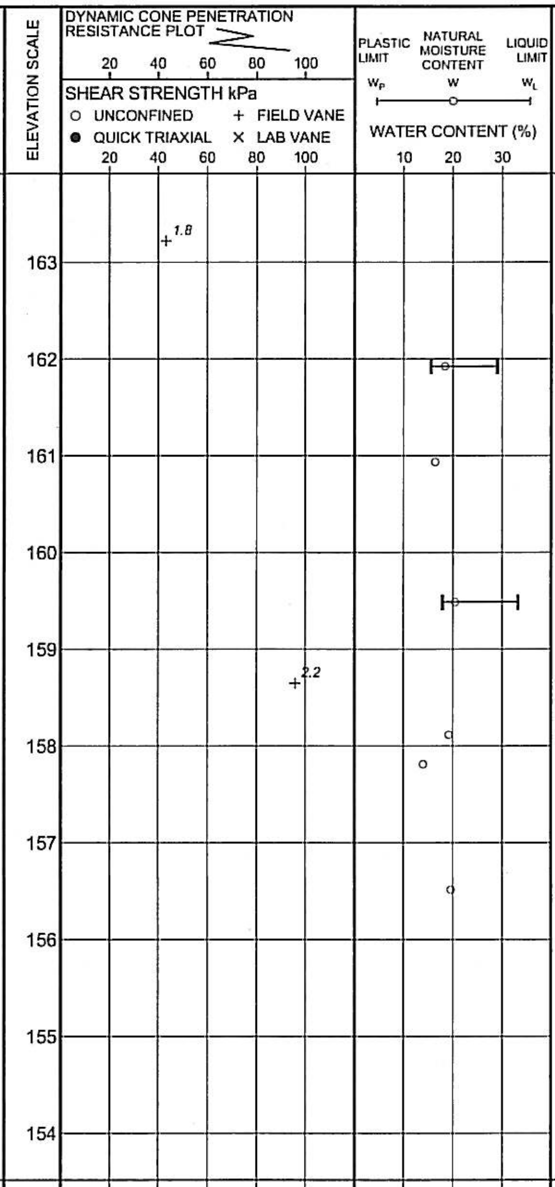
ELEV DEPTH	DESCRIPTION
	CLAYEY SILT, some sand, trace gravel, silty sand pockets Soft to very stiff Grey
158.19 20.73	SILT Compact Grey
20.88	CLAYEY SILT, some sand, trace gravel Firm Grey
156.36 22.56	LIMESTONE, fresh to slightly weathered, medium strong, light grey to brown *(FOR DETAILED DESCRIPTIONS REFER TO RECORD OF DRILLHOLE)*
153.52 25.40	END OF BOREHOLE

Water level in borehole at about elevation 180.59m during drilling on October 26, 2006.

Artesian water flow during rock coring, measured at 1.67m above ground surface. Hydrogen sulfide odour, borehole sealed with grout, piezometer installed at 14.8m depth in imdelately adjacent unsampled borehole.

Water level in Piezometer at about elevation 178.92m on November 14, 2006.

SAMPLES NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
12	TO	PH		
13	TO	PH	21.2	2 20 44 34 CICU, Oedometer
14	SS	18		
15	TO	PH	21.6	4 16 40 40 CICU, Oedometer
16	SS	20		
17	TO	PH		
18	WS			
19	NQ RC			
20	NQ RC			



+3, ×3: Numbers refer to Sensitivity

O3% STRAIN AT FAILURE

PROJECT: 04-1111-060

RECORD OF DRILLHOLE: 23

SHEET 3 OF 3

LOCATION: 328529E, 4682323N

DRILLING DATE: November 24, 2006 - November 26, 2006

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: —

DRILL RIG:

DRILLING CONTRACTOR:

JN - Joint	BD - Bedding	PL - Planar	PO - Polished
FLT - Fault	FO - Foliation	CU - Curved	K - Slickensided
SHR - Shear	CO - Contact	UN - Undulating	SM - Smooth
VN - Vein	OR - Orthogonal	ST - Stepped	Ro - Rough
CJ - Conjugate	CL - Cleavage	IR - Irregular	

Br - Broken Rock

NOTE: For additional abbreviations refer to list of abbreviations & symbols.

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	DISCONTINUITY DATA TYPE AND SURFACE DESCRIPTION	NOTES WATER LEVELS INSTRUMENTATION
		ROCK SURFACE	156.36			
23	MUD ROTARY NW CASING	LIMESTONE, fresh to slightly weathered, medium strong, thinly laminated, fine to medium grained, faintly porous, greyish brown - Broken section (to 23.83m) contains mica and quartz crystals, bitumen/hydrocarbon odour	22.56	1		
24	DIAMOND DRILLING NQ ROCK CORE	LIMESTONE, freshly weathered, medium strong, thinly laminated, fine grained, faintly porous, light greywith healed black jagged seams and whits pacthes, fossiliferous	154.87 24.05	2	,IR,Ro CI ,IR,Ro CI BD,CU,SM CI BD,IR,Ro CI BD,PL,SM CI	
25		END OF BOREHOLE	153.52 25.40			
26						
27						
28						
29						
30						
31						
32						
33						
34						
35						
36						
37						

MISS-ROCK-2 04-1111-060.GPJ GAL-CANADA.GDT 6/14/07 DATA INPUT: T.M.

DEPTH SCALE

1 : 75

Golder Associates

LOGGED: C.C.

CHECKED: SJG

TABLE 1

PRELIMINARY COMPARISON OF FEASIBLE FOUNDATION ALTERNATIVES

Detroit River International Crossing –
Bridge Approach Corridor[1]

Foundation Option	***Feasibility***	***Advantages***	***Disadvantages***	***Constructability / Practicality***	***Relative Costs***
Spread footings on native soil	• Feasible for support of lightly loaded foundations	• Potentially easier and faster construction	• Lower geotechnical resistance values and probably not suitable for bridge foundations • Greater potential for differential settlement due to subsurface variability..	• Conventional excavation and construction techniques • In some cases, dewatering measures may be required	• Foundation costs less expensive than deep foundations.
Steel H-piles driven to found on bedrock	• Feasible for support of bridge foundations	• Minimize differential settlement • Higher geotechnical resistance than shallow foundations • Readily installed	• Bridge foundation piles may have to accommodate downdrag loads in areas of high fill, unless adequate settlement time is allowed for, wick drains are installed, preloading and/or surcharging or some combination of these options is carried out • Vibrations and noise may be a concern in some areas immediately near structures or residences • Greater potential for seepage of artesian water along pile-soil interface if pile tip reinforcement is used – special measures required to prevent loss of fine soil particles by means of filter blanket and drainage	• Conventional construction methods for H-pile foundations.	• Less expensive than caissons (drilled shafts).
Caissons (drilled shafts) bored to found on or socketed into bedrock	• Feasible for support of bridge foundations	• Minimize differential settlement • Higher bearing resistances than for steel H-piles	• Temporary or permanent liner required due to soil conditions • Groundwater control may be problematic in areas of artesian pressure • Bridge foundation piles may have to accommodate downdrag loads in areas of high fill, unless adequate settlement time is allowed for, wick drains are installed, preloading and/or surcharging or some combination of these options is carried out	• Conventional construction methods with the exception of groundwater control issues • Drilling with slurry will likely be required • Potential for encountering hydrogen sulphide gas during construction, requiring subsequent gas control measures	• More expensive than steel H-piles,

NOTE: 1. Table should not be used without reference to the corresponding sections of the report. The details and the complexities of each system with respect to site specific issues are not addressed in this comparison table. Relative costs will vary depending on scope of selected construction method as mobilization costs may be a significant proportion of the total costs.

TABLE 2

PRELIMINARY COMPARISON OF FEASIBLE RETAINING STRUCTURE ALTERNATIVES

Detroit River International Crossing –
Bridge Approach Corridor[1]

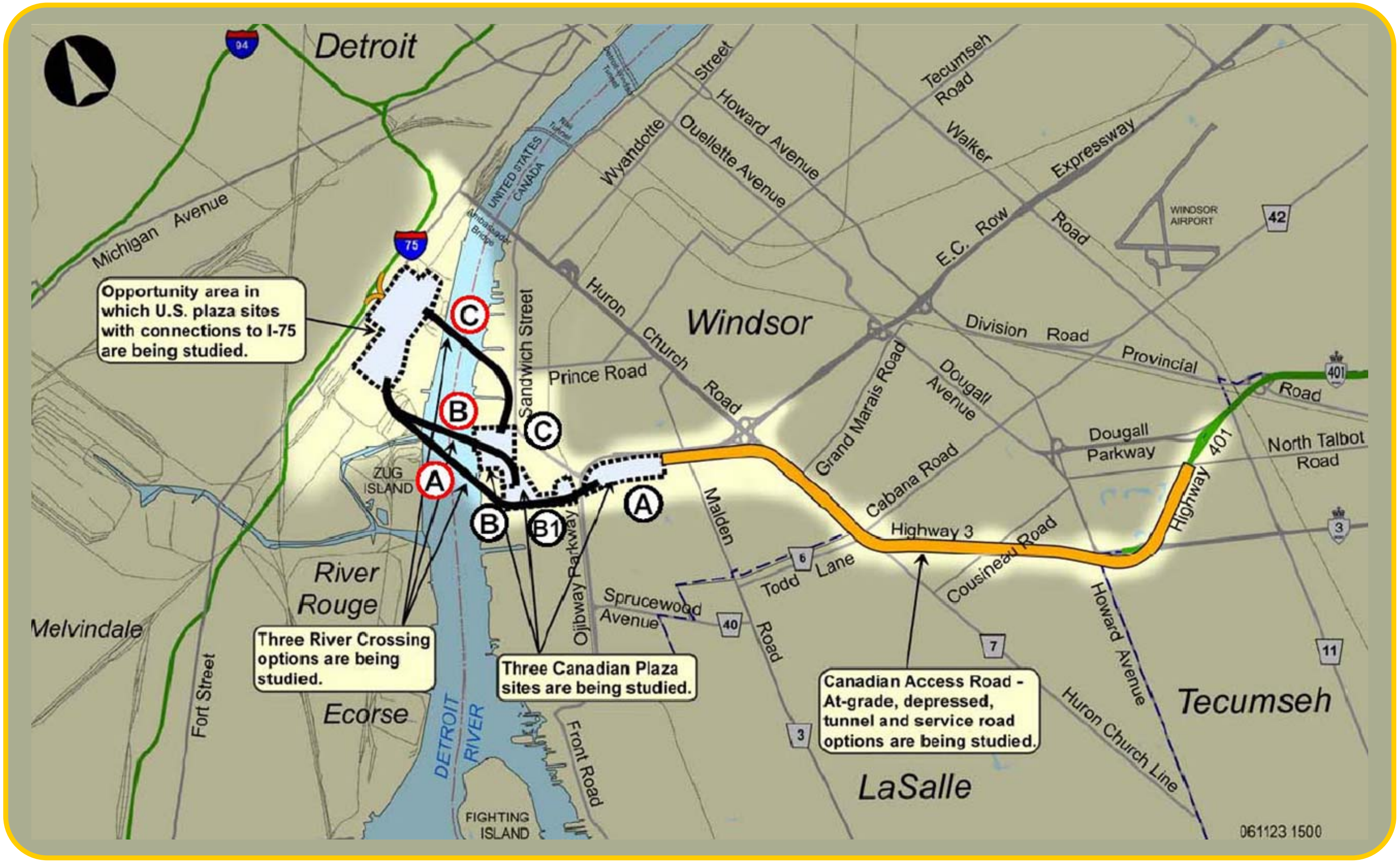
Retaining Structure Option	*Feasibility*	*Advantages*	*Disadvantages*	*Constructability / Practicality*	*Relative Costs*
GRAVITY WALL SYSTEMS					
Cast-in-Place Concrete Wall Supported by Shallow Foundations	• Feasible for low-height walls (< 5m high)	• Conventional construction methods • Can be constructed in relatively small sections • Design geometry can be readily adapted to site specific requirements	• Potential for differential settlement for walls supported on shallow foundations • Moderate to high walls (greater than 5 to 6 m high) will require deep foundations • Temporary cut slopes and granular backfill required • Temporary excavation support may be required • Temporary right of way or easement may be required for cut slopes	• Conventional construction techniques	• More expensive than pre-cast or MSE systems for walls above 4 to 5 m high
Pre-cast "bin" or Crib Wall on Shallow Foundations	• Feasible for low-height walls (<5m high)	• Conventional construction methods • Faster construction than cast-in-place wall • Variety of precast systems available to suit cost and aesthetic concerns	• Potential for differential settlement for walls supported by shallow foundations • Moderate to high walls (greater than 5 to 6 m high) will require deep foundations • Temporary cut slopes and granular backfill required • Temporary shoring may be required • Temporary right of way or easement may be required for cut slopes	• Conventional construction techniques	• May be less expensive than cast in place construction depending on geometry, backfill and temporary construction requirements
Mechanically Stabilized Earth Wall on Shallow Foundation	• Feasible for low-height walls (<5m high)	• Conventional construction methods • Ease of construction compared to cast-in-place wall or "bin" or "crib" walls • Variety of precast panel wall facings to suit cost and aesthetic concerns • Some wall types can have a relatively high load carrying capability for both static and dynamic loads • May be constructed as geogrid-reinforced earth with sloped (1:1 or steeper) and provided with vegetated or shot-crete facing (similar to reinforced earth slope)	• Potential for differential settlement • Moderate to high walls, where deep foundations would be required, are not suitable for MSE walls (RSS) • Temporary cut slopes or shoring and backfill required • Select backfill is required • Temporary right of way or easement may be required for cut slopes • Stabilized earth mass will contain reinforcement elements that may inhibit future underground construction behind wall • Specialized design may be required for sloped and vegetated systems	• Conventional construction techniques	• May be less expensive than cast in place construction depending on backfill and temporary construction requirements
IN SITU WALL SYSTEMS					
Soil Nail Wall	• Feasible for low-height walls (<5m high)	• Little space behind wall face required for construction (less than for other in-situ wall types) • Wall may be constructed as excavation proceeds to its full height (depth) or in steps for higher walls	• Limited industry experience in Ontario • May require additional right of way for nails (beyond face of wall) • Frost protection treatments may be required	• Limited industry experience in south western Ontario	• May be the lowest cost retaining system
Soldier-Piles and Wood Lagging Wall	• Feasible for temporary walls of moderate height	• Relatively rapid construction of temporary wall • Frequently used in Ontario • Adaptable to different horizontal support systems and horizontal geometry • Construction may operate mostly from within wall perimeter lines (may be constructed close to right of way limits) • May be used for permanent wall if lagging constructed as pre-cast concrete panels	• Not suitable for soft soils at depths that might induce squeezing of ground before installation of lagging can be completed • This flexible wall system may not be suitable near displacement-sensitive utilities, roads, or buildings • Driving of temporary casings (if needed) or soldier piles may induce unwanted vibrations • Pre-drilling required in bouldery soils • Pre-drilling for piles may require slurry in some areas • Frost protection treatment at wall face may be required • Permanent wall systems will require architectural facing (e.g. concrete panels), and detailed frost and corrosion protection systems • Can not prevent seepage through wall	• Conventional construction techniques	• One of least expensive in situ retaining system
Sheet Pile Wall	• Feasible for most wall heights, where deflection is not a concern	• Relatively rapid construction of temporary wall • Adaptable to different horizontal support systems and horizontal geometry • Construction may operate mostly from within wall perimeter lines (may be constructed close to right of way limits) • May also be used for permanent wall • Seepage through wall can be minimized	• This flexible wall system may not be suitable near displacement-sensitive utilities, roads, or buildings • Driving of piles may induce unwanted vibrations • Frost protection treatment may be required for temporary wall • Permanent wall systems may require architectural facing and will require fully designed frost and corrosion protection system	• Conventional construction techniques	• One of least expensive temporary retaining systems

TABLE 2

PRELIMINARY COMPARISON OF FEASIBLE RETAINING STRUCTURE ALTERNATIVES
(Cont'd)

Retaining Structure Option	*Feasibility*	*Advantages*	*Disadvantages*	*Constructability / Practicality*	*Relative Costs*
Secant Pile Wall	• Feasible for most wall heights	• Adaptable to different horizontal support systems and horizontal geometry • Construction may operate from within wall perimeter lines (may be constructed close to right of way limits) • May be used for permanent wall • Relatively high stiffness walls can be constructed to control displacements • Seepage through wall can be minimized	• Driving of temporary casings (if needed) may induce unwanted vibrations • Pre-drilling for piles may require slurry and/or dewatering in some areas • Permanent wall systems will require architectural facing, and detailed frost protection systems • Slower construction than for soldier-pile and lagging or sheet pile temporary walls • Achieving uniform face for permanent wall requires trimming of piles or installation of architectural facing	• Conventional construction techniques	• More expensive than gravity walls for permanent wall construction • More expensive than soldier-pile and lagging or sheet pile walls (may be on the order of twice as expensive as these walls) • May be uneconomical for low height walls
Concrete Diaphragm Wall	• Feasible for most wall heights	• Adaptable to different horizontal support systems and horizontal geometry • Construction may operate mostly from within wall perimeter lines (may be constructed close to right of way limits) • May be used for permanent wall • Relatively high stiffness walls can be constructed to control displacements, stiffness can be greater than for secant pile walls • Seepage through wall can be minimized • May be more rapidly constructed than secant pile wall	• Slurry management systems required • Permanent wall systems will require architectural facing, and detailed frost protection and seepage draingage systems • Slower construction than for soldier-pile and lagging or sheet pile temporary walls	• Specialized construction techniques • Limited industry experience in Ontario	• Expensive temporary support system, may have economic advantages if wall can be integrated into permanent structure • Uneconomical for low height walls
Soil Mix Wall	• May not be feasible	• Relatively rapid to construct compared to secant pile and concrete diaphragm walls	• Limited industry experience in Ontario and North America • Achieving adequate mixing of clayey silt soil may be difficult resulting in significant volumes of waste mix • May require significant right of way or easement beyond limits of wall face line • Durability concerns with wall face could require frost protection for a temporary wall and a facing and insulation system for a permanent wall.	• Limited industry experience in Ontario and North America • Specialized equipment and skills required	• May be more economical than secant pile or concrete diaphragm wall systems in specific instances

NOTE: 1. Table should not be used out of context or without reference to the corresponding sections of the report. Full detail and complexities of each system are not addressed in comparison table. Relative costs will vary depending on scope of selected construction method as mobilization costs may be a significant proportion of total costs.



Opportunity area in which U.S. plaza sites with connections to I-75 are being studied.

Three River Crossing options are being studied.

Three Canadian Plaza sites are being studied.

Canadian Access Road - At-grade, depressed, tunnel and service road options are being studied.

Notes:

1. This figure is to be read with the accompanying reports “Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor” prepared by Golder Associates, October 2007.
2. Site plan of area provided by URS Corporation.

Golder Associates

FILE: Fig 1.ppt

PROJECT NO: 04-1111-060

SCALE NTS

DATE June 2007

DESIGN SJB

CAD SJB

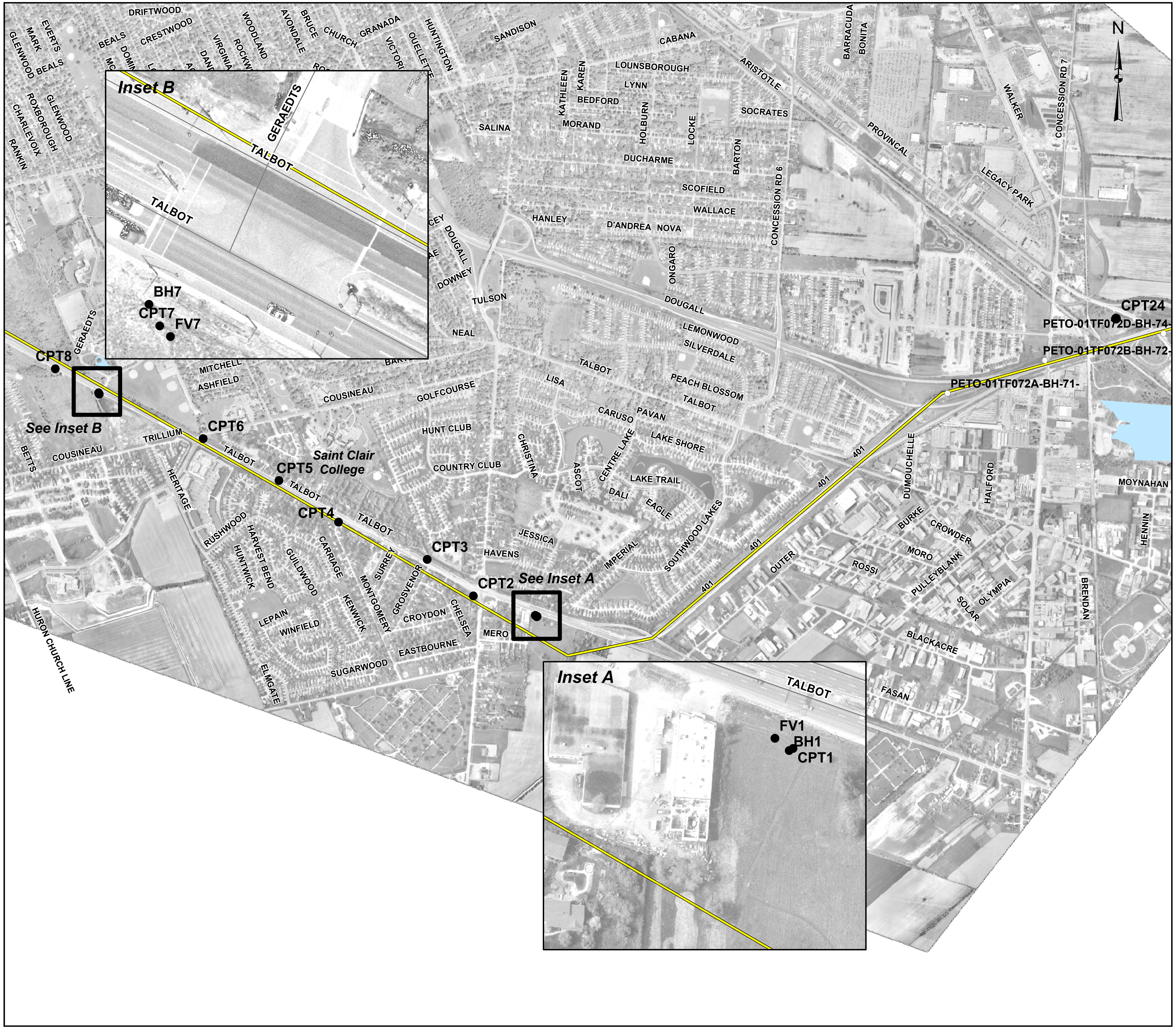
CHECK JW

REVIEW FJH

AREA OF CONTINUED ANALYSIS

DETROIT RIVER
INTERNATIONAL CROSSING

FIGURE 1



LEGEND

- ○ Borehole Locations - other projects
- ● Boreholes or Cone Penetration Test Locations for DRIC project
- 🟨 Location of Subsurface Profile

Note:
1. This drawing is to be read with and is not to be separated from the accompanying report text "Preliminary Foundation Investigation and Design Report, Detroit River International Crossing, Bridge Approach Corridor," prepared by Golder Associates Ltd., dated June 2007.
2. See report references for sources of borehole data for other projects.

Location	Easting-UTM	Northing-UTM
CPT24	338376	4679216
BH1	335500	4677738
CPT1	335502	4677739
FV1	335493	4677744
CPT2	335185	4677841
CPT3	334957	4678022
CPT4	334516	4678208
CPT5	334220	4678413
CPT6	333844	4678621
BH7	333325	4678848
CPT7	333327	4678844
FV7	333329	4678842
CPT8	333109	4678967

REFERENCE

Base Data - MNR NRVIS, obtained 2004, CANMAP v7.3 2003
Produced by Golder Associates Ltd under licence from Ontario Ministry of Natural Resources, © Queens Printer 2005
Datum: NAD 83 Projection: UTM Zone 17N

0 250 500 750 1,000
Metres

PROJECT
DETROIT RIVER INTERNATIONAL CROSSING

TITLE
APPROACH CORRIDOR EXPLORATION LOCATIONS

Golder Associates
Mississauga, Ontario

PROJECT No. 04-1111-060			SCALE 1:20,000	REV. 1
DESIGN	CC	24 May 2006	FIGURE: 2A	
GIS	CC	08 Jun. 2007		
CHECK	SB	08 Jun. 2007		
REVIEW	SB	08 Jun. 2007		

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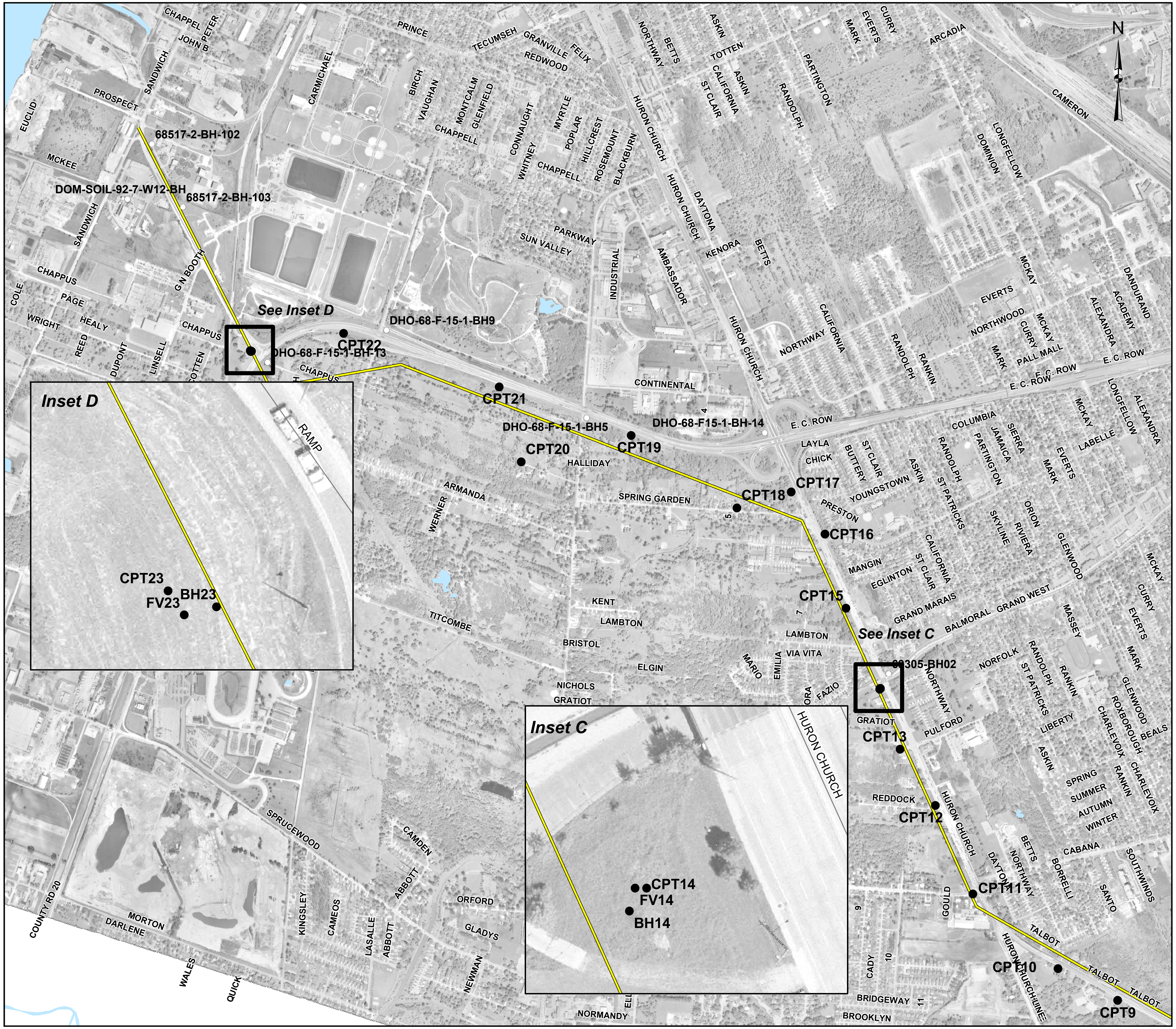


LEGEND

- ○ Borehole Locations - other projects
- ● Boreholes or Cone Penetration Test Locations for DRIC project
- ▬ Location of Subsurface Profile

Note:
1. This drawing is to be read with and is not to be separated from the accompanying report text "Preliminary Foundation Investigation and Design Report, Detroit River International Crossing, Bridge Approach Corridor," prepared by Golder Associates Ltd., June, 2007.
2. See report references for sources of borehole data for other projects.

Location	Easting-UTM	Northing-UTM
CPT9	332828	4679105
CPT10	332533	4679264
CPT11	332110	4679634
CPT12	331924	4680072
CPT13	331749	4680350
BH14	331648	4680648
FV14	331649	4680652
CPT14	331651	4680652
CPT15	331480	4681049
CPT16	331376	4681417
CPT17	331208	4681625
CPT18	330938	4681547
CPT19	330413	4681906
CPT20	329868	4681775
CPT21	329759	4682147
CPT22	328986	4682412
CPT23	328523	4682325
BH23	328529	4682323
FV23	328525	4682322

REFERENCE

Base Data - MNR NRVIS, obtained 2004, CANMAP v7.3 2003
Produced by Golder Associates Ltd under licence from Ontario Ministry of Natural Resources, © Queens Printer 2005
Datum: NAD 83 Projection: UTM Zone 17N

0 0.5 1 1.5
Kilometres

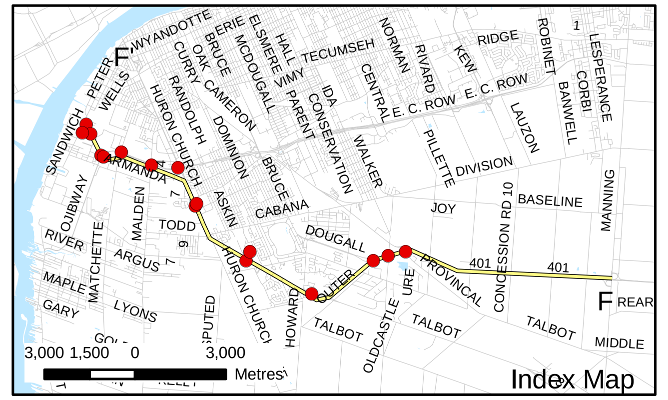
PROJECT
DETROIT RIVER INTERNATIONAL CROSSING

TITLE
APPROACH CORRIDOR EXPLORATION LOCATIONS

Golder Associates
Mississauga, Ontario

PROJECT No. 04-1111-060			SCALE 1:20,000	REV. 1
DESIGN	CC	24 May 2006	FIGURE: 2B	
GIS	CC	08 Jun. 2007		
CHECK	SB	08 Jun. 2007		
REVIEW	SB	08 Jun. 2007		

G:\Projects\2004\04-1111-060_Windsor_tunnel\GIS\MXDs\Draft\June2007_edits\Figure2B_Distances_along_ProfileF_pt1.mxd

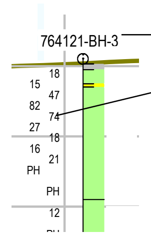


LEGEND

• CPT7	Cone Penetration Test Locations and Identification
▬	Ground Surface
▬	Bedrock Surface
╌	Approximate Static Ground Water Pressure Elevation in Soils
╌	Approximate Static Ground Water Pressure Elevation at Bedrock Interface

CPT Stratigraphy	Ic (Soil Type Index)
🟦	< 1.3 - Gravelly Sand
🟨	1.31 - 1.80 - Sand to Silty Sand
🟧	1.81 - 2.10 - Silty Sand to Sandy Silt
🟩	2.11 - 2.95 - Clayey Silt to Silty Clay
🟩	2.96 - 3.80 - Clay
🟪	> 3.80 - Clay and Organics

Borehole Stratigraphy	
⬛	Bedrock
⬜	Fill &/or Organic
🟦	Sand & Gravel
🟨	Sand & Silt
🟩	Clayey Silt to Silty Clay



Borehole Identification

Standard Penetration Test (blows/0.3m)
PH = Push Sampler Hydraulically
PM = Push Sampler Manually

REFERENCE
Base Data - Produced by Golder Associates Ltd under licence from Ontario Ministry of Natural Resources, © Queens Printer 2005
Datum: NAD 83 Projection: UTM Zone 17

Note:
1. The simplified stratigraphy is based on 13 boreholes and 23 cone penetration tests. Specific ground conditions between boreholes and cone penetration tests will be different than as shown.
2. Index map illustrates approximate locations of boreholes. See Figure 2 for locations of all boreholes and cone penetration tests.
3. See report text for methods of interpreting and interpolating soil type index between boreholes and cone penetration tests. Soil type index for cone penetration tests based on site-specific correlation as described in report text.
4. Vertical exaggeration = 25x
5. Boreholes and cone penetration test data projected from actual location onto vertical plane beneath profile as shown on Figure 2 and index map.
6. This drawing is to be read with and is not to be separated from the accompanying report text “Preliminary Foundation InOvestigation and Design Report, Detroit River International Crossing, Bridge Approach Corridor,” prepared by Golder Associates Ltd., October 2007.

250 0 250 500 750
Metres

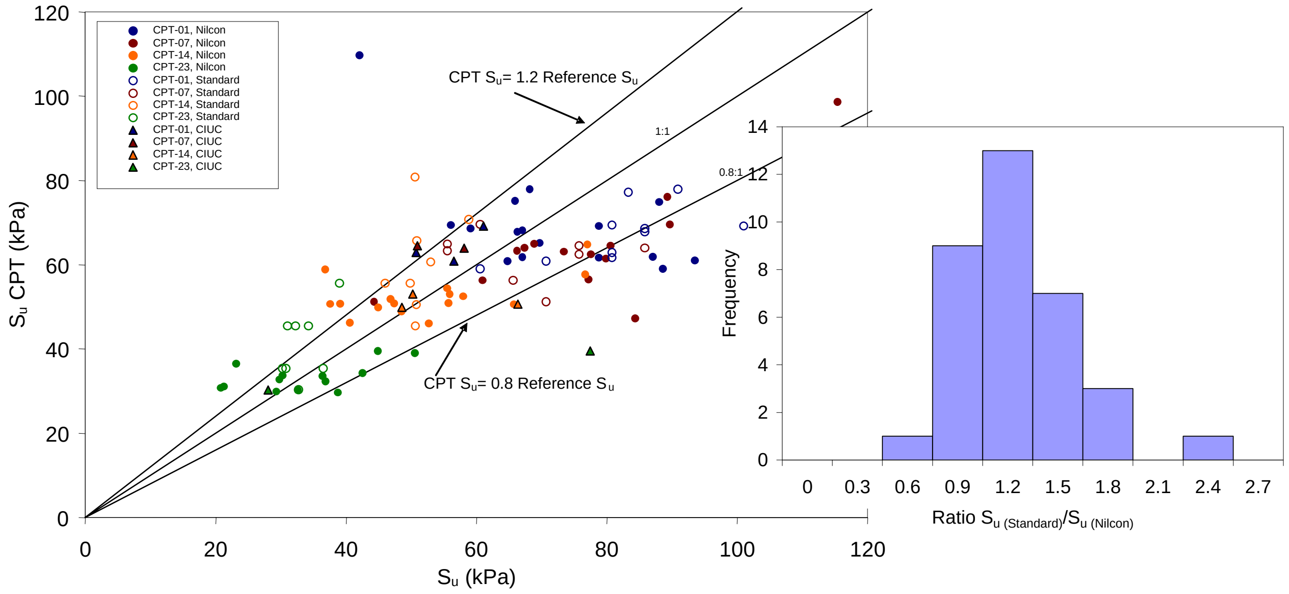
PROJECT
DETROIT RIVER INTERNATIONAL CROSSING

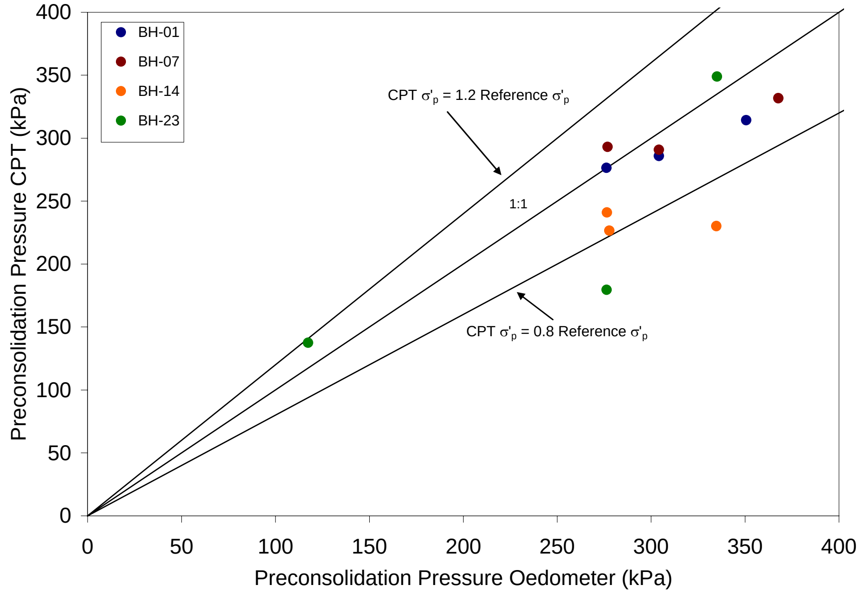
TITLE
SIMPLIFIED SUBSURFACE STRATIGRAPHY

Golder Associates
Mississauga, Ontario

PROJECT No. 04-1111-060			SCALE 1: 15,000	REV. 2
DESIGN	AW	15 Mar. 2005	FIGURE: 3	
GIS	CC	26 Sep. 2007		
CHECK	SB	26 Sep. 2007		
REVIEW	SB	26 Sep. 2007		

G:\Projects\2004\04-1111-060_Windsor_tunnel\GIS\MXDs\Draft\June2007_edits\Figure3_profile_making_Ic.mxd





Notes:

1. This figure is to be read with the accompanying reports “Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor” prepared by Golder Associates, October 2007.
2. “Standard” vane test refers to conventional MTO field vane shear testing device and “Nilcon” refers to use of push-in field vane shear test device as described within the referenced report.

Golder Associates

FILE: Soil Properties.ppt

PROJECT NO: 04-1111-060

SCALE NTS

DATE June 2007

DESIGN SJB

CAD SJB

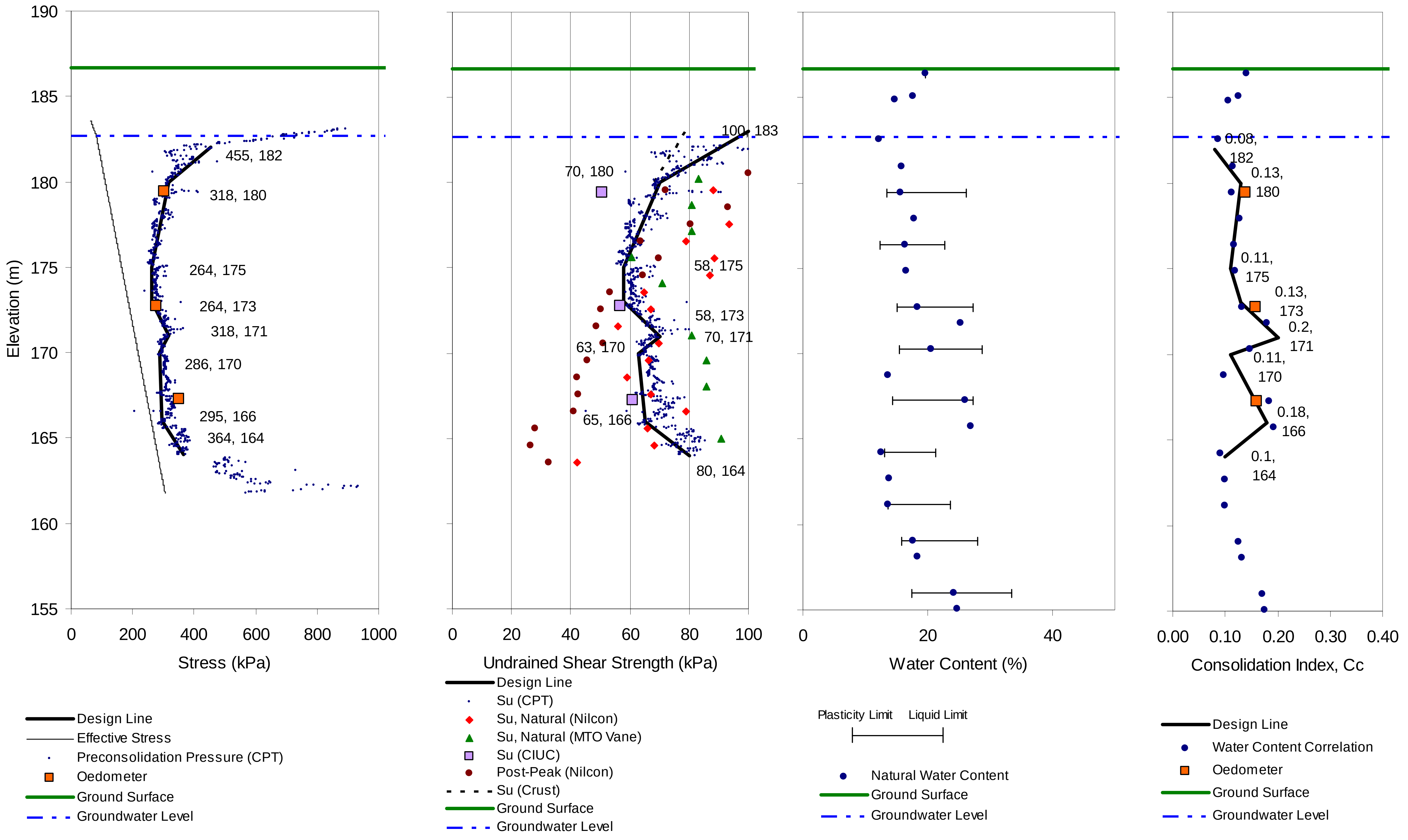
CHECK JW

REVIEW FJH

CORRELATION OF TESTING METHODS

DETROIT RIVER
INTERNATIONAL CROSSING

FIGURE
4



Notes:
1. This figure is to be read with the accompanying reports "Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor" prepared by Golder Associates, October 2007.
2. Data and correlations used for development of this figure are discussed in the above referenced reports.
3. Limitations of the Design Lines are discussed in "Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor" prepared by Golder Associates, October 2007.
4. Conditions between the exploration location identified in this figure and other locations will vary. Variation of soil characteristics and engineering properties between samples will also occur.

PROJECT
DETROIT RIVER INTERNATIONAL CROSSING

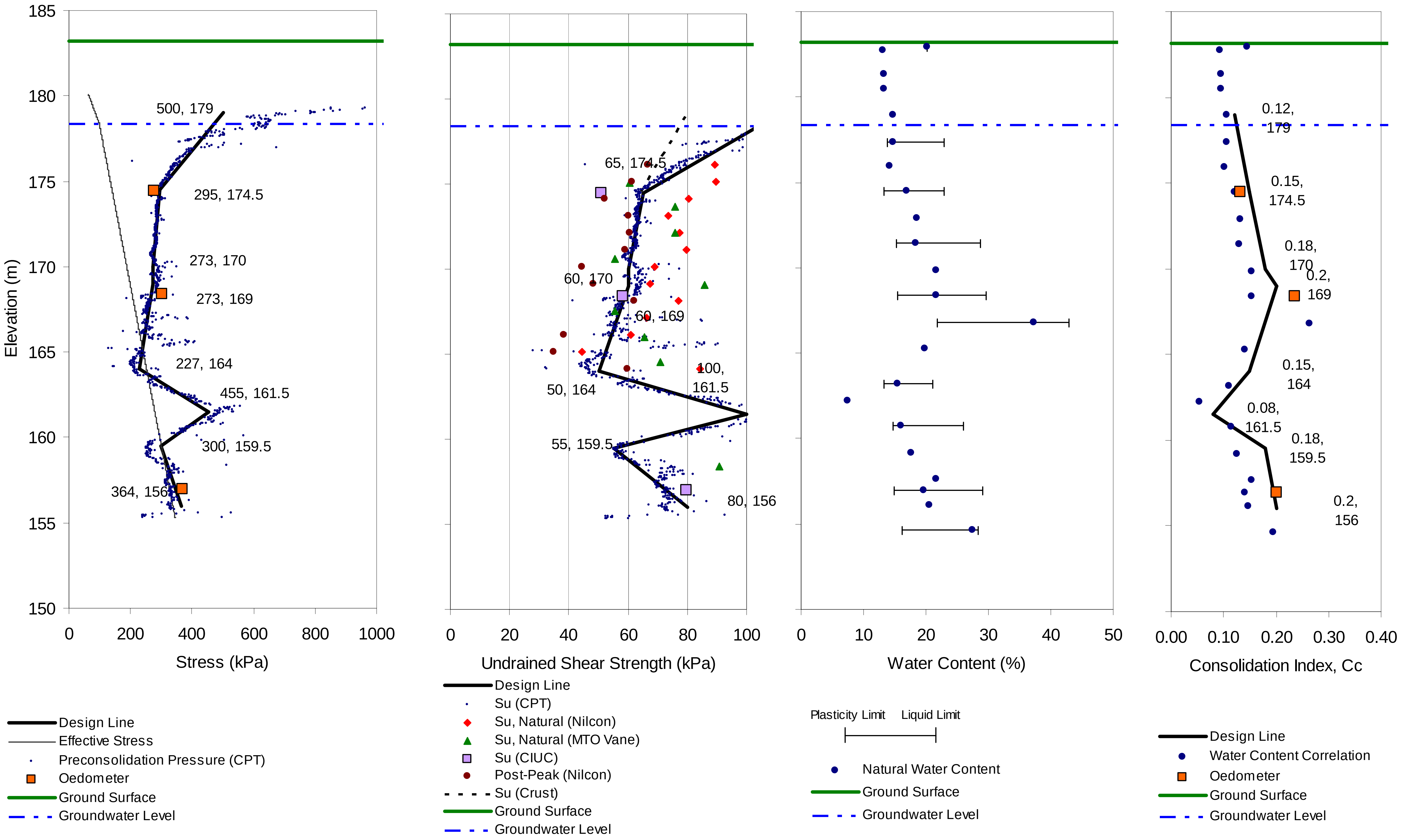
TITLE
SUMMARY OF GEOTECHNICAL PROPERTIES
BOREHOLE BH/FV/CPT-1

Golder Associates

PROJECT No.: 04-1111-060
SCALE: AS SHOWN

DESIGN	SJB	June 2007
DRAW	SJB	June 2007
CHECK	JW	June 2007
REVIEW	FJH	June 2007

FIGURE: 5



Notes:
1. This figure is to be read with the accompanying reports “Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor” prepared by Golder Associates, October 2007.
2. Data and correlations used for development of this figure are discussed in the above referenced reports.
3. Limitations of the Design Lines are discussed in “Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor” prepared by Golder Associates, October 2007.
4. Conditions between the exploration location identified in this figure and other locations will vary. Variation of soil characteristics and engineering properties between samples will also occur.

PROJECT
DETROIT RIVER INTERNATIONAL CROSSING

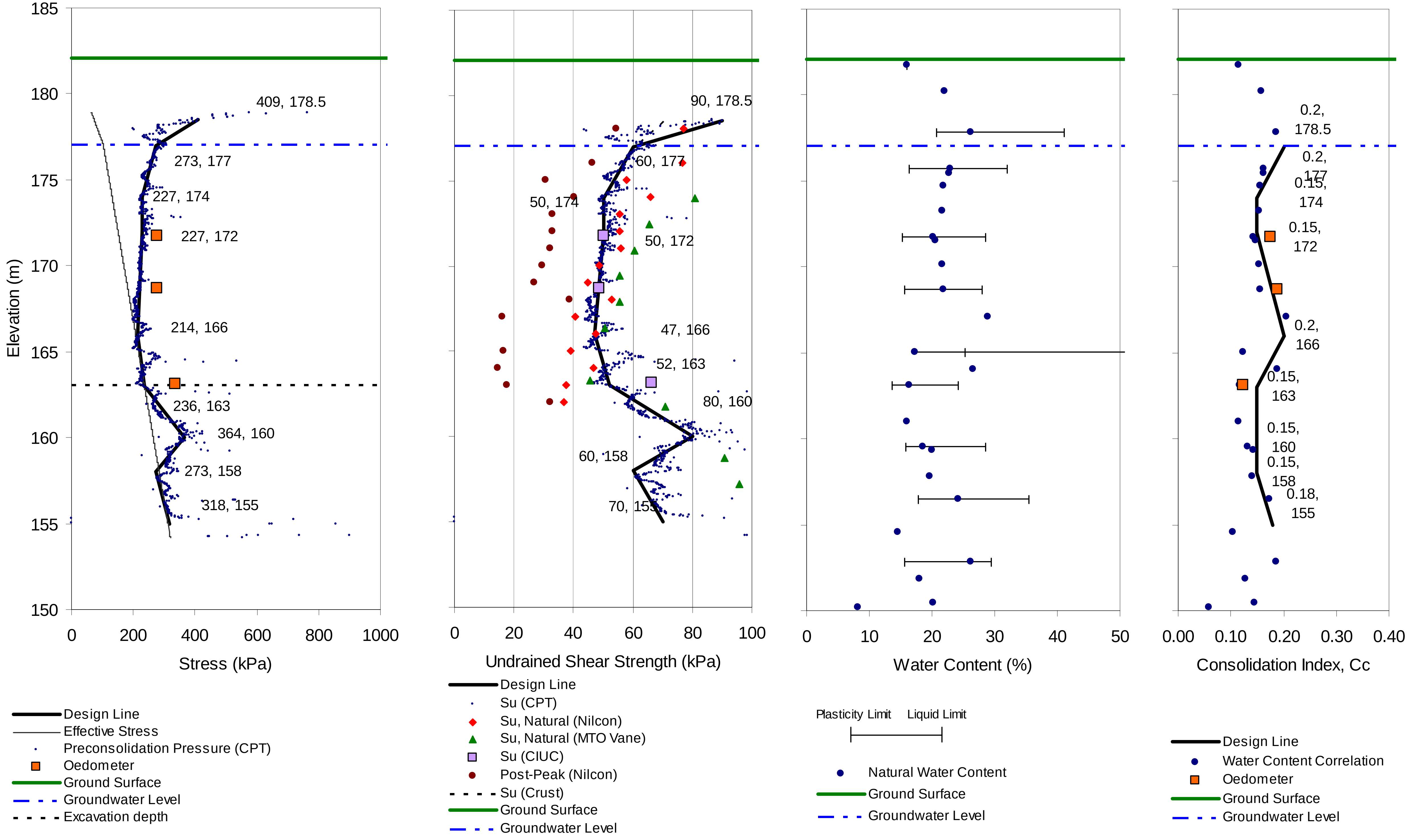
TITLE
SUMMARY OF GEOTECHNICAL PROPERTIES
BOREHOLE BH/FV/CPT-7

Golder Associates

PROJECT No.: 04-1111-060
SCALE: AS SHOWN

DESIGN	SJB	June 2007
DRAW	SJB	June 2007
CHECK	JW	June 2007
REVIEW	FJH	June 2007

FIGURE: 6



Notes:

1. This figure is to be read with the accompanying reports “Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor” prepared by Golder Associates, October 2007.
2. Data and correlations used for development of this figure are discussed in the above referenced reports.
3. Limitations of the Design Lines are discussed in “Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor” prepared by Golder Associates, October 2007.
4. Conditions between the exploration location identified in this figure and other locations will vary. Variation of soil characteristics and engineering properties between samples will also occur.

PROJECT
DETROIT RIVER INTERNATIONAL CROSSING

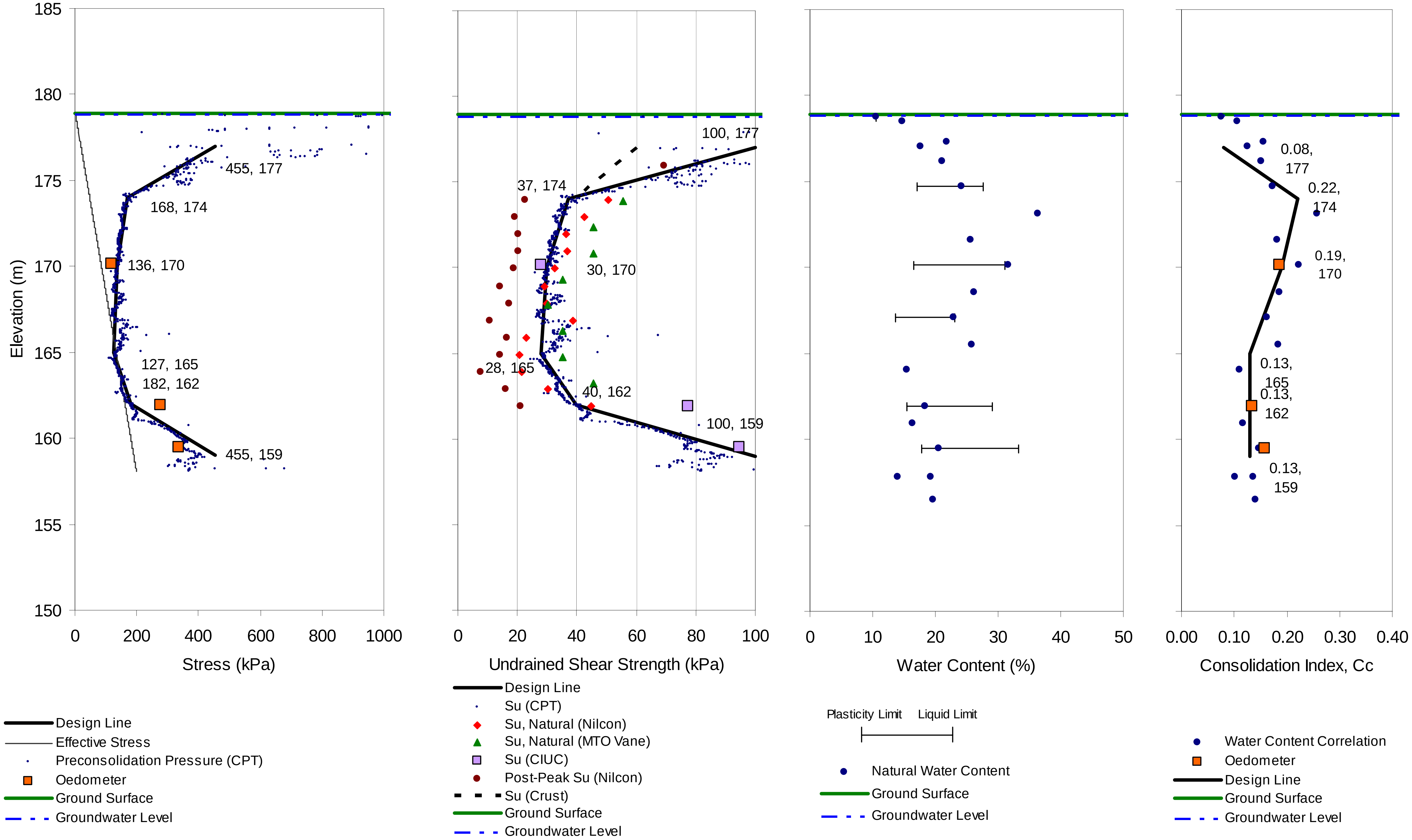
TITLE
SUMMARY OF GEOTECHNICAL PROPERTIES
BOREHOLE BH/FV/CPT-14

Golder Associates

PROJECT No.: 04-1111-060
SCALE: AS SHOWN

DESIGN	SJB	June 2007
DRAW	SJB	June 2007
CHECK	JW	June 2007
REVIEW	FJH	June 2007

FIGURE: 7



Notes:

1. This figure is to be read with the accompanying reports “Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor” prepared by Golder Associates, October 2007.
2. Data and correlations used for development of this figure are discussed in the above referenced reports.
3. Limitations of the Design Lines are discussed in “Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor” prepared by Golder Associates, October 2007.
4. Conditions between the exploration location identified in this figure and other locations will vary. Variation of soil characteristics and engineering properties between samples will also occur.

PROJECT
DETROIT RIVER INTERNATIONAL CROSSING

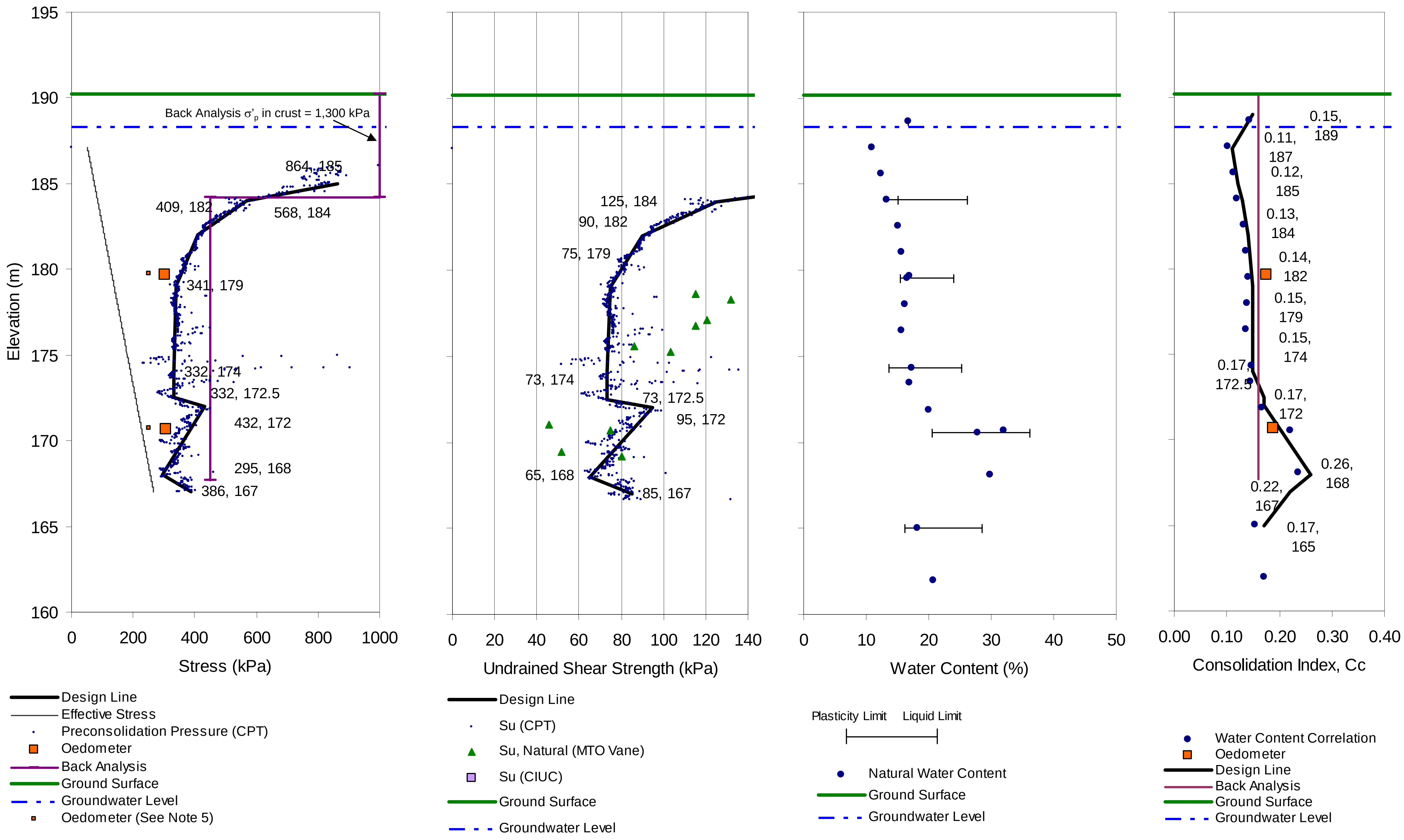
TITLE
SUMMARY OF GEOTECHNICAL PROPERTIES
BOREHOLE BH/FV/CPT-23

Golder Associates

PROJECT No.: 04-1111-060
SCALE: AS SHOWN

DESIGN	SJB	June 2007
DRAW	SJB	June 2007
CHECK	JW	June 2007
REVIEW	FJH	June 2007

FIGURE: 8



Notes:

1. This figure is to be read with the accompanying reports "Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor" prepared by Golder Associates, October 2007.
2. Data and correlations used for development of this figure are discussed in the above referenced reports and Golder Associates Ltd. (2006) Structure Settlement Study, Highway 401 Reconstruction, GWP 64-00-00, Ministry of Transportation Ontario, Southwestern Region, Geocres No. 40J2-79, November, 2006.
3. Limitations of the Design Lines are discussed in "Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor" prepared by Golder Associates, October 2007.
4. Conditions between the exploration location identified in this figure and other locations will vary. Variation of soil characteristics and engineering properties between samples will also occur.
5. Oedometer interpretation as shown based on Schmertmann (1955) and Soderman and Kim (1970)

PROJECT
DETROIT RIVER INTERNATIONAL CROSSING

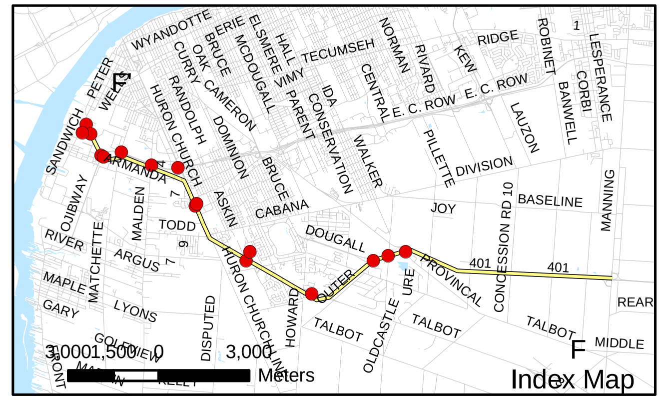
TITLE
SUMMARY OF GEOTECHNICAL PROPERTIES
BOREHOLE BH/FV/CPT-24

Golder Associates

PROJECT No.: 04-1111-060
SCALE: AS SHOWN

DESIGN	SJB	June 2007
DRAW	SJB	June 2007
CHECK	JW	June 2007
REVIEW	FJH	June 2007

FIGURE: 9



LEGEND

CPT7 • Cone Penetration Test Locations and Identification

Ground Surface

Bedrock Surface

Approximate Static Ground Water Pressure Elevation in Soils

Approximate Static Ground Water Pressure Elevation at Bedrock Interface

Undrained Shear Strength (kPa)

🟦	0 - 20
🟦	20 - 40
🟪	40 - 60
🟥	60 - 80
🟧	80 - 100
🟨	> 100

Borehole Stratigraphy

⬛	Bedrock
⬜	Fill &/or Organic
🟦	Sand & Gravel
🟨	Sand & Silt
🟩	Clayey Silt to Silty Clay

764121-BH-3 — Borehole Identification

Standard Penetration Test (blows/0.3m)
PH = Push Sampler Hydraulically
PM = Push Sampler Manually

REFERENCE
Base Data - Produced by Golder Associates Ltd under licence from Ontario Ministry of Natural Resources, © Queens Printer 2005
Datum: NAD 83 Projection: UTM Zone 17

250 0 250 500 750
Metres

Note:
1. The simplified stratigraphy is based on 13 boreholes and 23 cone penetration tests. Specific ground conditions between boreholes and cone penetration tests will be different than as shown.
2. Index map illustrates approximate locations of boreholes. See Figure 2 for locations of all boreholes and cone penetration tests.
3. See report text for methods of interpreting and interpolating undrained shear strength values between boreholes and cone penetration tests.
4. Vertical exaggeration = 25x
5. Boreholes and cone penetration test data projected from actual location onto vertical plane beneath profile as shown on Figure 2 and index map.
6. This drawing is to be read with and is not to be separated from the accompanying report text “Preliminary Foundation Investigation and Design Report, Detroit River International Crossing, Bridge Approach Corridor,“ prepared by Golder Associates Ltd., dated October 2007.

PROJECT
DETROIT RIVER INTERNATIONAL CROSSING

TITLE
PROFILE OF UNDRAINED SHEAR STRENGTH

Golder Associates
Mississauga, Ontario

PROJECT No. 04-1111-060			SCALE 1: 15,000	REV. 2
DESIGN	AW	15 Mar. 2005	**FIGURE: 10**	
GIS	CC	26 Sep. 2007		
CHECK	SB	26 Sep. 2007		
REVIEW	SB	26 Sep. 2007		

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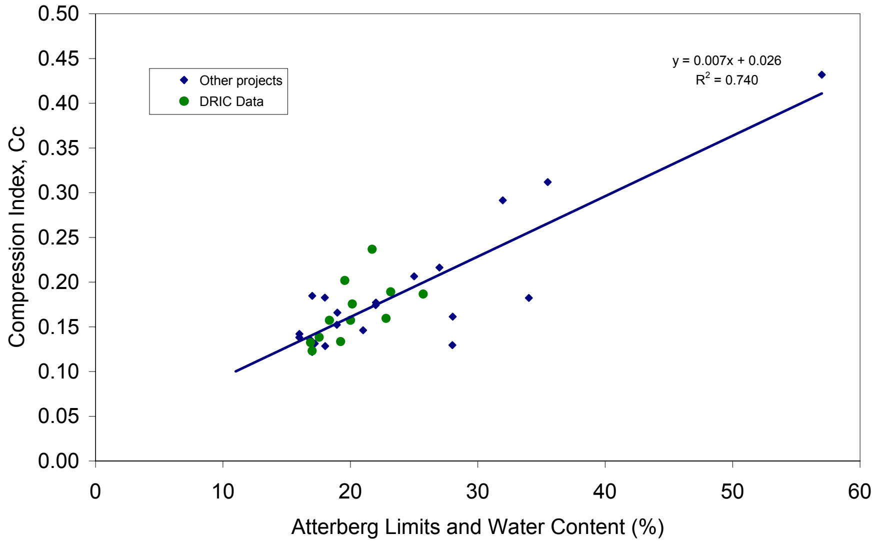
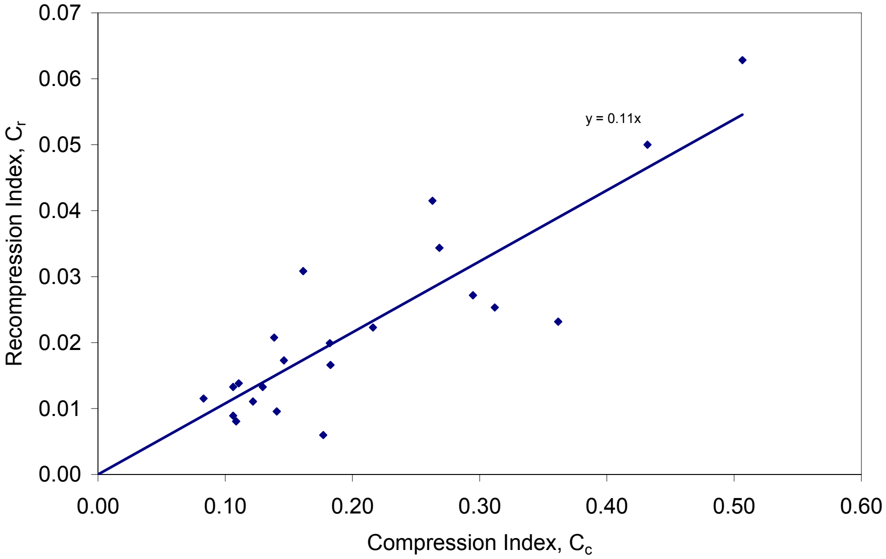




Notes:

1. This figure is to be read with the accompanying reports “Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor” prepared by Golder Associates, October 2007.

Golder Associates

FILE:	Soil Properties.ppt
PROJECT NO:	04-1111-060

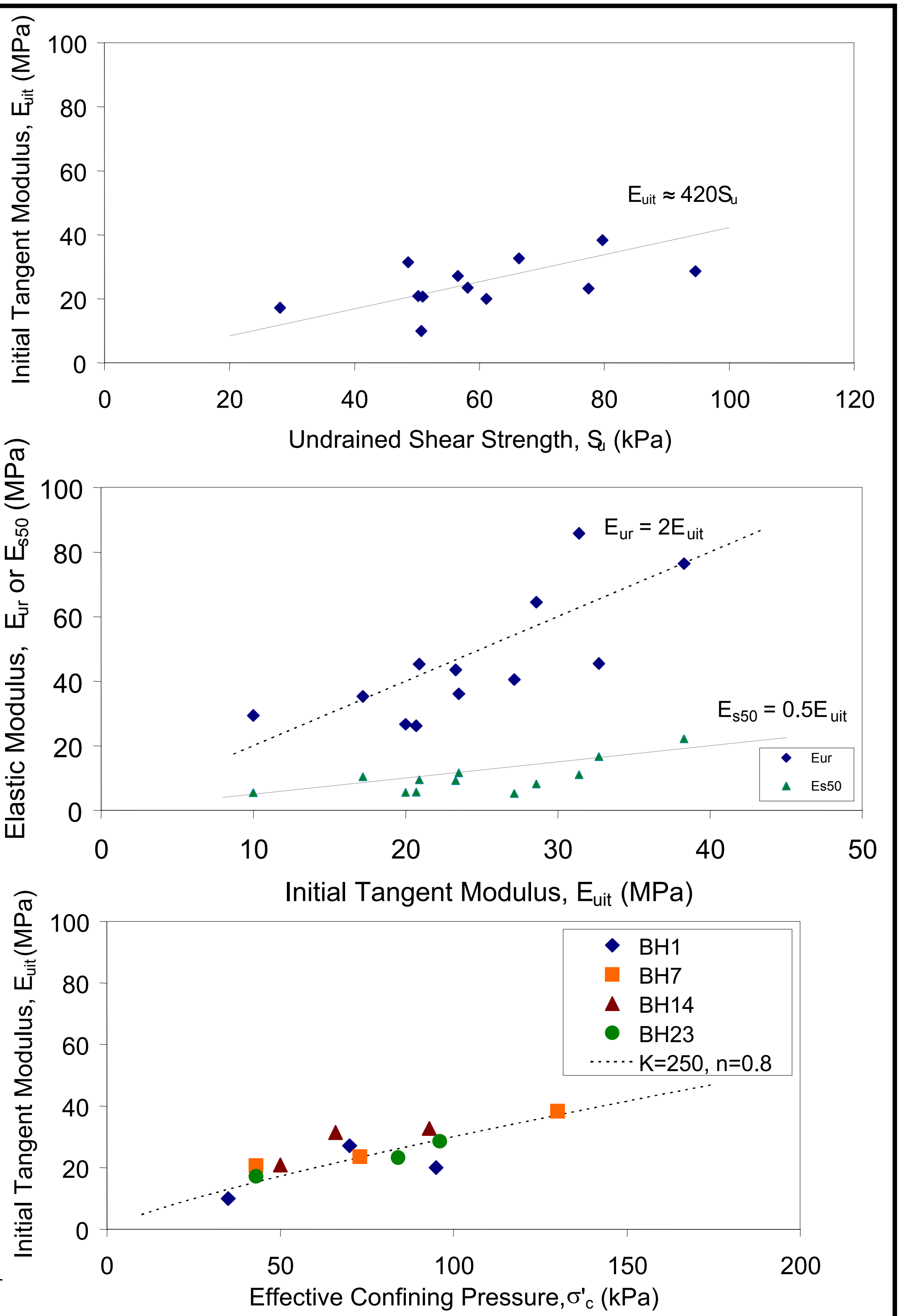
SCALE	NTS
DATE	June 2007
DESIGN	SJB
CAD	SJB
CHECK	JW
REVIEW	FJH

STRESS-STRAIN PROPERTIES OF SILTY CLAY

DETROIT RIVER
INTERNATIONAL CROSSING

FIGURE

11a



Notes:

1. This figure is to be read with the accompanying reports “Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor” prepared by Golder Associates, October 2007.

Golder Associates

SCALE	NTS
DATE	June 2007
DESIGN	SJB
CAD	SJB
CHECK	JW
REVIEW	FJH

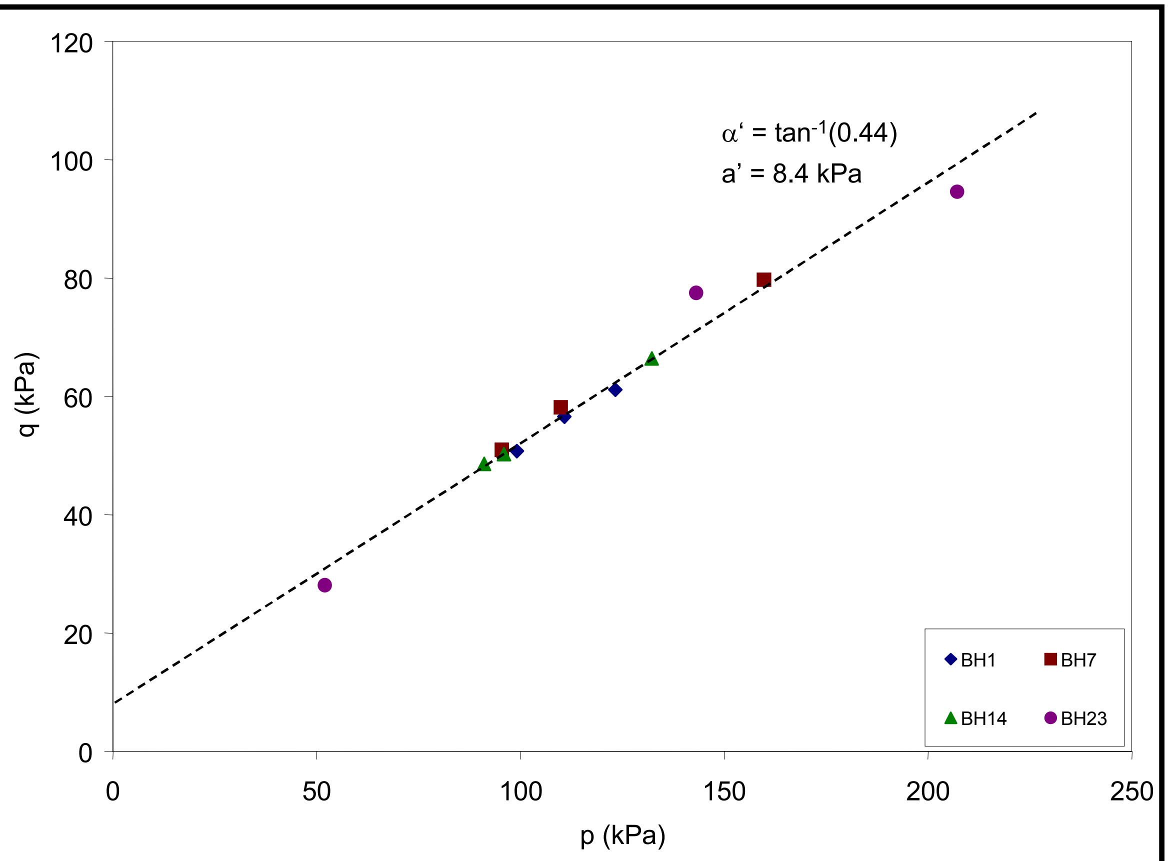
FILE: Soil Properties.ppt

PROJECT NO: 04-1111-060

STRESS-STRAIN PROPERTIES OF SILTY CLAY

DETROIT RIVER
INTERNATIONAL CROSSING

FIGURE
11b



Calculation of Effective Stress Angle of Internal Friction

Sample	p (kPa)	q (kPa)	σ'_{vo} (kPa)	σ'_{p} (kPa)	σ'_{c} (kPa)	a'(kPa)	tanα'	c' (kPa)	ϕ'
BH1									
A	99.2	50.7	131	304	35				
B	110.8	56.5	199	276	70	8.5	0.43	9.4	25.4
C	123.3	61.1	247	351	95				
BH7									
A	95.5	50.9	148	277	43				
B	110.0	58.1	208	304	73	8.9	0.44	9.9	26.3
C	159.8	79.7	340	368	130				
BH14									
A	96.0	50.2	164	276	50				
B	91.1	48.6	187	278	66	8.5	0.44	9.4	26.0
C	132.2	66.4	267	335	93				
BH23									
A	52.1	28.1	89	117	43				
B	143.2	77.5	195	276	84	8.2	0.44	9.1	25.9
C	207.2	94.6	230	335	96				

All Data

a (kPa)	α'	c' (kPa)	ϕ'
8.4	23.7	9.3	26.0

Effective Cohesion Intercept = 0

a (kPa)	α'	c' (kPa)	ϕ'
0	26.7	0.0	30.1

Notes:

1. This figure is to be read with the accompanying reports “Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor” prepared by Golder Associates,October 2007.

Golder Associates

SCALE	NTS
DATE	June 2007
DESIGN	SJB
CAD	SJB
CHECK	JW
REVIEW	FJH

EFFECTIVE STRESS SHEAR STRENGTH PROPERTIES OF SILTY CLAY

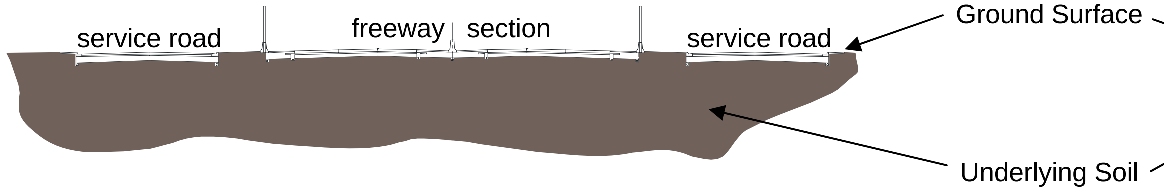
DETROIT RIVER
INTERNATIONAL CROSSING

FIGURE **12**

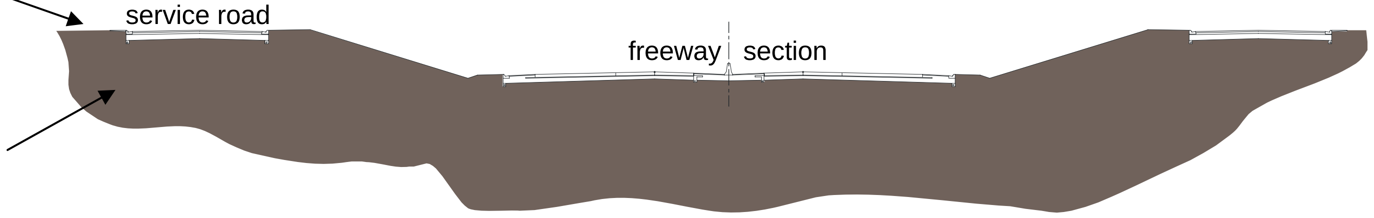
FILE: Soil Properties.ppt

PROJECT NO: 04-1111-060

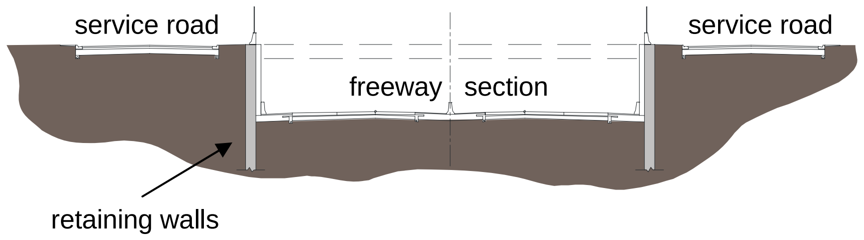
AT-GRADE FREEWAY
SERVICE ROADS BOTH SIDES



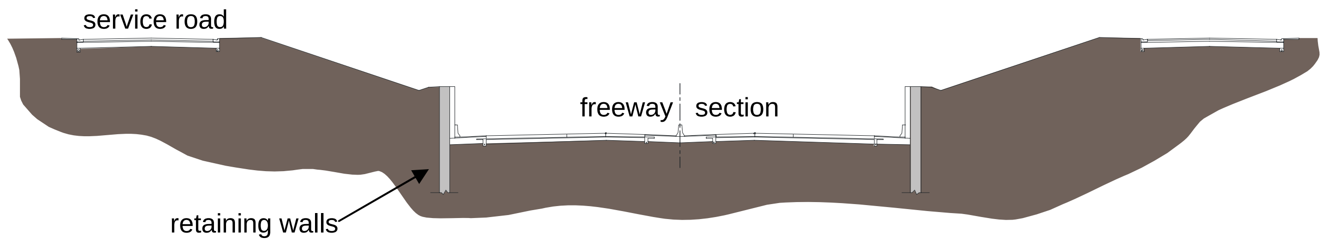
BELOW-GRADE FREEWAY WITH SIDE SLOPES
SERVICE ROADS BOTH SIDES



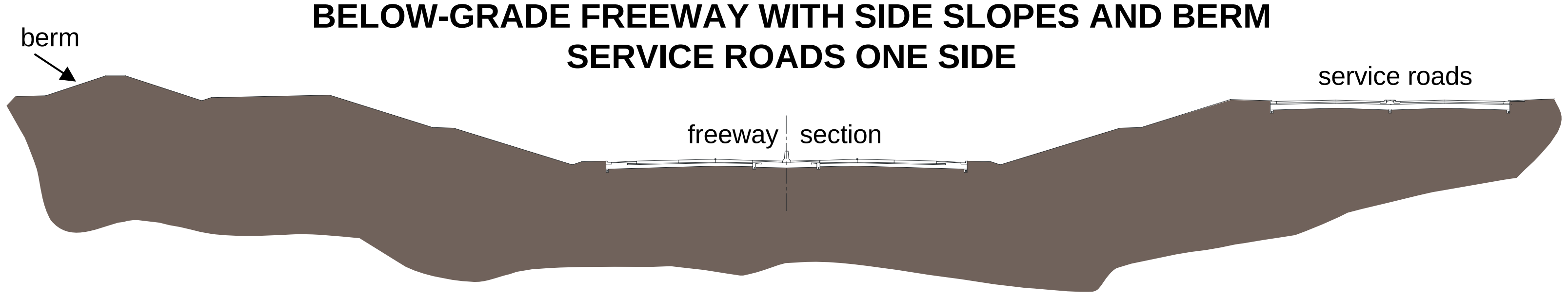
BELOW-GRADE FREEWAY WITH RETAINING WALLS
SERVICE ROADS BOTH SIDES



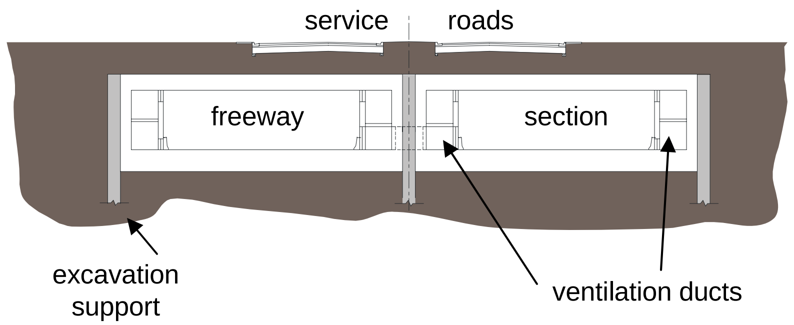
BELOW-GRADE FREEWAY WITH RETAINING WALLS AND SLOPES
SERVICE ROADS BOTH SIDES



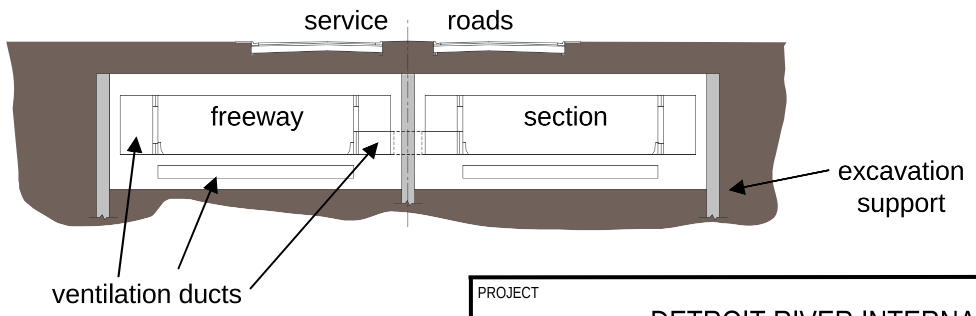
BELOW-GRADE FREEWAY WITH SIDE SLOPES AND BERM
SERVICE ROADS ONE SIDE



CUT AND COVER TUNNEL (LESS THAN 2 KM LONG)
SERVICE ROADS AT GRADE



CUT AND COVER TUNNEL (MORE THAN 2 KM LONG)
SERVICE ROADS AT GRADE



Notes:

1. This figure is to be read with the accompanying reports “Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor” prepared by Golder Associates, October 2007.
2. Example conceptual roadway cross-sections are based on information provided by URS Corporation and are not comprehensive of all cross sections considered for this project. Service roads may be on one side, both sides, or neither side of the proposed freeway depending on location and option underconsideration.

PROJECT
DETROIT RIVER INTERNATIONAL CROSSING

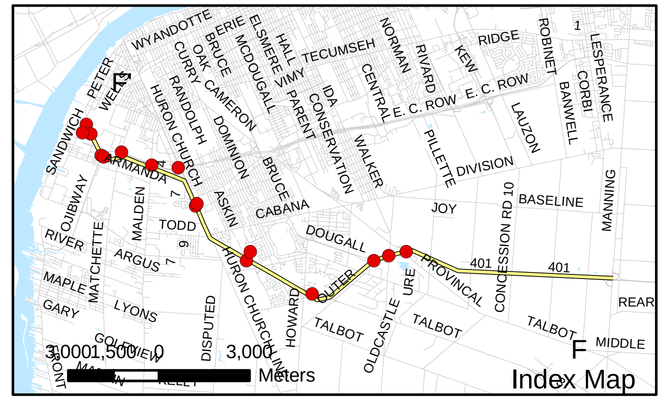
TITLE
CONCEPTUAL ROADWAY CROSS SECTIONS

Golder Associates

PROJECT No.: 04-1111-060
SCALE: NOT TO SCALE

DESIGN	SJB	June 2007
DRAW	SJB	June 2007
CHECK	JW	June 2007
REVIEW	FJH	June 2007

FIGURE: 13



Index Map

LEGEND

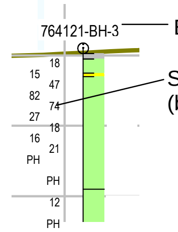
- CPT7 Cone Penetration Test Locations and Identification
- Ground Surface
- Bedrock Surface
- At-Grade Highway (Proposed)
- Approximate Static Ground Water Pressure Elevation in Soils
- Approximate Static Ground Water Pressure Elevation at Bedrock Interface

Factor of Safety [1, 2]

🟥	FS < 1.0
🟧	1.0 - 1.3
🟨	1.3 - 1.5
🟩	1.5 - 2.0
🟫	FS > 2.0

Borehole Stratigraphy

■	Bedrock
■	Fill &/or Organic
🟦	Sand & Gravel
🟨	Sand & Silt
🟩	Clayey Silt to Silty Clay



Borehole Identification

Standard Penetration Test (blows/0.3m)
PH = Push Sampler Hydraulically
PM = Push Sampler Manually

REFERENCE

Base Data - Produced by Golder Associates Ltd under licence from Ontario Ministry of Natural Resources, © Queens Printer 2005
Datum: NAD 83 Projection: UTM Zone 17

Note:

1. The simplified and interpolated factor of safety for excavation base stability is based on interpreted undrained shear strength values from 13 boreholes and 23 cone penetration tests. Specific conditions between boreholes will be different than as shown.
2. Factor of safety (FS) as illustrated is based on FS = 5.5Su/(γH) where Su is taken as the average value to about 0.5H below road line shown.
Actual factor of safety will differ depending on dimensions of excavations and will be influenced by conditions above and below the points indicated on this drawing. This drawing is provided only as a guide to assist in defining where additional stability enhancement works may be required. See report test for additional description
on the use and limitations of this drawing.
3. The need for stability enhancement measures will vary along the alignment and is not to be interpreted as precisely defined where shown. Changes between areas will be transitional. General extent of need for stability enhancement, areas that may not require stability enhancement, and transition zones, are shown for conceptual design purposes only. Additional exploration, testing, and detailed evaluation and design will be necessary for all locations along the alignment and zones requiring enhancement may be greater or less than shown.
4. Index map illustrates approximate locations of boreholes. See Figure 2 for locations of all boreholes and cone penetration tests.
5. See report text for methods of interpreting and interpolating undrained shear strength values between boreholes and cone penetration tests.
6. Vertical exaggeration = 25x
7. Boreholes and cone penetration test data projected from actual location onto vertical plane beneath profile as shown on Figure 2 and index map.
8. Profile of highway road surface based on data provided by URS Corporation. Profile as shown is projected from conceptual design horizontal and vertical alignment onto the simplified profile illustrated on Figure 2 and curvature and slopes will therefore be distorted from those shown on the conceptual design drawings.
9. This drawing is to be read with and is not to be separated from the accompanying report text "Preliminary Foundation Investigation and Design Report, Detroit River International Crossing, Bridge Approach Corridor," prepared by Golder Associates Ltd., October 2007.

250 0 250 500 750
Metres

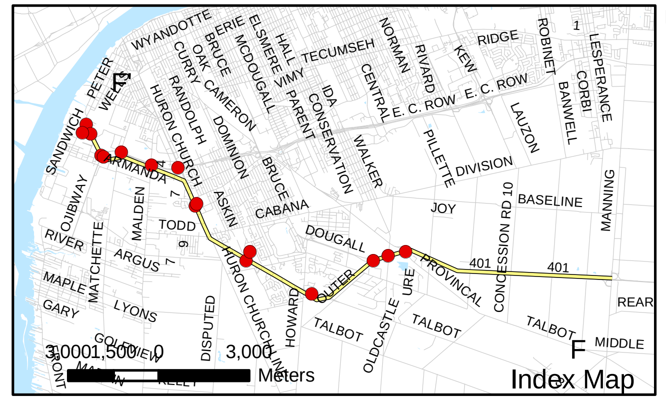
PROJECT
DETROIT RIVER INTERNATIONAL CROSSING

TITLE
EXCAVATION STABILITY, AT-GRADE HIGHWAY

Golder Associates
Mississauga, Ontario

PROJECT No. 04-1111-060			SCALE 1: 15,000	REV. 2
DESIGN	AW	15 Mar. 2005	**FIGURE: 14**	
GIS	CC	27 Sep. 2007		
CHECK	SB	27 Sep. 2007		
REVIEW	SB	27 Sep. 2007		

G:\Projects\2004\04-1111-060_Windsor_tunnel\GIS\MXDs\Draft\June2007_edits\Fig14_Excavation Stability_At-Grade Highway.mxd

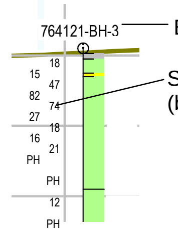


LEGEND

- ● CPT7 Cone Penetration Test Locations and Identification
- Ground Surface
- Bedrock Surface
- Below-Grade Highway (Proposed)
- Approximate Static Ground Water Pressure Elevation in Soils
- Approximate Static Ground Water Pressure Elevation at Bedrock Interface

Factor of Safety [1, 2]	
🟥	FS < 1.0
🟧	1.0 - 1.3
🟨	1.3 - 1.5
🟩	1.5 - 2.0
🟫	FS > 2.0

Borehole Stratigraphy	
▨	Bedrock
▨	Fill &/or Organic
🟦	Sand & Gravel
🟨	Sand & Silt
🟩	Clayey Silt to Silty Clay



Borehole Identification

Standard Penetration Test (blows/0.3m)
PH = Push Sampler Hydraulically
PM = Push Sampler Manually

REFERENCE
Base Data - Produced by Golder Associates Ltd under licence from Ontario Ministry of Natural Resources, © Queens Printer 2005
Datum: NAD 83 Projection: UTM Zone 17

Note:
1. The simplified and interpolated factor of safety for excavation base stability is based on interpreted undrained shear strength values from 13 boreholes and 23 cone penetration tests. Specific conditions between boreholes will be different than as shown.
2. Factor of safety (FS) as illustrated is based on FS = 5.5Su/(γH) where Su is taken as the average value to about 0.5H below road line shown.
Actual factor of safety will differ depending on dimensions of excavations and will be influenced by conditions above and below the points indicated on this drawing. This drawing is provided only as a guide to assist in defining where additional stability enhancement works may be required. See report test for additional description on the use and limitations of this drawing.
3. The need for stability enhancement measures will vary along the alignment and is not to be interpreted as precisely defined where shown. Changes between areas will be transitional. General extent of need for stability enhancement, areas that may not require stability enhancement, and transition zones, are shown for conceptual design purposes only. Additional exploration, testing, and detailed evaluation and design will be necessary for all locations along the alignment and zones requiring enhancement may be greater or less than shown.
4. Index map illustrates approximate locations of boreholes. See Figure 2 for locations of all boreholes and cone penetration tests.
5. See report text for methods of interpreting and interpolating undrained shear strength values between boreholes and cone penetration tests.
6. Vertical exaggeration = 25x
7. Boreholes and cone penetration test data projected from actual location onto vertical plane beneath profile as shown on Figure 2 and index map.
8. Profile of highway road surface based on data provided by URS Corporation. Profile as shown is projected from conceptual design horizontal and vertical alignment onto the simplified profile illustrated on Figure 2 and curvature and slopes will therefore be distorted from those shown on the conceptual design drawings.
9. This drawing is to be read with and is not to be separated from the accompanying report text "Preliminary Foundation Investigation and Design Report, Detroit River International Crossing, Bridge Approach Corridor," prepared by Golder Associates Ltd., October 2007.

250 0 250 500 750
Metres

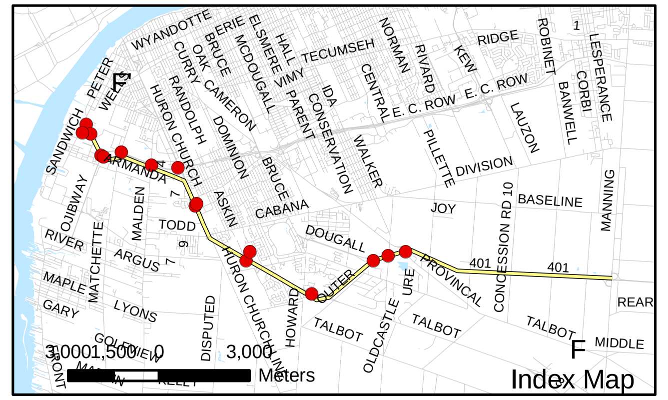
PROJECT
DETROIT RIVER INTERNATIONAL CROSSING

TITLE
EXCAVATION STABILITY, BELOW-GRADE HIGHWAY

Golder Associates
Mississauga, Ontario

PROJECT No. 04-1111-060			SCALE 1: 15,000	REV. 2
DESIGN	AW	15 Mar. 2005	**FIGURE: 15**	
GIS	CC	27 Sep. 2007		
CHECK	SB	27 Sep. 2007		
REVIEW	SB	27 Sep. 2007		

G:\Projects\2004\04-1111-060_Windsor_tunnel\GIS\MXDs\Draft\June2007_edits\Fig15_Excavation Stability\Below-Grade Highway .mxd



LEGEND

- CPT7 • Cone Penetration Test Locations and Identification
- Ground Surface
- Bedrock Surface
- Cut and Cover Tunnel
- Approximate Static Ground Water Pressure Elevation in Soils
- Approximate Static Ground Water Pressure Elevation at Bedrock Interface

Factor of Safety[1, 2]

- 🟥 FS < 1.0
- 🟧 1.0 - 1.3
- 🟨 1.3 - 1.5
- 🟩 1.5 - 2.0
- 🟫 FS > 2.0

Borehole Stratigraphy

- ■ Bedrock
- ■ Fill &/or Organic
- 🟦 Sand & Gravel
- 🟨 Sand & Silt
- 🟩 Clayey Silt to Silty Clay

764121-BH-3 — Borehole Identification

Standard Penetration Test (blows/0.3m)
PH = Push Sampler Hydraulically
PM = Push Sampler Manually

REFERENCE

Base Data - Produced by Golder Associates Ltd under licence from Ontario Ministry of Natural Resources, © Queens Printer 2005
Datum: NAD 83 Projection: UTM Zone 17

Note:

1. The simplified and interpolated factor of safety for excavation base stability is based on interpreted undrained shear strength values from 13 boreholes and 23 cone penetration tests. Specific conditions between boreholes will be different than as shown.
2. Factor of safety (FS) as illustrated is based on FS = 5.5Su/(γH) where Su is taken as the average value to about 0.5H below road line shown.
Actual factor of safety will differ depending on dimensions of excavations and will be influenced by conditions above and below the points indicated on this drawing. This drawing is provided only as a guide to assist in defining where additional stability enhancement works may be required. See report test for additional description on the use and limitations of this drawing.
3. The need for stability enhancement measures will vary along the alignment and is not to be interpreted as precisely defined where shown. Changes between areas will be transitional. General extent of need for stability enhancement, areas that may not require stability enhancement, and transition zones, are shown for conceptual design purposes only. Additional exploration, testing, and detailed evaluation and design will be necessary for all locations along the alignment and zones requiring enhancement may be greater or less than shown.
4. Index map illustrates approximate locations of boreholes. See Figure 2 for locations of all boreholes and cone penetration tests.
5. See report text for methods of interpreting and interpolating undrained shear strength values between boreholes and cone penetration tests.
6. Vertical exaggeration = 25x
7. Boreholes and cone penetration test data projected from actual location onto vertical plane beneath profile as shown on Figure 2 and index map.
8. Profile of highway road surface based on data provided by URS Corporation. Profile as shown is projected from conceptual design horizontal and vertical alignment onto the simplified profile illustrated on Figure 2 and curvature and slopes will therefore be distorted from those shown on the conceptual design drawings.
9. This drawing is to be read with and is not to be separated from the accompanying report text "Preliminary Foundation Investigation and Design Report, Detroit River International Crossing, Bridge Approach Corridor," prepared by Golder Associates Ltd., October 2007.

250 0 250 500 750
Metres

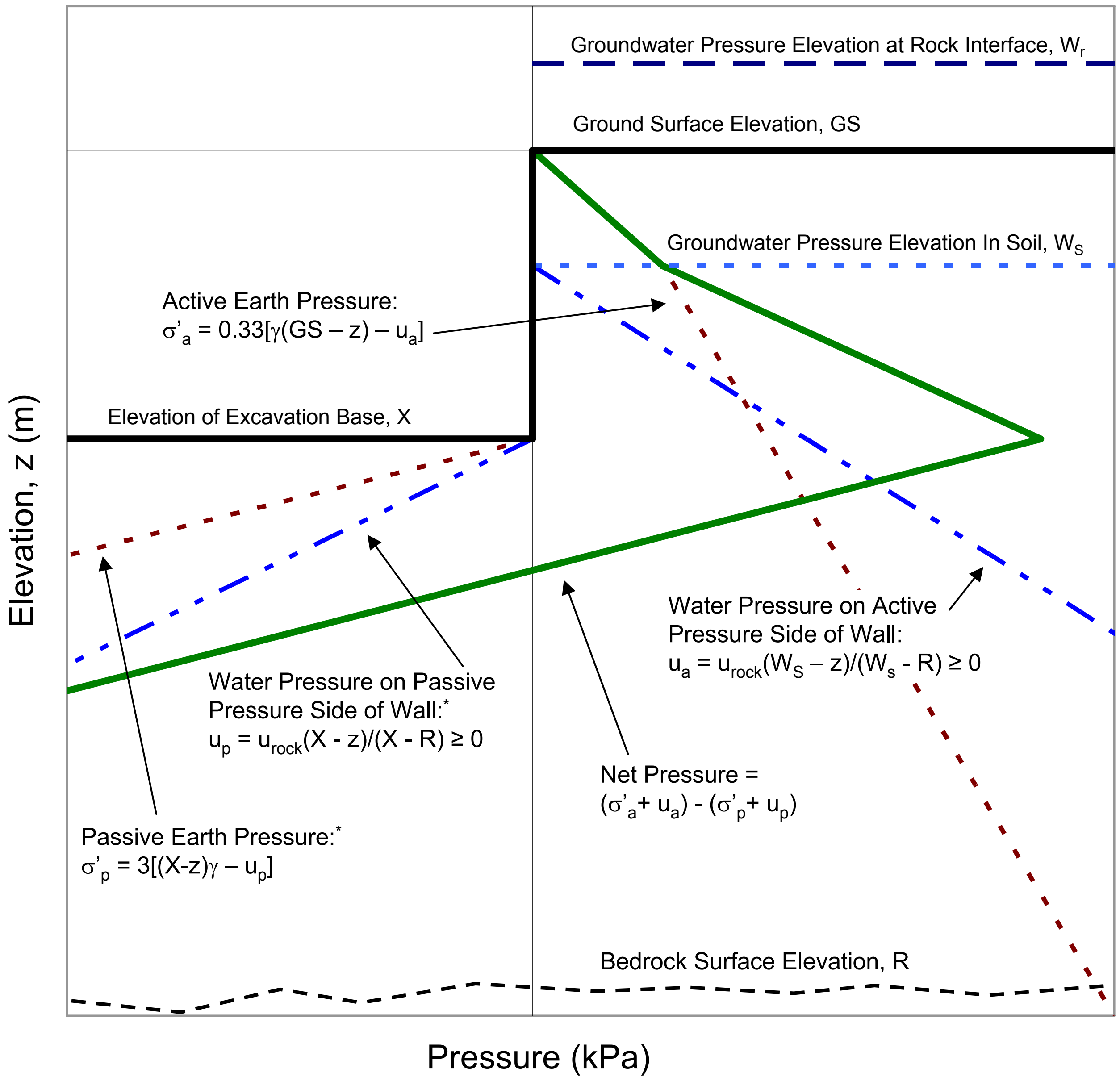
PROJECT
DETROIT RIVER INTERNATIONAL CROSSING

TITLE
EXCAVATION STABILITY, CUT AND COVER TUNNEL

Golder Associates
Mississauga, Ontario

PROJECT No. 04-1111-060			SCALE 1: 15,000	REV. 2
DESIGN	AW	15 Mar. 2005	FIGURE: 16	
GIS	CC	27 Sep. 2007		
CHECK	SB	27 Sep. 2007		
REVIEW	SB	27 Sep. 2007		

G:\Projects\2004\04-1111-060_Windsor_tunnel\GIS\MXDs\Draft\June2007_edits\Fig16_ExcavationStability_CutCoverTunnel.mxd



*Provided: $21(X - R)/[9.81(W_r - R)] \geq 1.1$ and that soil type between X and R is Silty Clay/Clayey Silt deposit as identified on Figure 2. If gravel, sand and silt (granular) deposits exist between base of Silty Clay/Clayey Silt deposit, R shall be taken as the top of such granular deposits.

Notes:

1. This figure is to be read with the accompanying reports “Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor” prepared by Golder Associates, October 2007.

Golder Associates

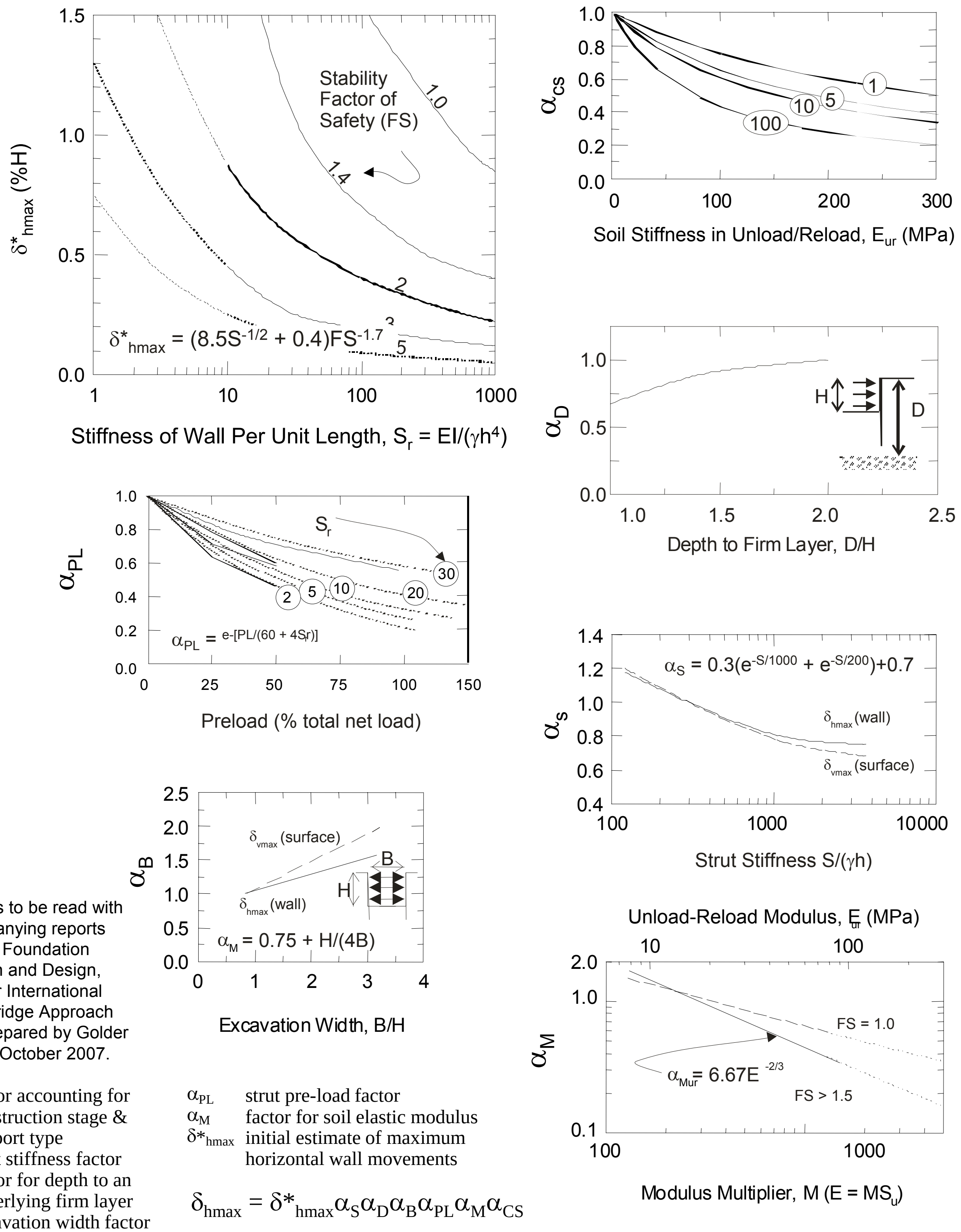
FILE: Excavations.ppt
PROJECT NO: 04-1111-060

SCALE	NTS
DATE	June 2007
DESIGN	SJB
CAD	SJB
CHECK	JW
REVIEW	FJH

LATERAL PRESSURES FOR PRELIMINARY DESIGN OF EXCAVATION SUPPORT

DETROIT RIVER
INTERNATIONAL CROSSING

FIGURE
17



Notes:

1. This figure is to be read with the accompanying reports "Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor" prepared by Golder Associates, October 2007.

α_{cs}	factor accounting for construction stage & support type
α_S	strut stiffness factor
α_D	factor for depth to an underlying firm layer
α_B	excavation width factor
α_{PL}	strut pre-load factor
α_M	factor for soil elastic modulus
δ^*_{hmax}	initial estimate of maximum horizontal wall movements

$$\delta_{hmax} = \delta^*_{hmax}\alpha_S\alpha_D\alpha_B\alpha_{PL}\alpha_M\alpha_{CS}$$

Golder Associates

SCALE	NTS
DATE	June 2007
DESIGN	SJB
CAD	SJB
CHECK	JW
REVIEW	FJH

FILE: Excavations.ppt

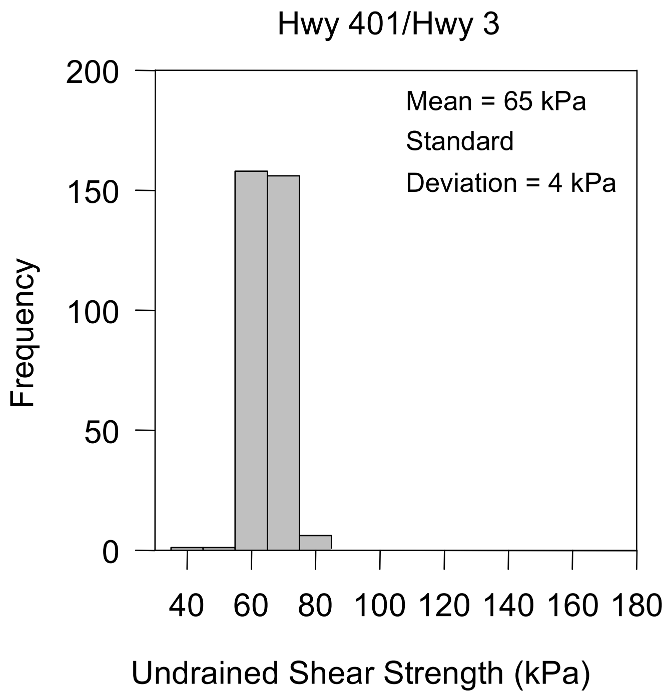
PROJECT NO: 04-1111-060

FACTORS CONTRIBUTING TO DEEP EXCAVATION-INDUCED DISPLACEMENTS

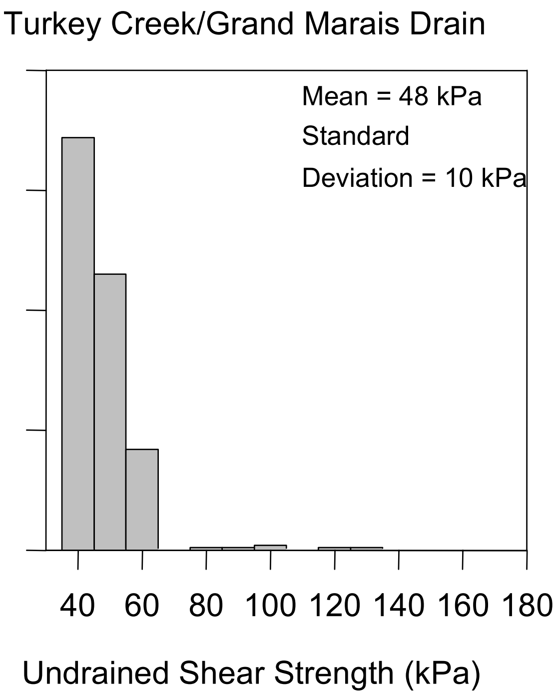
DETROIT RIVER
INTERNATIONAL CROSSING

FIGURE

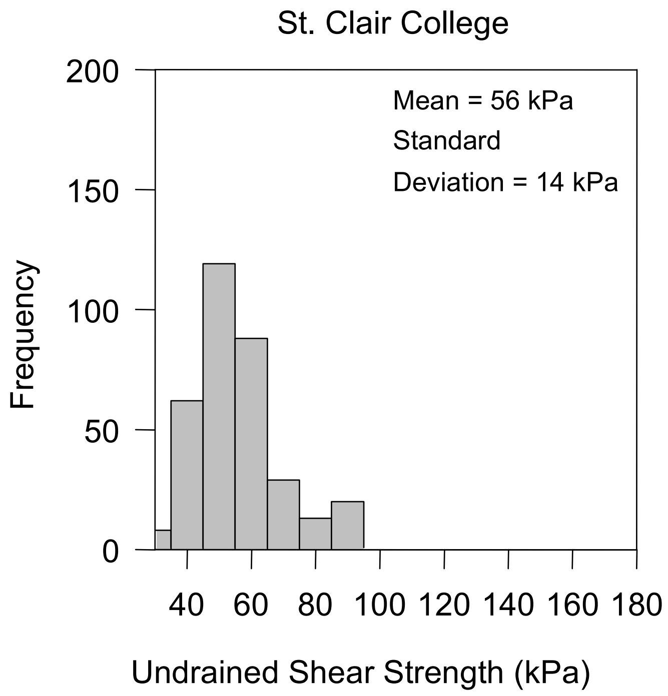
18


Hwy 401/Hwy 3

Mean = 65 kPa
Standard
Deviation = 4 kPa


Turkey Creek/Grand Marais Drain

Mean = 48 kPa
Standard
Deviation = 10 kPa


St. Clair College

Mean = 56 kPa
Standard
Deviation = 14 kPa

Notes:

1. This figure is to be read with the accompanying reports “Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor” prepared by Golder Associates, October 2007.
2. Frequency histograms are based on a depth of excavation of 14 m below ground surface.

Golder Associates

FILE: Excavations.ppt

PROJECT NO: 04-1111-060

SCALE NTS

DATE June 2007

DESIGN SJB

CAD SJB

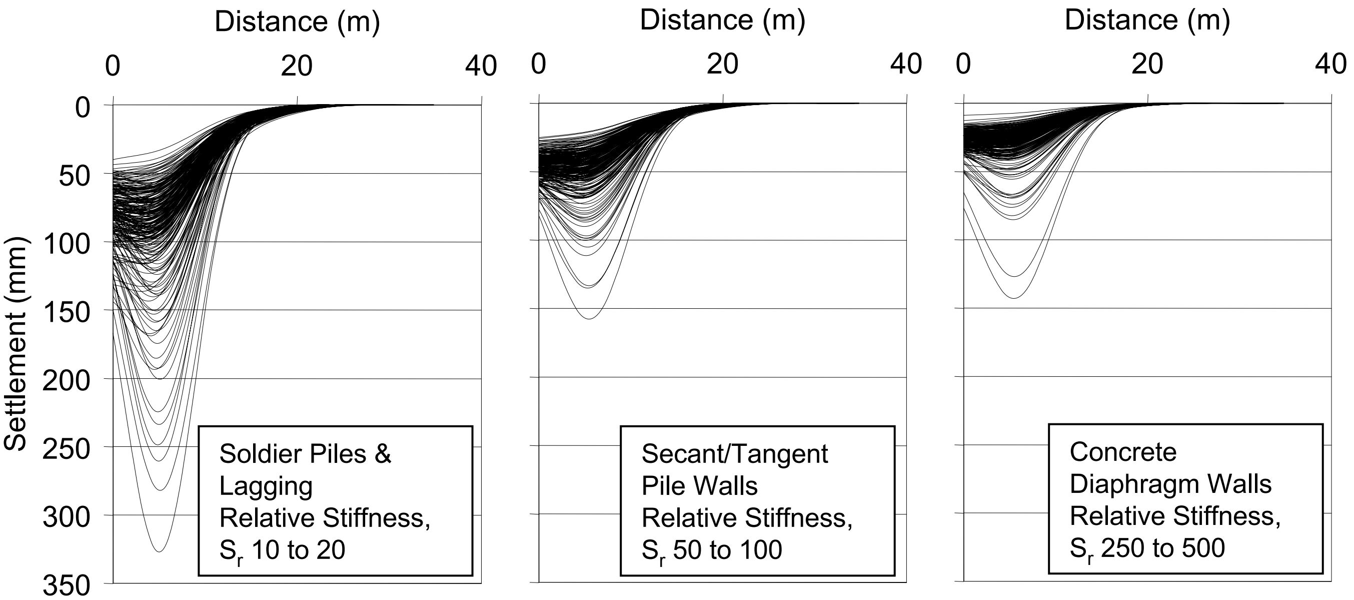
CHECK JW

REVIEW FJH

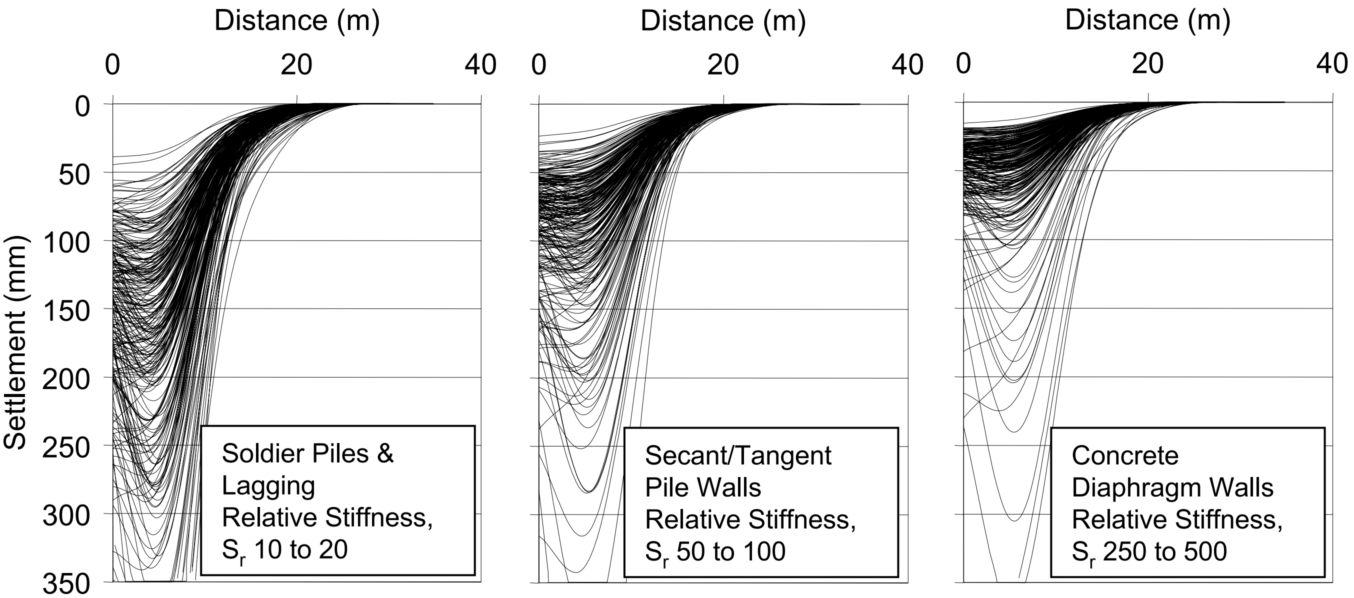
FREQUENCY HISTOGRAMS OF SOIL STRENGTH AT AND BELOW BASE OF EXCAVATION

DETROIT RIVER
INTERNATIONAL CROSSING

FIGURE
19



Vicinity of Highway 401/Highway 3 Intersection



Vicinity of Turkey Creek/Grand Marais Drain

Notes:

1. This figure is to be read with the accompanying reports "Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor" prepared by Golder Associates, October 2007.
2. Displacement estimates assume relative wall stiffness values as shown, target preloading of supports of 50%, design base stability factor of safety is 1.3 with stability enhancements in areas where factor of safety is less than 1.3, internal struts removed during construction, no other displacement controls, and soil properties as presented in reports referenced above.

Golder Associates

FILE: Excavations.ppt

PROJECT NO: 04-1111-060

SCALE NTS

DATE June 2007

DESIGN SJB

CAD SJB

CHECK JW

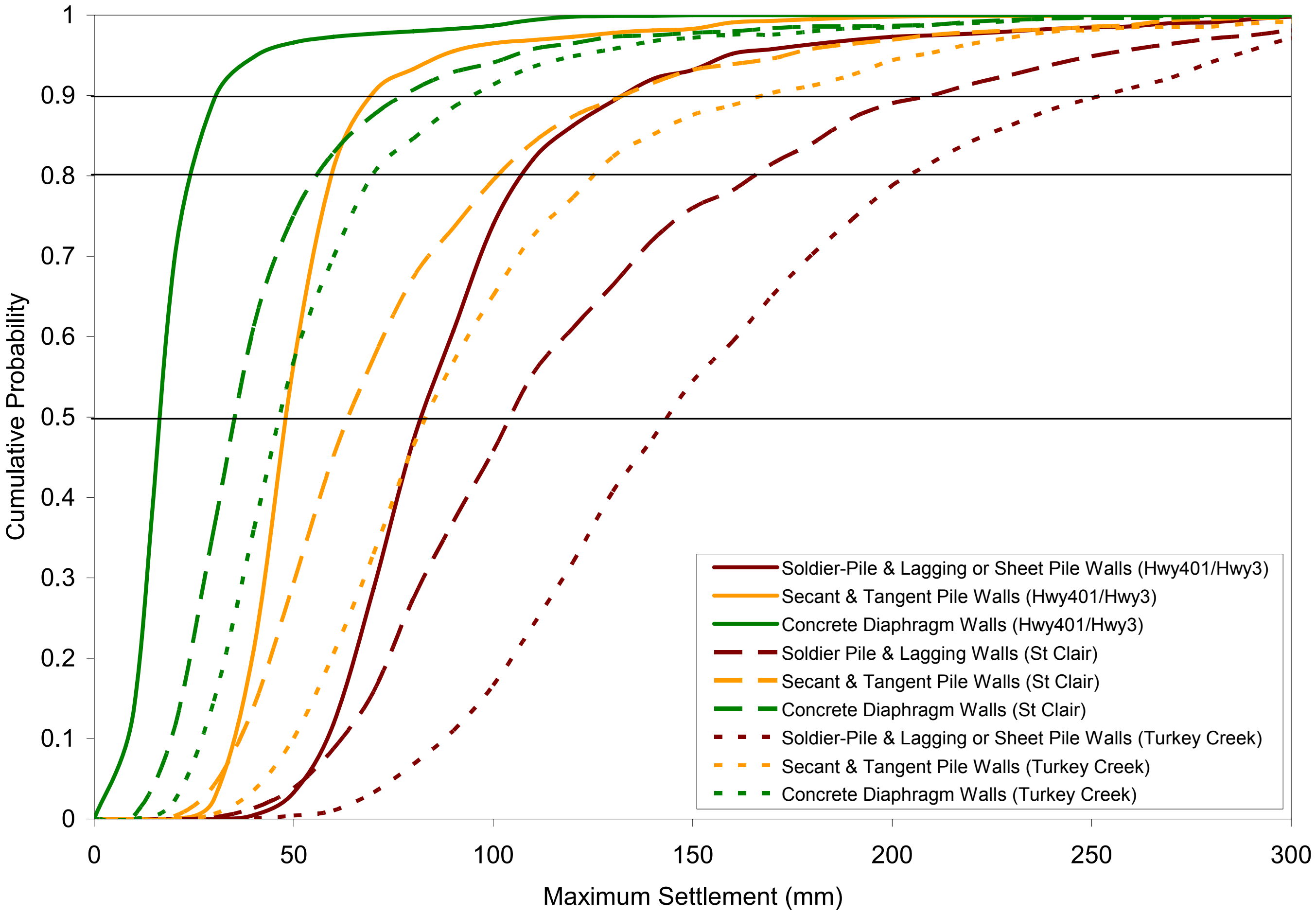
REVIEW FJH

ESTIMATED DISPLACEMENTS DUE TO DEEP EXCAVATIONS EXAMPLE PROFILES

DETROIT RIVER
INTERNATIONAL CROSSING

FIGURE

20



Notes:

1. This figure is to be read with the accompanying reports “Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor” prepared by Golder Associates, October 2007.
2. Displacement estimates assume relative wall stiffness values as shown in Fig 21, target preloading of supports of 50%, design base stability factor of safety is 1.3, internal struts removed during construction, no other displacement controls, and soil properties as presented in report referenced above.

Golder Associates

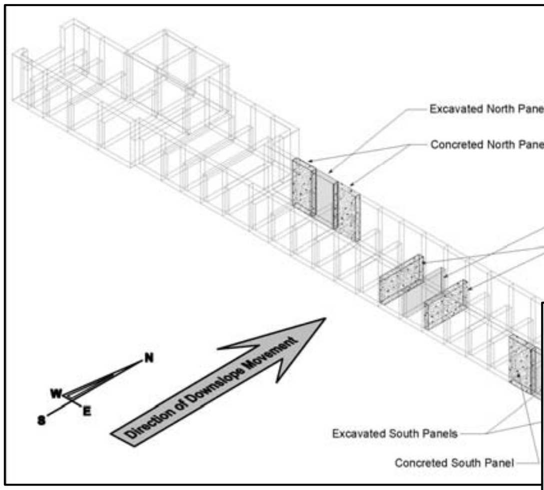
FILE:	Excavations.ppt
PROJECT NO:	04-1111-060

SCALE	NTS
DATE	June 2007
DESIGN	SJB
CAD	SJB
CHECK	JW
REVIEW	FJH

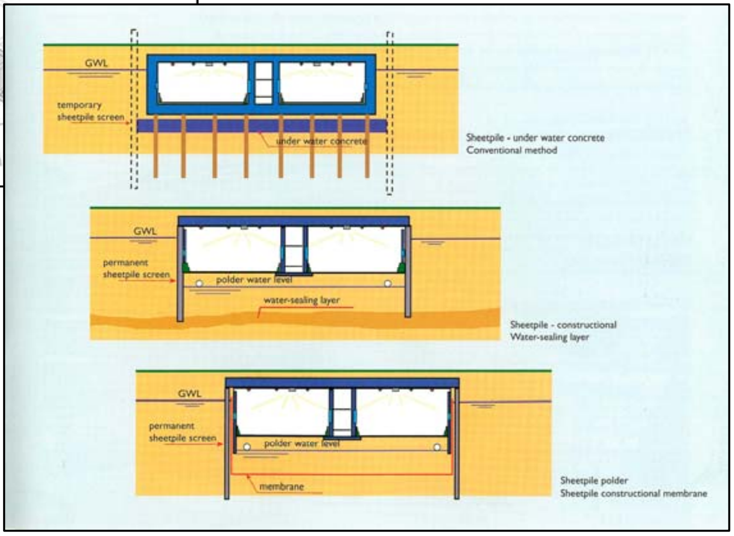
ESTIMATED DISPLACEMENTS DUE TO DEEP EXCAVATIONS CUMULATIVE PROBABILITY

DETROIT RIVER
INTERNATIONAL CROSSING

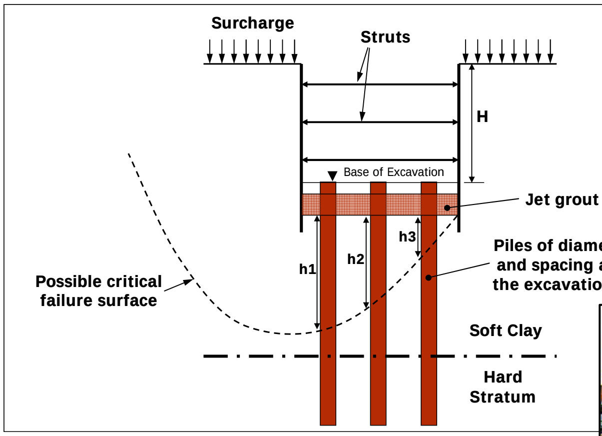
FIGURE
21



Isometric view of stability improvement by use of cross-excavation support walls below the excavation level constructed using diaphragm wall techniques (Dittrich 2000)



Stability improvement by use of tension piles and tremie-constructed base slabs (top), groundwater barriers, and dewatering (middle and bottom). Dewatering and barrier methods not applicable to DRIC project (van Beek 2003)



Stability improvement by use of tension piles and jet-grout base slabs.

Notes:

1. This figure is to be read with the accompanying reports “Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor” prepared by Golder Associates, October 2007.

Golder Associates

FILE: Excavations.ppt

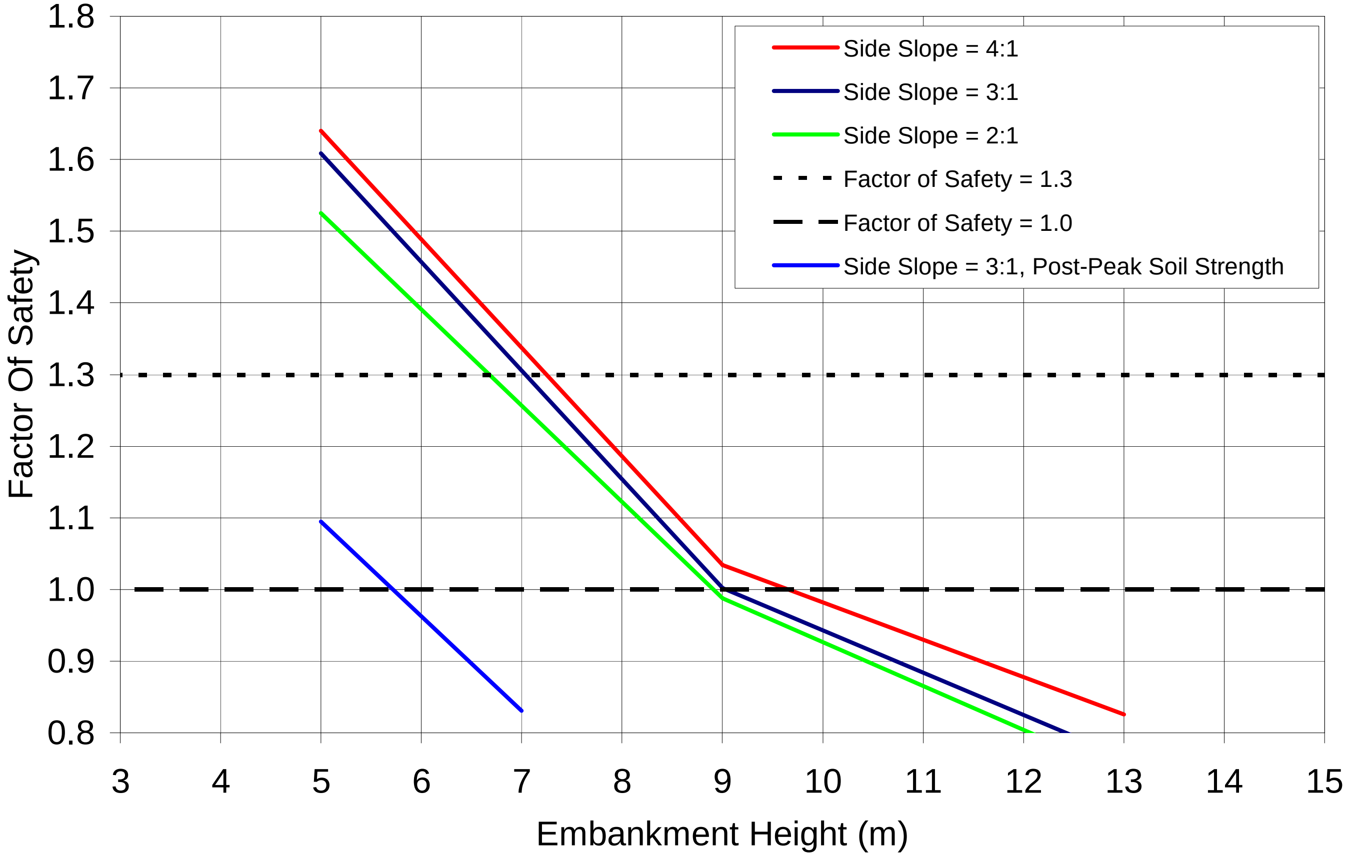
PROJECT NO: 04-1111-060

SCALE	NTS
DATE	June 2007
DESIGN	SJB
CAD	SJB
CHECK	JW
REVIEW	FJH

METHODS FOR IMPROVING EXCAVATION STABILITY AND DISPLACEMENT PERFORMANCE

DETROIT RIVER
INTERNATIONAL CROSSING

FIGURE
22



Notes:

1. This figure is to be read with the accompanying reports “Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor” prepared by Golder Associates, October 2007.
2. See report text for limitations of analysis.

Golder Associates

FILE: Embankments.ppt

PROJECT NO: 04-1111-060

SCALE NTS

DATE June 2007

DESIGN SJB

CAD SJB

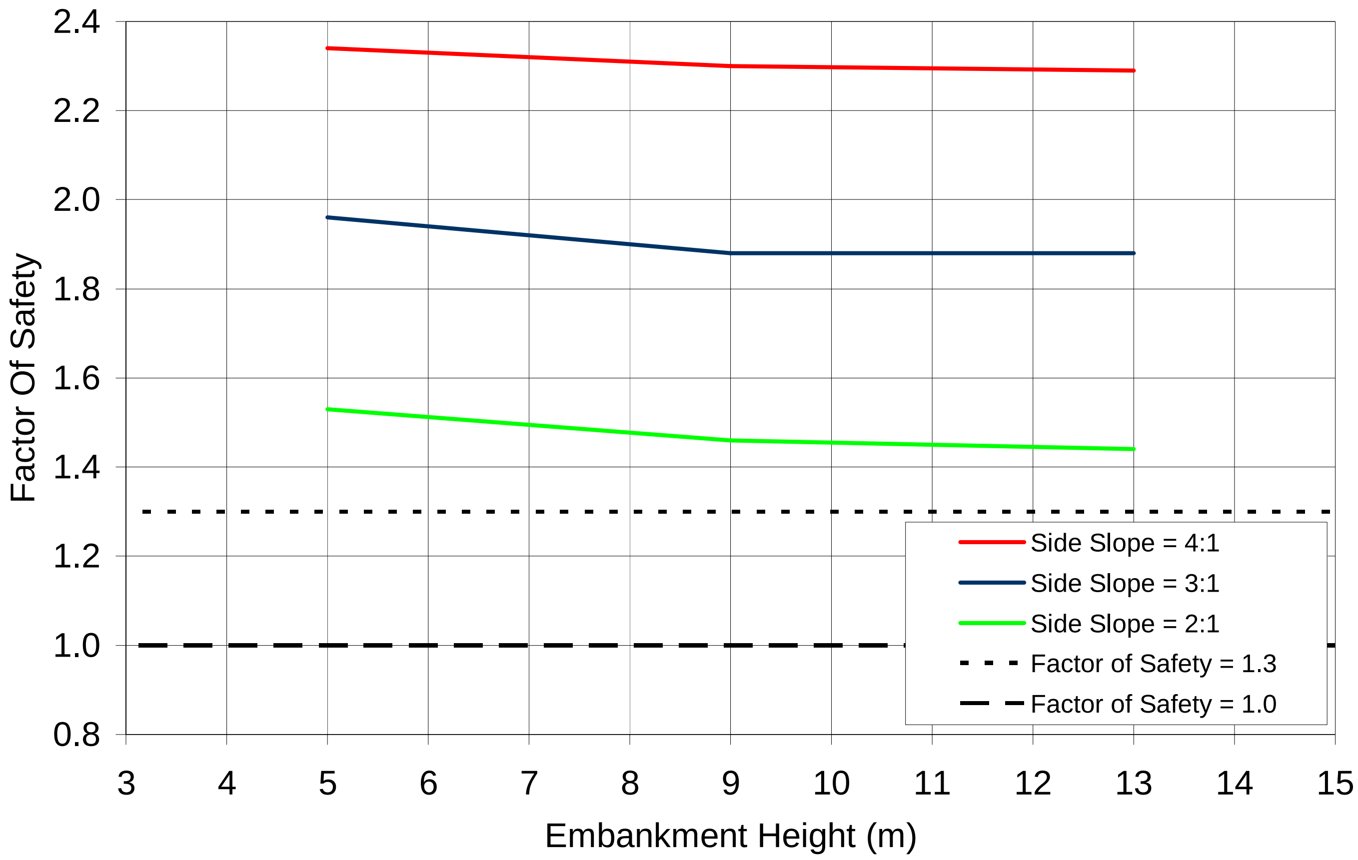
CHECK JW

REVIEW FJH

EMBANKMENT STABILITY IMMEDIATE CONSTRUCTION CONDITIONS

DETROIT RIVER
INTERNATIONAL CROSSING

FIGURE 23



Notes:

1. This figure is to be read with the accompanying reports “Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor” prepared by Golder Associates, October 2007.
2. See report text for limitations of analysis.

Golder Associates

FILE: Embankments.ppt

PROJECT NO: 04-1111-060

SCALE NTS

DATE June 2007

DESIGN SJB

CAD SJB

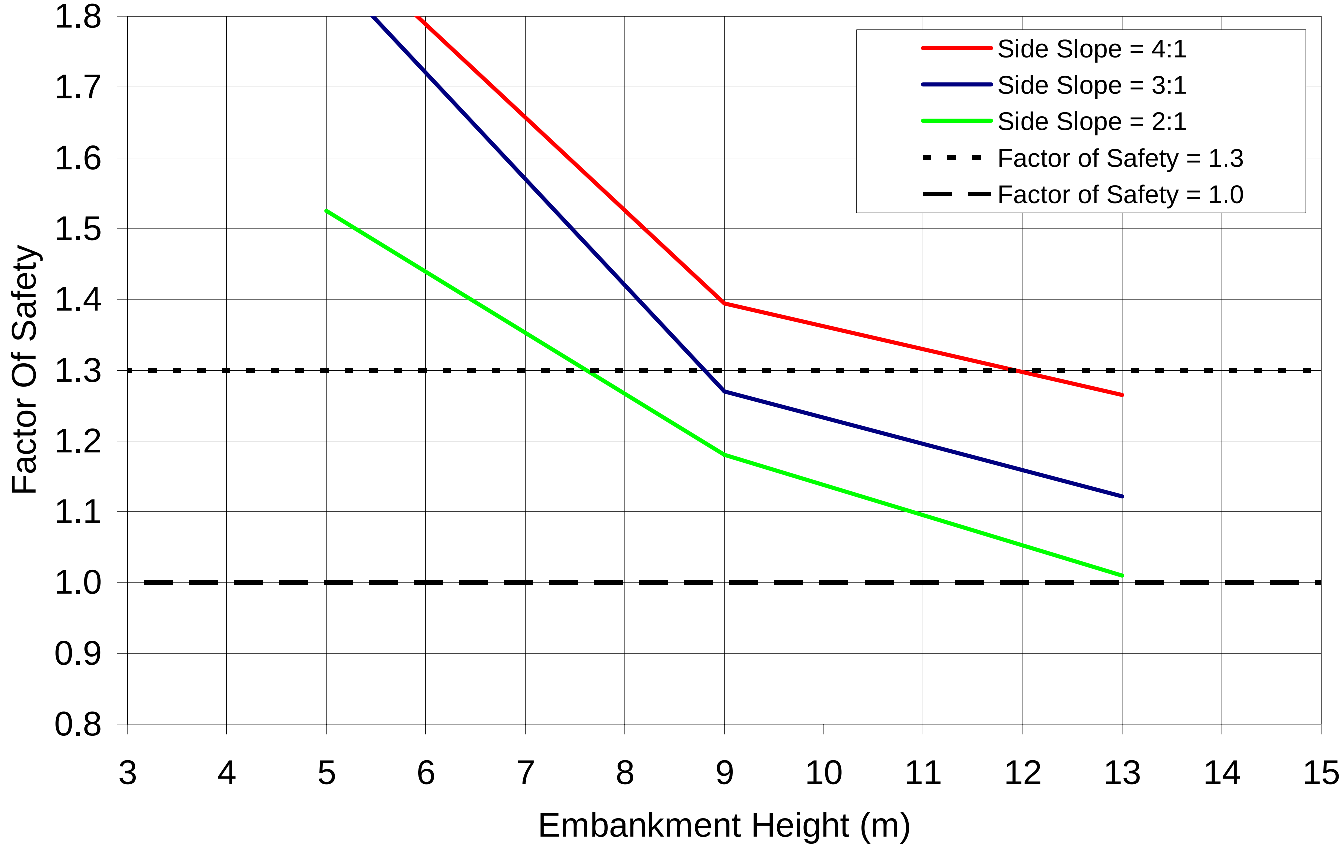
CHECK JW

REVIEW FJH

EMBANKMENT STABILITY LONG-TERM CONDITIONS

DETROIT RIVER
INTERNATIONAL CROSSING

FIGURE 24



Notes:

1. This figure is to be read with the accompanying reports “Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor” prepared by Golder Associates, October 2007.
2. See report text for limitations of analysis.

Golder Associates

FILE: Embankments.ppt

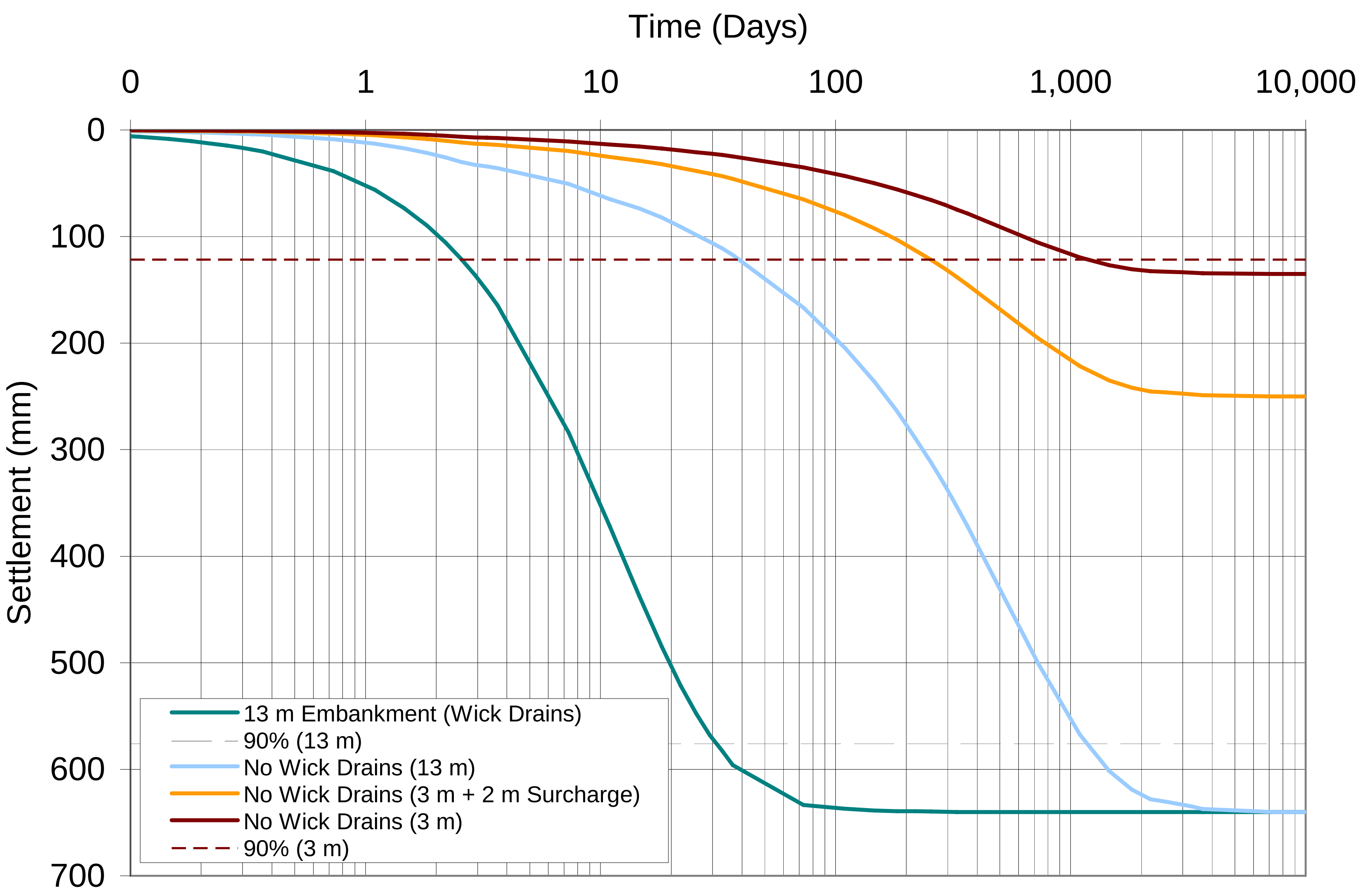
PROJECT NO: 04-1111-060

SCALE	NTS
DATE	June 2007
DESIGN	SJB
CAD	SJB
CHECK	JW
REVIEW	FJH

EMBANKMENT STABILITY
STAGED CONSTRUCTION CONDITIONS

DETROIT RIVER
INTERNATIONAL CROSSING

FIGURE **25**



Notes:

1. This figure is to be read with the accompanying reports "Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor" prepared by Golder Associates, October 2007.
2. See report text for limitations of analysis.

Golder Associates

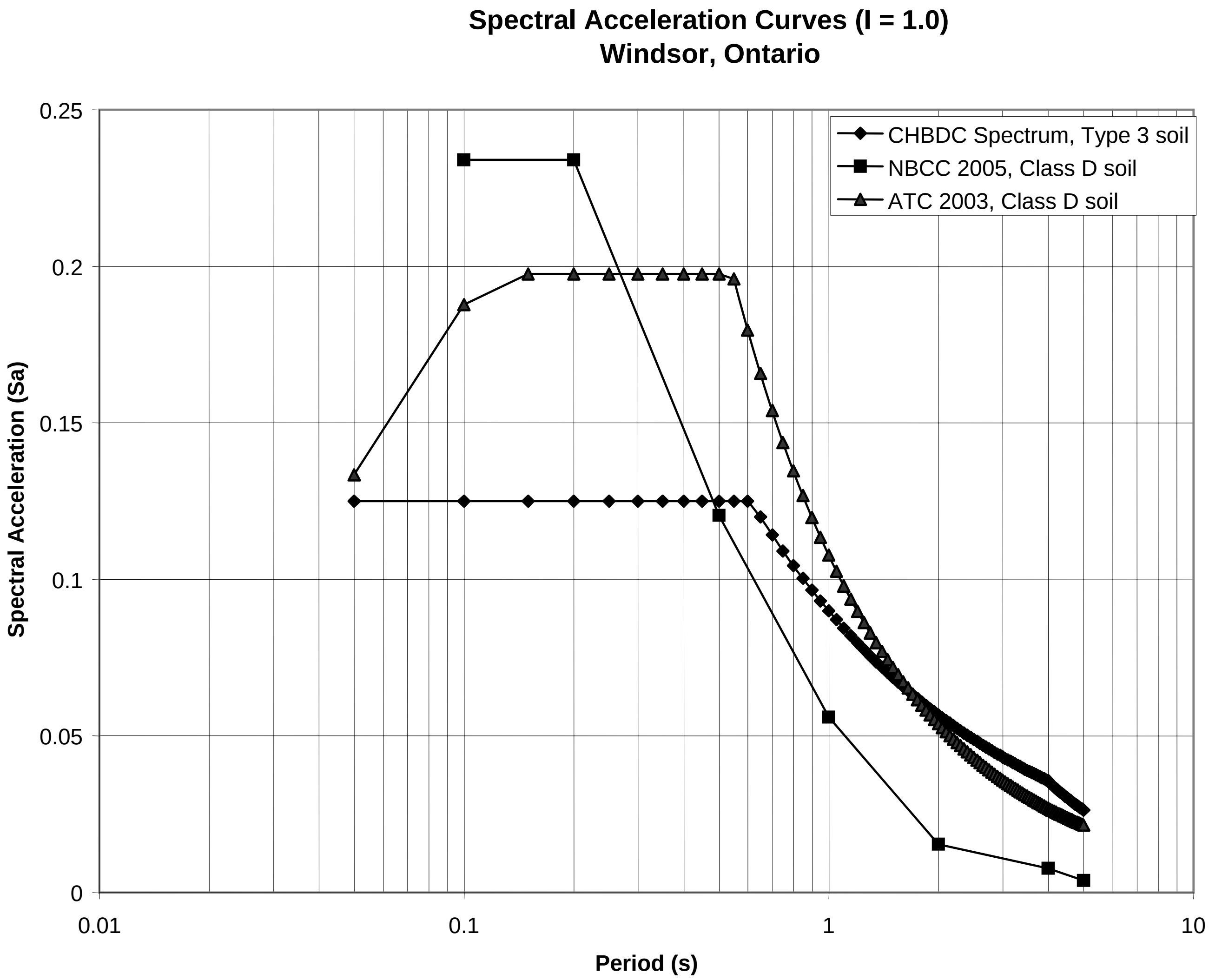
FILE:	Embankments.ppt
PROJECT NO:	04-1111-060

SCALE	NTS
DATE	June 2007
DESIGN	SJB
CAD	SJB
CHECK	JW
REVIEW	FJH

EMBANKMENT SETTLEMENT

DETROIT RIVER
INTERNATIONAL CROSSING

FIGURE
26



Notes:
1. This figure is to be read with the accompanying report “Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor” prepared by Golder Associates, October 2007.
2. See report text for limitations of analysis.

Golder Associates

FILE: Spectral.ppt
PROJECT NO: 04-1111-060

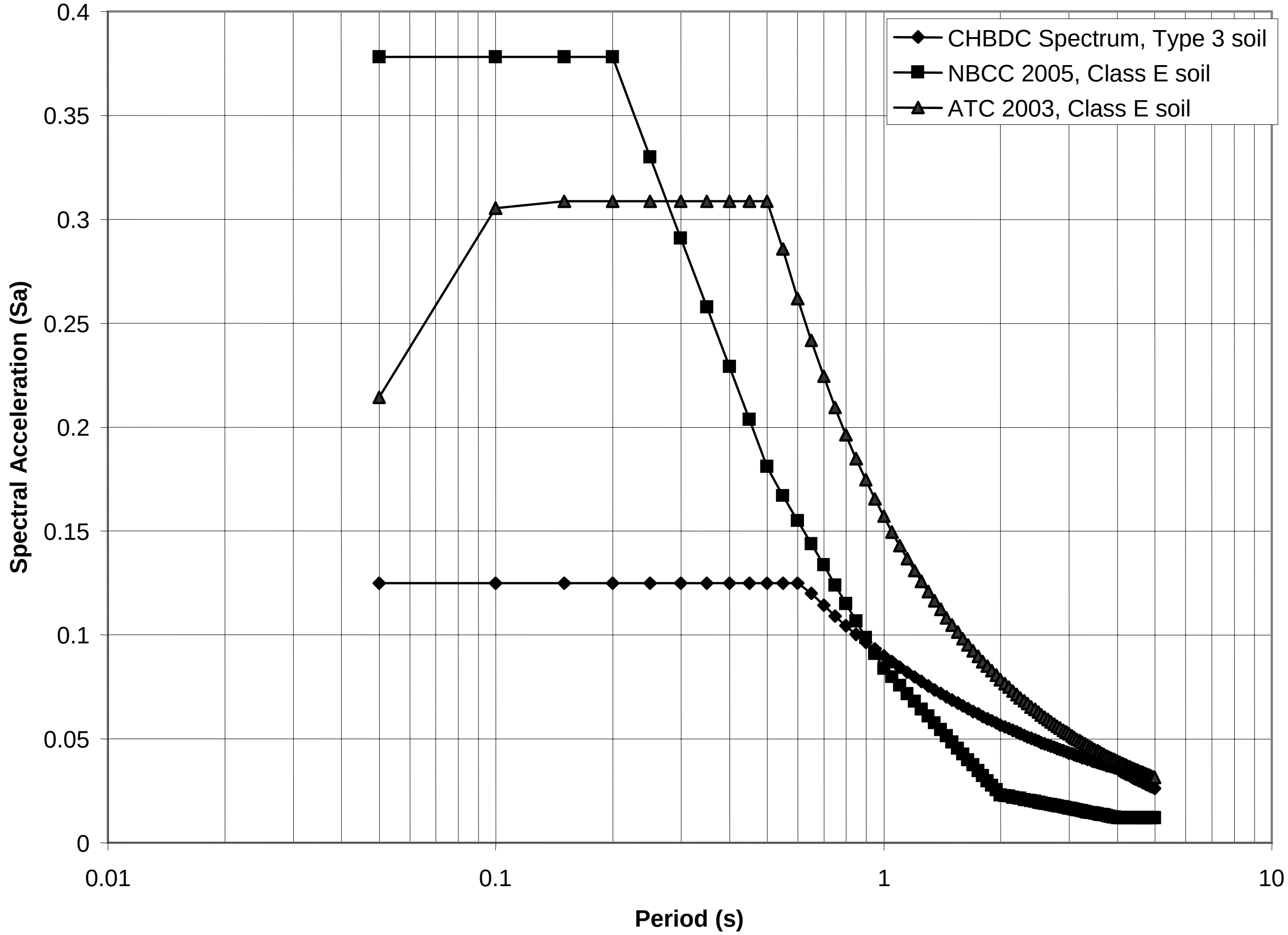
SCALE As Shown
DATE June 2007
DESIGN JS
CAD
CHECK MS
REVIEW SJB/FJH

SPECTRAL ACCELERATION CURVES
Class D Soil

DETROIT RIVER
INTERNATIONAL CROSSING

FIGURE 27

Spectral Acceleration Curves (I = 1.0)
Windsor, Ontario



Notes:

1. This figure is to be read with the accompanying report “Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor” prepared by Golder Associates, October 2007.
2. See report text for limitations of analysis.

Golder Associates

FILE: Spectral.ppt

PROJECT NO: 04-1111-060

SCALE As Shown

DATE June 2007

DESIGN JS

CAD

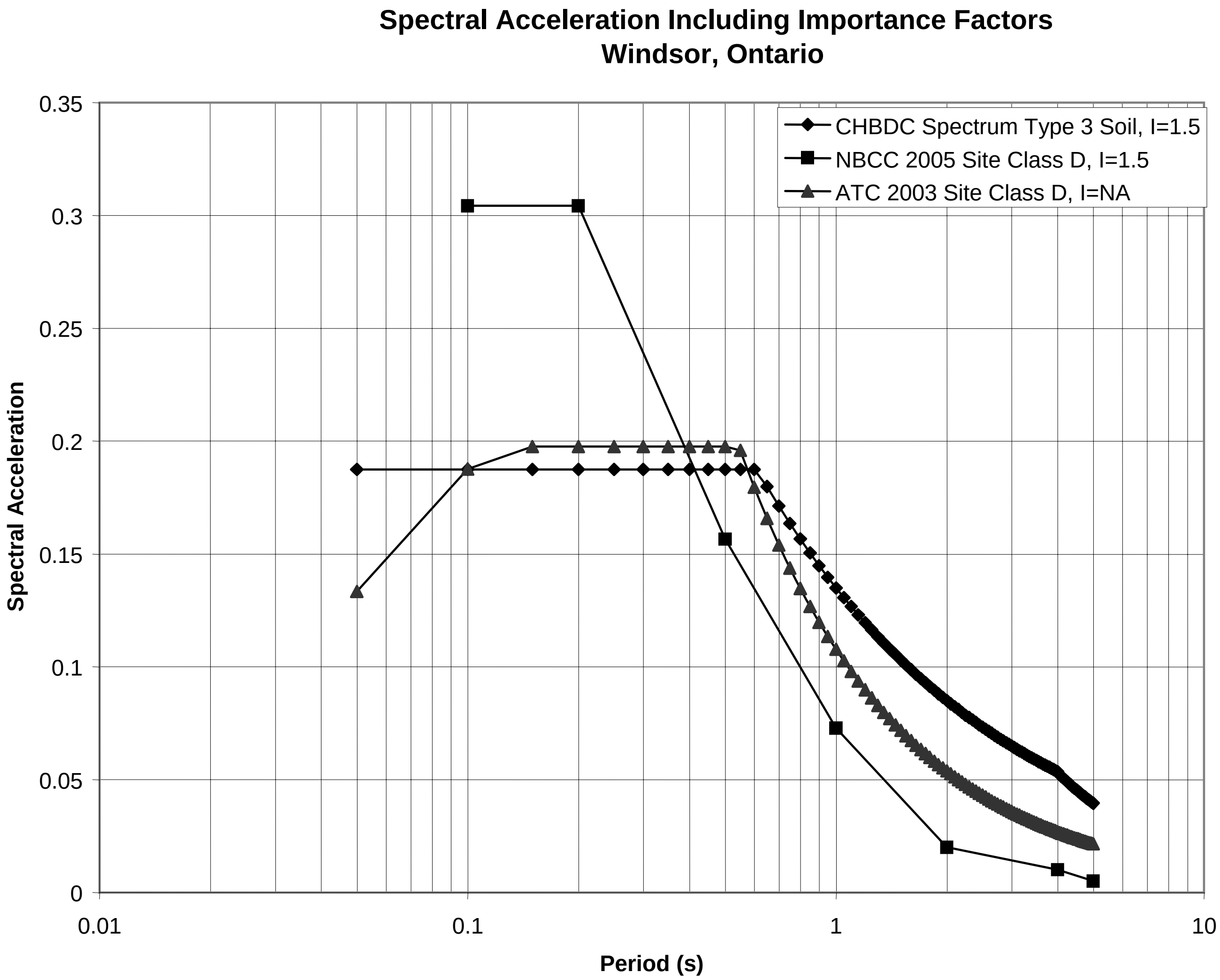
CHECK MS

REVIEW SJB/FJH

SPECTRAL ACCELERATION CURVES
Class E Soil

DETROIT RIVER
INTERNATIONAL CROSSING

FIGURE **28**



Notes:

1. This figure is to be read with the accompanying report "Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor" prepared by Golder Associates, October 2007.
2. See report text for limitations of analysis.

Golder Associates

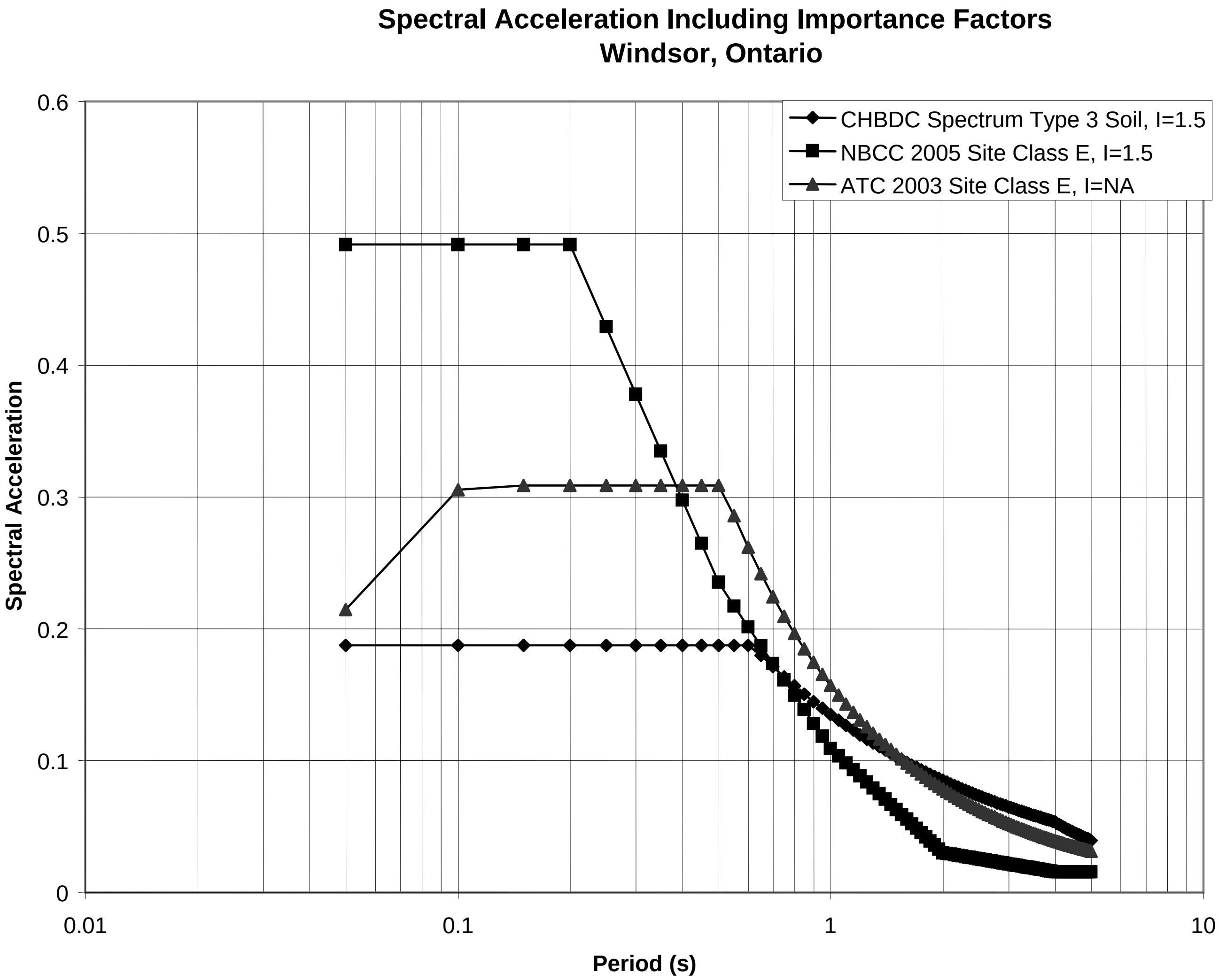
FILE:	Spectral.ppt
PROJECT NO:	04-1111-060

SCALE	As Shown
DATE	June 2007
DESIGN	JS
CAD	
CHECK	MS
REVIEW	SJB/FJH

SPECTRAL ACCELERATION CURVES
Class D Soil Including Importance Factors

DETROIT RIVER
INTERNATIONAL CROSSING

FIGURE
29



Notes:

1. This figure is to be read with the accompanying report “Preliminary Foundation Investigation and Design, Detroit River International Crossing, Bridge Approach Corridor” prepared by Golder Associates, October 2007.
2. See report text for limitations of analysis.

Golder Associates

SCALE	As Shown
DATE	June 2007
DESIGN	JS
CAD	
CHECK	MS
REVIEW	SJB/FJH

FILE: Spectral.ppt

PROJECT NO: 04-1111-060

SPECTRAL ACCELERATION CURVES
Class E Soil Including Importance Factors

DETROIT RIVER
INTERNATIONAL CROSSING

FIGURE 30

APPENDIX A

LABORATORY TEST DATA

LABORATORY TEST DATA SUMMARY
Detroit River International Crossing

BOREHOLE No.	SAMPLE No.	Depth (m)	w_n (%)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	LL	PL	PI	γ_{sat} (kN/m^3)	g_s	σ'_{vo} (kPa)	σ'_p (kPa)	C_c	e_o	σ'_c (kPa)	S_u (kPa)	Dolomite (%)	Calcite (%)	Total (%)	UC (MPa)
1	1A	0.28	19.6																			
1	1B	1.63	17.6																			
1	2	1.83	14.8																			
1	3	4.13	12.1																			
1	4	5.72	15.9																			
1	5	7.25	15.6	9	30	36	25	26.1	13.5	12.6	21.1	2.73	131	304	0.1383	0.49	35	50.7	14.8	11.0	25.8	
1	6	8.76	17.8																			
1	7	10.29	16.3					22.7	12.3	10.4												
1	8	11.81	16.6																			
1	10	13.95	18.4	2	28	41	29	27.3	15.1	12.2	20.5	2.73	199	276	0.1573	0.56	70	56.5	13.9	11.0	24.9	
1	11	14.86	25.2																			
1	12	16.39	20.6					28.8	15.4	13.4												
1	13	17.91	13.6																			
1	14	19.44	25.9	3	23	44	30	27.2	14.4	12.8	20.0	2.76	247	351	0.1594	0.66	95	61.1	17.1	6.7	23.8	
1	15	20.96	27.0																			
1	16	22.49	12.6					21.3	13.1	8.2												
1	17	24.01	13.7																			
1	18	25.54	13.7					23.7	13.6	10.1												
1	20	27.67	17.6					28.0	15.9	12.1												
1	21	28.58	18.3																			
1	23	30.72	24.1					33.5	17.5	16.0												
1	24	31.63	24.8																			
1	27	34.30																				49.2
7	1A	0.25	20.2																			
7	1B	0.43	13.0																			
7	2	1.83	13.3																			
7	3	2.67	13.3																			
7	4	4.19	14.7																			
7	5	5.81	14.7					22.9	13.8	9.1												

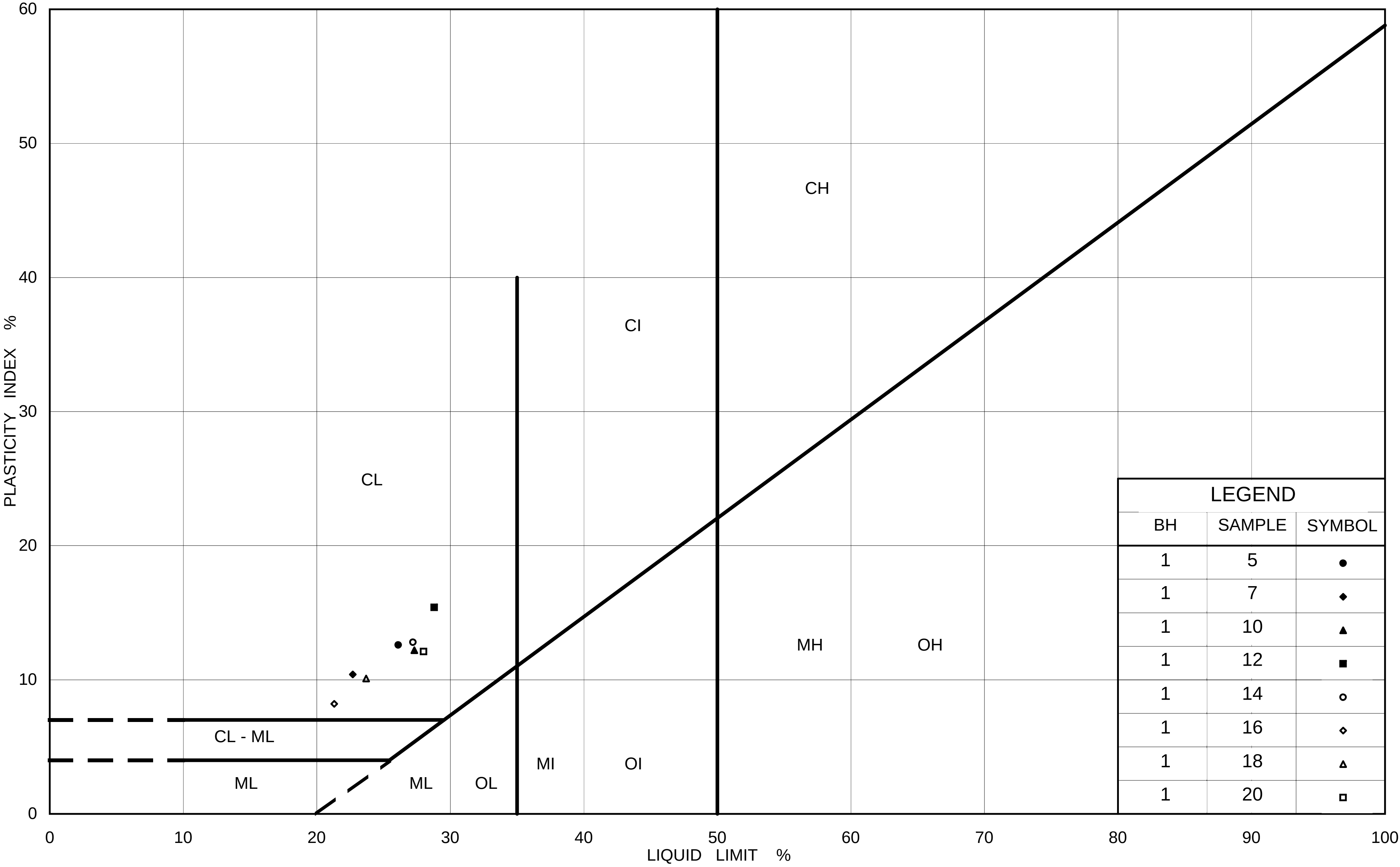
LABORATORY TEST DATA SUMMARY
Detroit River International Crossing

BOREHOLE No.	SAMPLE No.	Depth (m)	w_n (%)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	LL	PL	PI	γ_{sat} (kN/m^3)	g_s	σ'_{vo} (kPa)	σ'_p (kPa)	C_c	e_o	σ'_c (kPa)	S_u (kPa)	Dolomite (%)	Calcite (%)	Total (%)	UC (MPa)
7	6	7.24	14.2																			
7	7	8.70	16.9	1	31	39	30	22.9	13.3	9.6	21.4	2.73	148	277	0.1323	0.46	43	50.9	14.4	10.9	25.3	
7	8	10.29	18.6																			
7	9	11.75	18.3					28.8	15.3	13.5												
7	10	13.34	21.7																			
7	11	14.80	21.7	3	19	42	36	29.7	15.4	14.3	20.7	2.76	208	304	0.2367	0.59	73	58.1	14.9	8.6	23.5	
7	12	16.40	37.3					42.9	21.9	21.0												
7	13	17.91	19.8																			
7	15	19.98	15.5					21.1	13.2	7.9												
7	16	20.96	7.4																			
7	17	22.42	15.9					26.0	14.7	11.3												
7	18	24.01	17.6																			
7	19	25.53	21.7																			
7	20	26.22	19.6	2	19	42	37	29.1	14.9	14.2	21.0	2.74	340	368	0.2018	0.53	130	79.7	15.8	11.3	27.1	
7	21	27.05	20.6																			
7	22	28.52	27.5					28.4	16.1	12.3												
7	23	30.10	28.2																			
7	24	31.52	13.0					19.0	11.9	7.1												
7	25	33.00	11.1																			
7	29	37.20																				33.3
14	1	0.30	16.1																			
14	2	1.83	22.0																			
14	4	4.22	26.1					41.1	20.7	20.4												
14	6	6.36	22.8					32.0	16.4	15.6												
14	6	6.58	22.8																			
14	7	7.34	21.7																			
14	8	8.79	21.7																			
14	9	10.31	20.1	3	26	41	30	28.6	15.2	13.4	21.0	2.75	164	276	0.1755	0.54	50	50.2	12.3	11.2	23.5	
14	9	10.54	20.6																			
14	10	11.91	21.6																			
14	11	13.37	21.8	2	23	39	36	28.0	15.7	12.3	20.2	2.76	187	278	0.1891	0.65	66	48.6	11.8	11.5	23.3	
14	12	14.96	28.8																			
14	13	17.02	17.4					56.3	25.2	31.1												
14	15	18.01	26.5																			
14	16	18.93	16.4	7	24	45	25	24.2	13.6	10.6	21.4	2.75	267	335	0.1231	0.47	93	66.4	18.3	13.3	31.6	
14	17	21.06	16.0																			

LABORATORY TEST DATA SUMMARY
Detroit River International Crossing

BOREHOLE No.	SAMPLE No.	Depth (m)	w_n (%)	Gravel (%)	Sand (%)	Silt (%)	Clay (%)	LL	PL	PI	γ_{sat} (kN/m^3)	g_s	σ'_{vo} (kPa)	σ'_p (kPa)	C_c	e_o	σ'_c (kPa)	S_u (kPa)	Dolomite (%)	Calcite (%)	Total (%)	UC (MPa)
14	18	22.51	18.6					28.5	15.8	12.7												
14	18	22.74	20.0																			
14	19	24.26	19.7																			
14	20	25.56	24.2					35.4	17.8	17.6												
14	21	27.46	14.5																			
14	23	29.22	26.2					29.5	15.7	13.8												
14	24	30.20	18.0																			
14	25A	31.58	20.3																			
14	25B	31.88	8.1																			
14	26	33.48	11.9																			
14	30	37.00																				36.4
23	1A	0.15	10.6																			
23	1B	0.46	14.8																			
23	2A	1.60	21.7																			
23	2B	1.91	17.6																			
23	3	2.74	21.2																			
23	4	4.19	24.2					27.6	17.0	10.6												
23	5	5.79	36.4																			
23	6	7.32	25.7																			
23	7	8.76	31.6	1	24	37	39	31.1	16.6	14.5	19.9	2.75	89	117	0.1866	0.71	43	28.1	10.7	8.8	19.5	
23	8	10.36	26.1																			
23	9	11.81	22.9					23.1	13.6	9.5												
23	10	13.41	25.8																			
23	11	14.86	15.5																			
23	13	16.99	18.4	2	20	44	34	29.0	15.5	13.5	21.2	2.74	195	276	0.1334	0.52	84	77.5	15.3	11.3	26.6	
23	14	17.99	16.4																			
23	15	19.43	20.5	4	16	40	40	33.2	17.9	15.3	21.6	2.75	230	335	0.1573	0.48	96	94.6	14.1	10	24.1	
23	16A	21.11	19.2																			
23	16B	21.11	14.0																			
23	17	22.41	19.6																			
23	20	24.50																				55.4

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
1	5	●
1	7	◆
1	10	▲
1	12	■
1	14	○
1	16	◇
1	18	△
1	20	□

Ministry of Transportation

Ontario

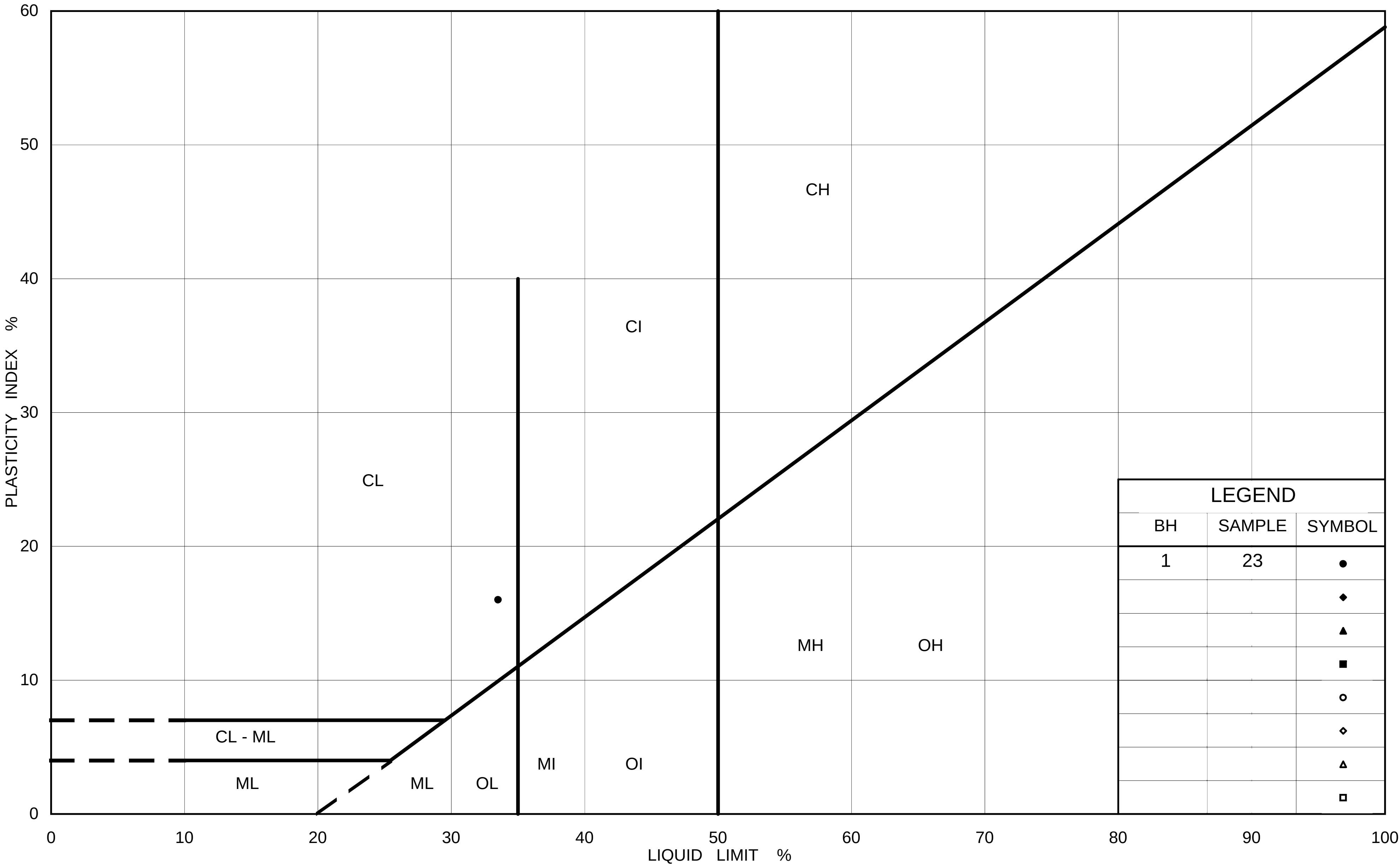
PLASTICITY CHART

FIG No. A1a

Project No. 04-1111-060

Checked By: SJB

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART

FIG No. A1b

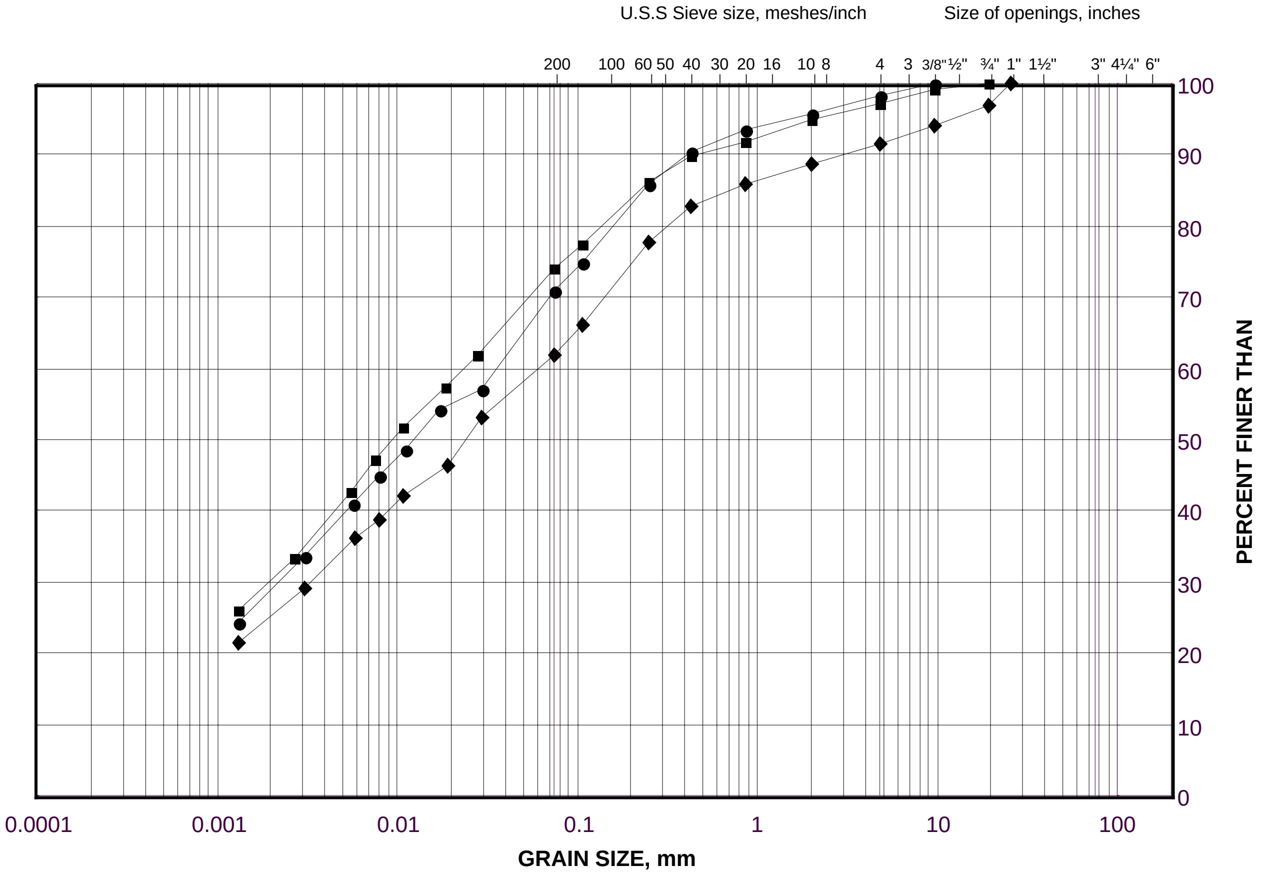
Project No. 04-1111-060

Checked By: SJB

GRAIN SIZE DISTRIBUTION

Clayey Silt to Silty Clay Deposit

FIGURE A2



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	1	10	13.64 - 14.25
■	1	14	19.13 - 19.74
◆	1	5	6.94 - 7.55

Project Number: 04-1111-060

Checked By: SJB

Golder Associates

Date: 06-Jun-07

OEDOMETER CONSOLIDATION SUMMARY

SAMPLE IDENTIFICATION

Project Number	04-1111-060	Sample Number	5
Borehole Number	1	Sample Depth, m	6.94-7.55

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	7		
Date Started	10/17/2006		
Date Completed	11/01/2006		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m^3	21.09
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	17.94
Area, cm^2	31.65	Specific Gravity, measured	2.73
Volume, cm^3	60.13	Solids Height, cm	1.273
Water Content, %	17.55	Volume of Solids, cm ^{3}	40.29
Wet Mass, g	129.31	Volume of Voids, cm ^{3}	19.84
Dry Mass, g	110.00	Degree of Saturation, %	97.3

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	0.492	1.900				
4.83	1.896	0.489	1.898	9	8.49E-02	4.36E-04	3.62E-06
9.46	1.893	0.487	1.895	15	5.07E-02	3.41E-04	1.70E-06
19.29	1.883	0.479	1.888	46	1.64E-02	5.35E-04	8.62E-07
38.58	1.869	0.468	1.876	46	1.62E-02	3.82E-04	6.07E-07
77.57	1.849	0.452	1.859	60	1.22E-02	2.70E-04	3.23E-07
154.88	1.815	0.426	1.832	14	5.08E-02	2.31E-04	1.15E-06
309.77	1.781	0.399	1.798	23	2.98E-02	1.16E-04	3.37E-07
619.14	1.739	0.366	1.760	19	3.46E-02	7.15E-05	2.42E-07
1237.66	1.692	0.329	1.716	98	6.37E-03	4.00E-05	2.50E-08
2475.57	1.639	0.287	1.666	89	6.61E-03	2.25E-05	1.46E-08
1237.66	1.648	0.294	1.644				
309.38	1.672	0.313	1.660				
77.57	1.695	0.331	1.684				
19.29	1.729	0.358	1.712				
4.83	1.755	0.379	1.742				

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

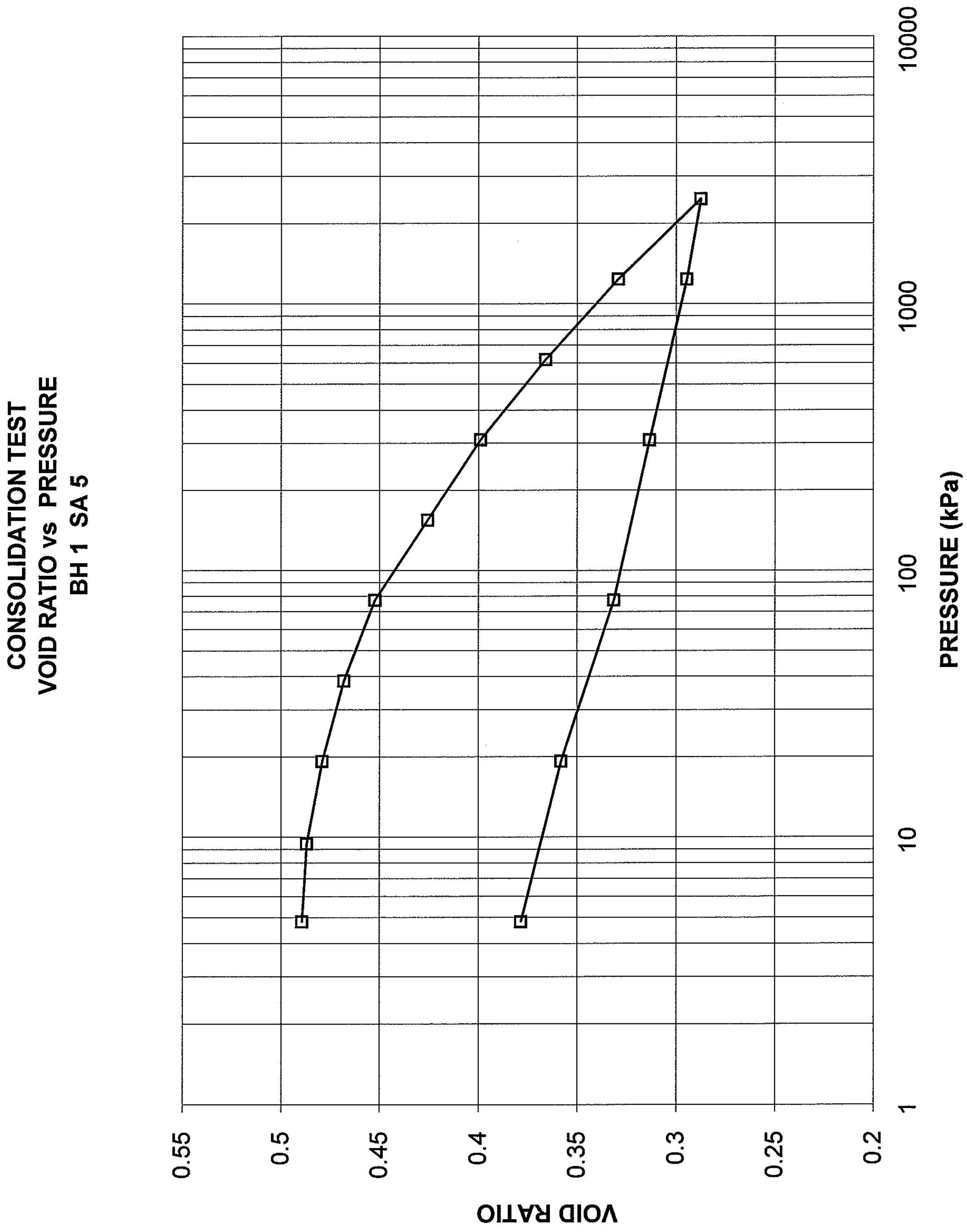
SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.76	Unit Weight, kN/m^3	22.29
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	19.42
Area, cm^2	31.65	Specific Gravity, measured	2.73
Volume, cm^3	55.54	Solids Height, cm	1.273
Water Content, %	14.78	Volume of Solids, cm ^{3}	40.29
Wet Mass, g	126.26	Volume of Voids, cm ^{3}	15.25
Dry Mass, g	110		

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

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE A3a



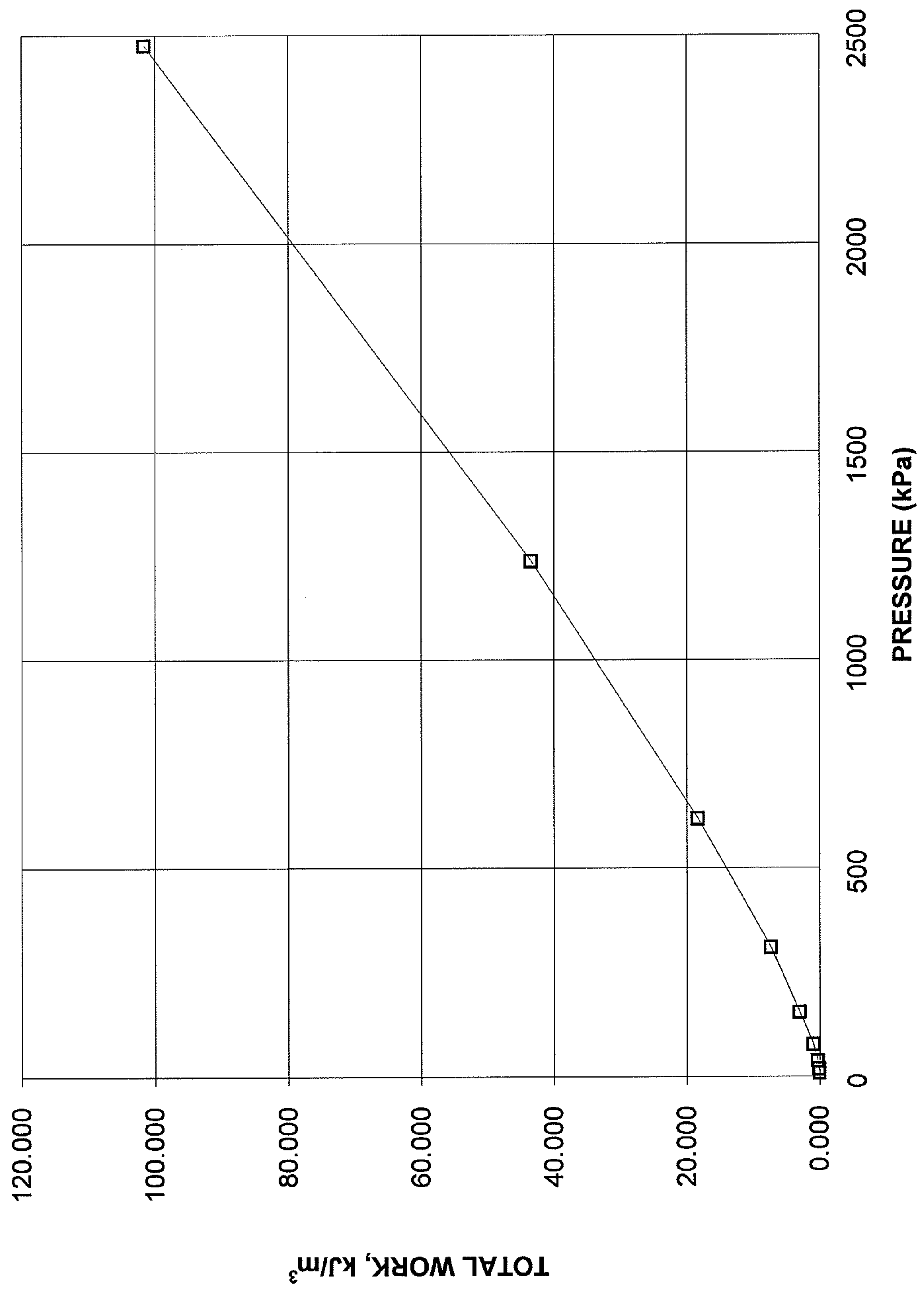
Project No. 04-1111-060

Prepared By: LFG

Golder Associates

Checked By: MM

FIGURE A3b



CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs PRESSURE
BH 1 SA 5

Project No. 04-1111-060

Golder Associates

Prepared By: LFG

Checked By: MM

OEDOMETER CONSOLIDATION SUMMARY

SAMPLE IDENTIFICATION

Project Number	04-1111-060	Sample Number	10
Borehole Number	1	Sample Depth, m	13.64-14.25

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	8		
Date Started	10/17/2006		
Date Completed	11/01/2006		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.92	Unit Weight, kN/m^3	20.54
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	17.11
Area, cm^2	31.67	Specific Gravity, measured	2.73
Volume, cm^3	60.65	Solids Height, cm	1.224
Water Content, %	20.02	Volume of Solids, cm ^{3}	38.76
Wet Mass, g	127.00	Volume of Voids, cm ^{3}	21.88
Dry Mass, g	105.82	Degree of Saturation, %	96.8

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	1.915	0.565	1.915				
4.84	1.911	0.561	1.913	17	4.56E-02	4.32E-04	1.93E-06
9.53	1.906	0.557	1.909	28	2.76E-02	5.57E-04	1.50E-06
18.98	1.897	0.550	1.902	60	1.28E-02	4.97E-04	6.23E-07
38.26	1.880	0.536	1.889	28	2.70E-02	4.60E-04	1.22E-06
77.43	1.857	0.517	1.869	19	3.90E-02	3.07E-04	1.17E-06
154.66	1.824	0.490	1.841	15	4.79E-02	2.23E-04	1.05E-06
309.11	1.783	0.457	1.804	46	1.50E-02	1.39E-04	2.04E-07
618.35	1.732	0.415	1.758	28	2.34E-02	8.61E-05	1.97E-07
1235.86	1.677	0.370	1.705	23	2.68E-02	4.65E-05	1.22E-07
2473.04	1.619	0.323	1.648	113	5.10E-03	2.45E-05	1.22E-08
1235.86	1.627	0.329	1.623				
309.11	1.651	0.349	1.639				
77.43	1.680	0.373	1.666				
18.98	1.720	0.405	1.700				
4.84	1.749	0.429	1.735				

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

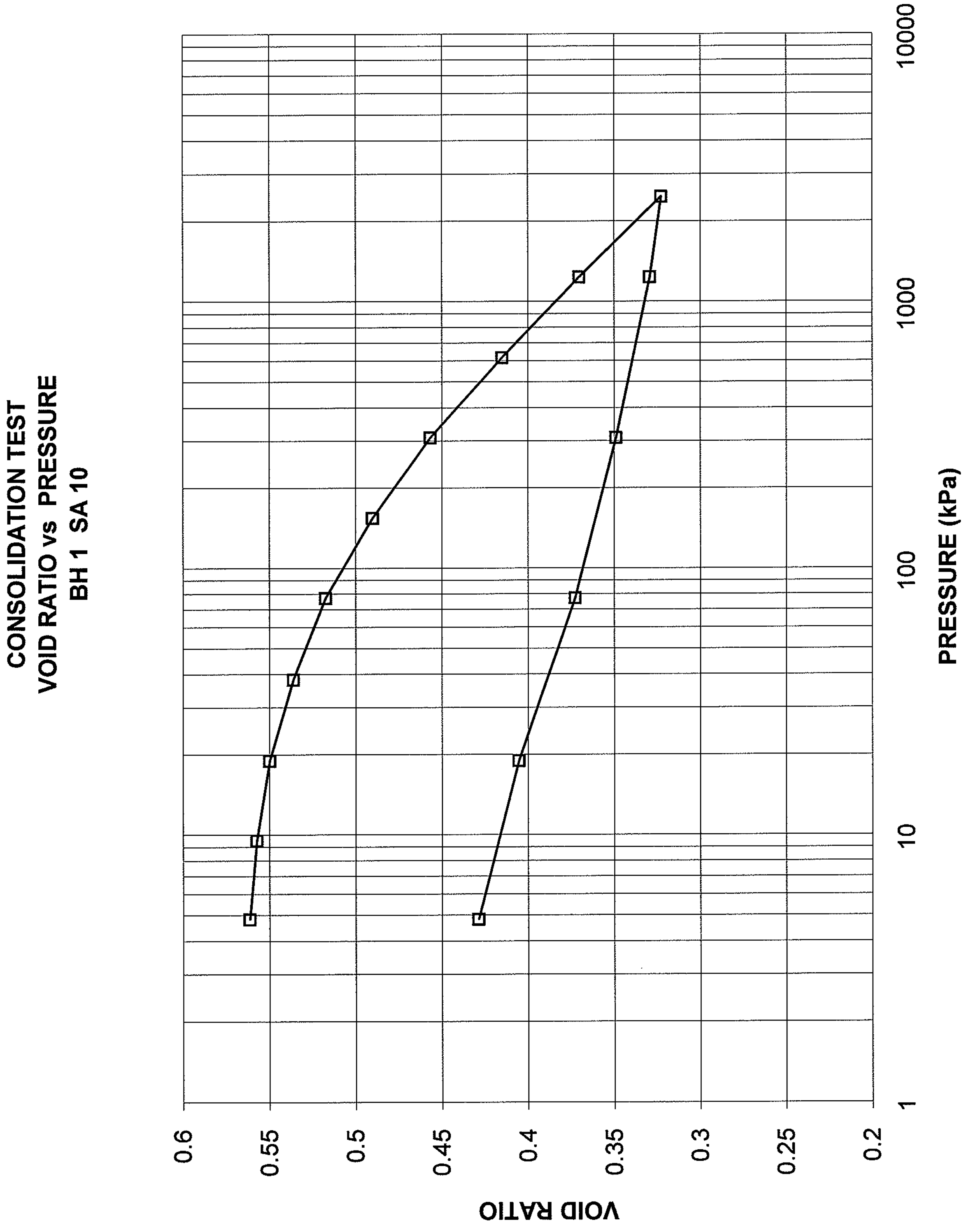
SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.75	Unit Weight, kN/m^3	21.80
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	18.74
Area, cm^2	31.67	Specific Gravity, measured	2.73
Volume, cm^3	55.39	Solids Height, cm	1.224
Water Content, %	16.37	Volume of Solids, cm ^{3}	38.76
Wet Mass, g	123.14	Volume of Voids, cm ^{3}	16.63
Dry Mass, g	105.82		

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

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE A4a



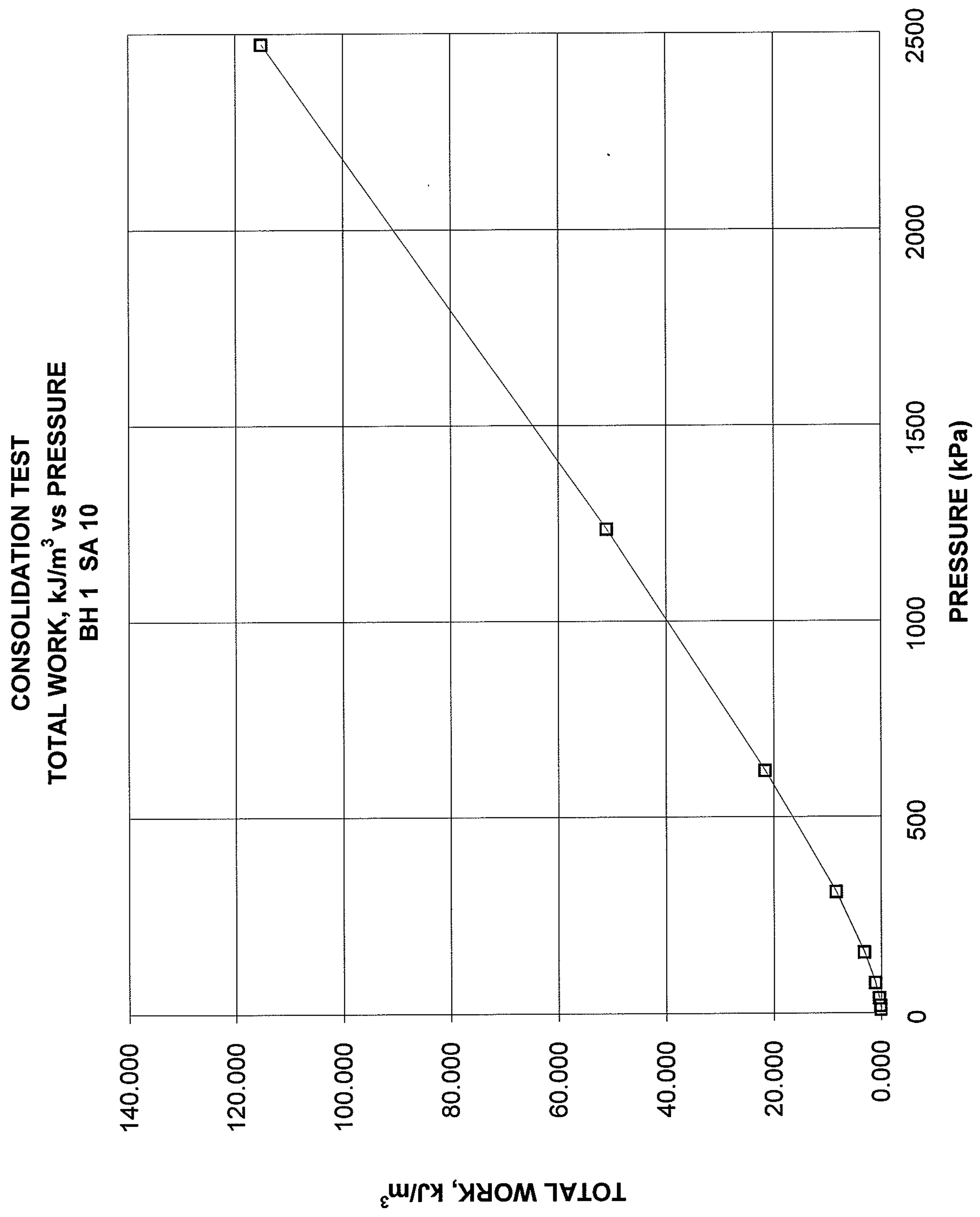
Project No. 04-1111-060

Prepared By: LFG

Golder Associates

Checked By: MM

FIGURE A4b



Project No. 04-1111-060

Golder Associates

Prepared By: LFG

Checked By: MM

OEDOMETER CONSOLIDATION SUMMARY

SAMPLE IDENTIFICATION

Project Number	04-1111-060	Sample Number	14
Borehole Number	1	Sample Depth, m	19.13-19.74

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	6		
Date Started	10/17/2006		
Date Completed	11/11/2006		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m^3	20.02
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	16.30
Area, cm^2	31.67	Specific Gravity, measured	2.76
Volume, cm^3	60.17	Solids Height, cm	1.144
Water Content, %	22.81	Volume of Solids, cm ^{3}	36.23
Wet Mass, g	122.81	Volume of Voids, cm ^{3}	23.94
Dry Mass, g	100.00	Degree of Saturation, %	95.3

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	0.661	1.900				
4.75	1.887	0.649	1.894	12	6.33E-02	1.44E-03	8.94E-06
9.54	1.878	0.642	1.883	56	1.34E-02	9.89E-04	1.30E-06
19.40	1.860	0.626	1.869	28	2.64E-02	9.61E-04	2.49E-06
38.88	1.834	0.603	1.847	40	1.81E-02	7.02E-04	1.24E-06
77.38	1.803	0.576	1.819	23	3.05E-02	4.24E-04	1.27E-06
154.70	1.765	0.543	1.784	17	3.97E-02	2.59E-04	1.01E-06
308.46	1.724	0.507	1.745	46	1.40E-02	1.40E-04	1.93E-07
617.46	1.680	0.468	1.702	60	1.02E-02	7.49E-05	7.52E-08
1233.50	1.631	0.426	1.656	56	1.04E-02	4.19E-05	4.26E-08
2470.61	1.576	0.378	1.604	76	7.17E-03	2.34E-05	1.64E-08
1233.50	1.589	0.389	1.583				
308.46	1.612	0.409	1.601				
77.38	1.638	0.432	1.625				
19.40	1.673	0.462	1.656				
4.75	1.711	0.496	1.692				

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

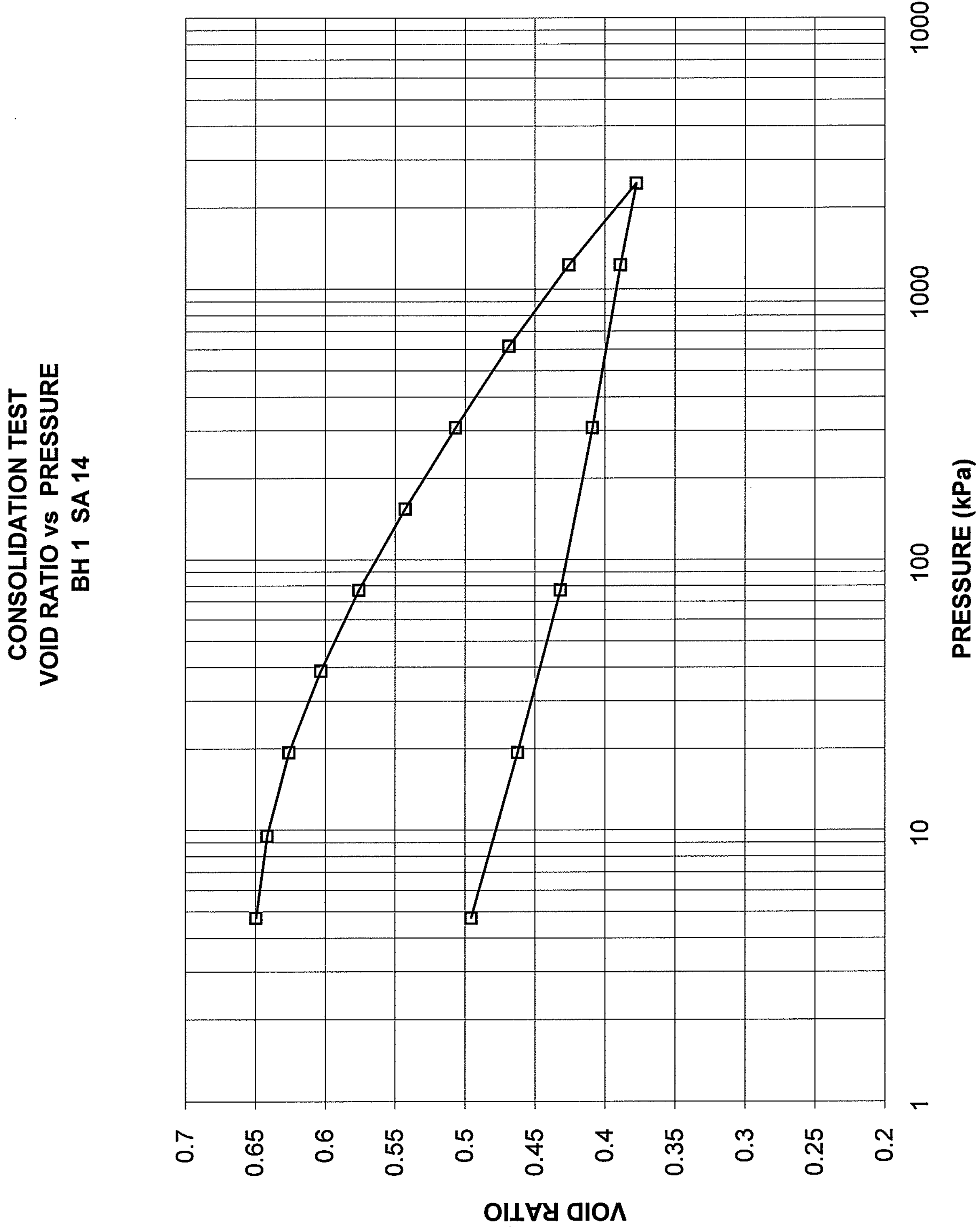
SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.71	Unit Weight, kN/m^3	21.44
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	18.10
Area, cm^2	31.67	Specific Gravity, measured	2.76
Volume, cm^3	54.19	Solids Height, cm	1.144
Water Content, %	18.48	Volume of Solids, cm ^{3}	36.23
Wet Mass, g	118.48	Volume of Voids, cm ^{3}	17.95
Dry Mass, g	100.00		

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

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE A5a



Project No. 04-1111-060

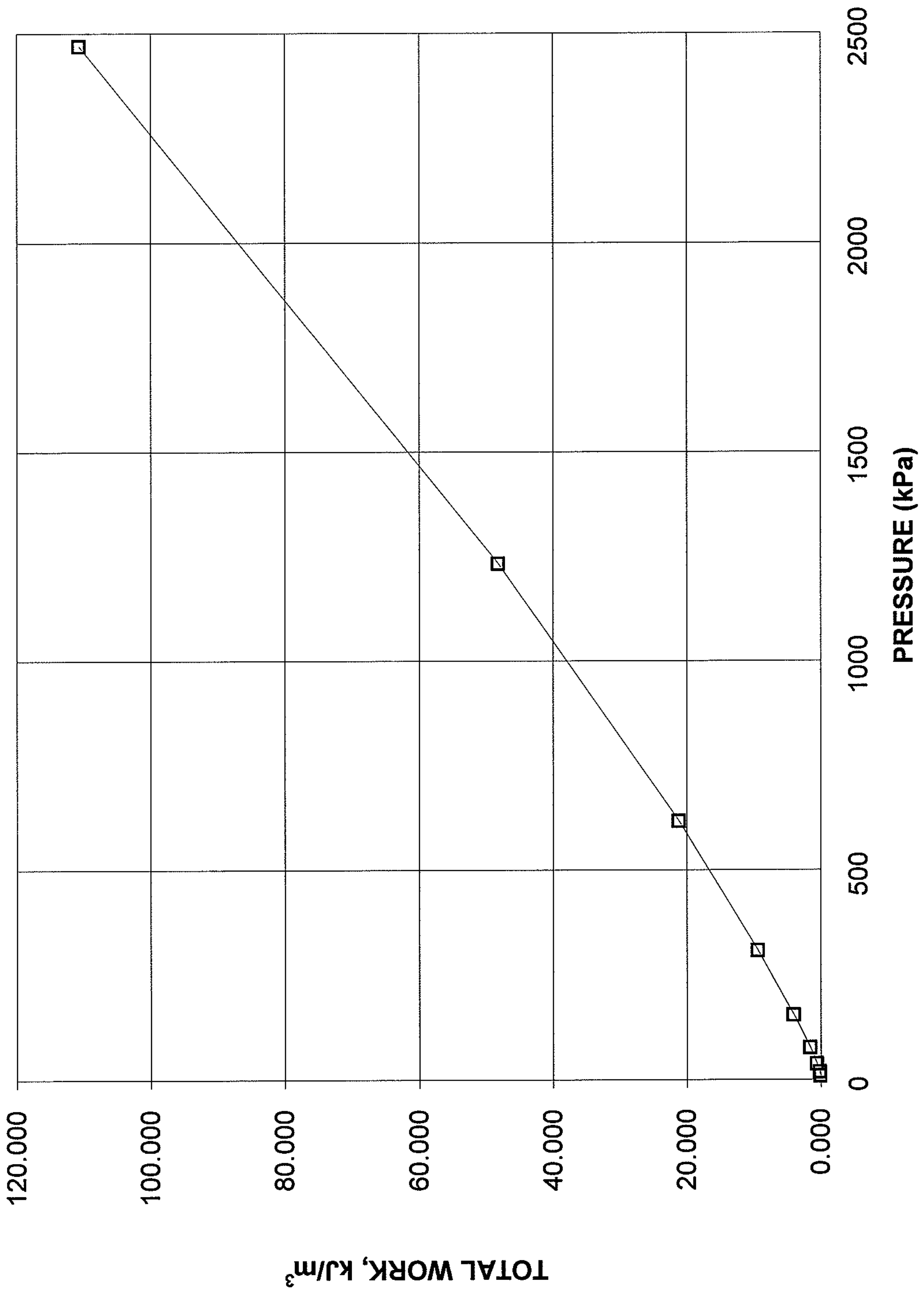
Prepared By: LFG

Golder Associates

Checked By: MM

FIGURE A5b

CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs PRESSURE
BH 1 SA 14



Project No. 04-1111-060

Golder Associates

Prepared By: LFG

Checked By: MM

CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS SHEET 1 OF 4	FIGURE A6a

TEST STAGE	A	B	C
BOREHOLE NUMBER	1	1	1
SAMPLE	5	10	14
SPECIMEN DIAMETER, cm	5.00	5.00	5.01
SPECIMEN HEIGHT, cm	10.13	10.09	10.00
WATER CONTENT BEFORE CONSOLIDATION, %	17.8	20.9	23.2
CELL PRESSURE, σ_3, kPa	310.0	275.0	300.0
BACK PRESSURE, kPa	275.0	205.0	205.0
PORE PRESSURE PARAMETER "B"	0.96	0.95	0.99
CONSOLIDATION PRESSURE, σc, kPa	35.0	70.0	95.0
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	1.2	3.5	7.1
WATER CONTENT AFTER CONSOLIDATION, %	17.2	19.0	26.6
AVERAGE RATE OF STRAIN, %/hr	0.5	0.5	0.5
TIME TO FAILURE, DAYS	1	1	1
WATER CONTENT AFTER TEST, %	16.8	18.4	24.6
MAX. DEVIATOR STRESS, (σ_1-σ_3), kPa	101.5	113.1	122.2
AXIAL STRAIN AT (σ_1-σ_3) MAXIMUM, %	19.9	20.2	16.9
MAX EFFECTIVE PRINCIPAL STRESS RATIO, (σ_1/σ_3) MAXIMUM	3.5	3.4	3.1
DEVIATOR STRESS AT (σ_1/σ_3) MAXIMUM, kPa	57.5	110.7	111.1
AXIAL STRAIN AT (σ_1/σ_3) MAXIMUM, %	4.9	9.3	10.1
PORE PRESSURE PARAMETER, Af, AT (σ_1-σ_3) MAXIMUM	-0.13	0.14	0.27
PORE PRESSURE PARAMETER, Af, AT (σ_1/σ_3) MAXIMUM	0.20	0.21	0.38
NATURAL WATER CONTENT, %	16.4	19.5	28.3
DRY DENSITY, Mg/m^3	1.89	1.82	1.55
FILTER DRAINS USED, y/n	y	y	y
TEST NOTES:			
CHANGED RATE OF STRAIN, %/hr	-	-	-
AXIAL STRAIN WHERE RATE OF STRAIN WAS CHANGED, %	-	-	-
FAILURE PLANE NUMBER	-	-	-
ANGLE OF FAILURE, DEGREES	bulging	bulging	bulging

Date: 10/31/2006

Project No. 04-1111-060

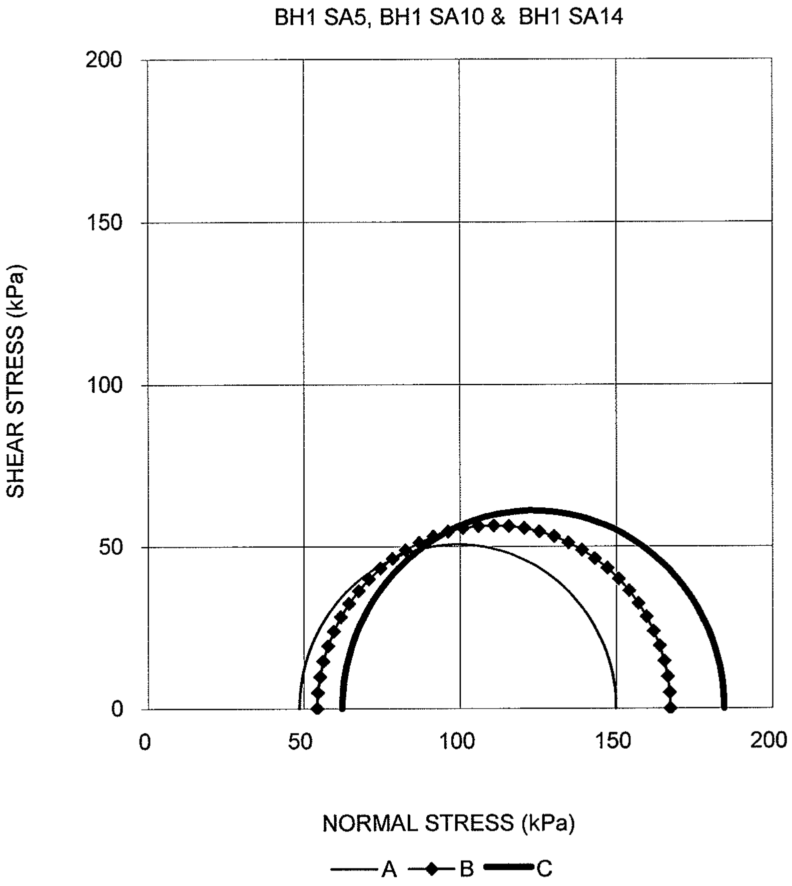
Golder Associates

Prepared By LFG

Checked By: MM

CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 2 OF 4

FIGURE A6b



Date: 10/31/2006

Project No. 04-1111-060

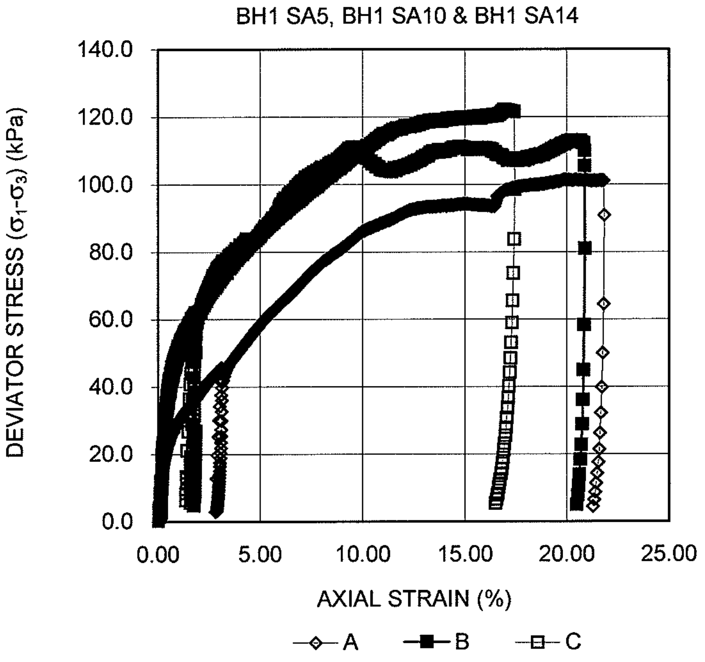
Golder Associates

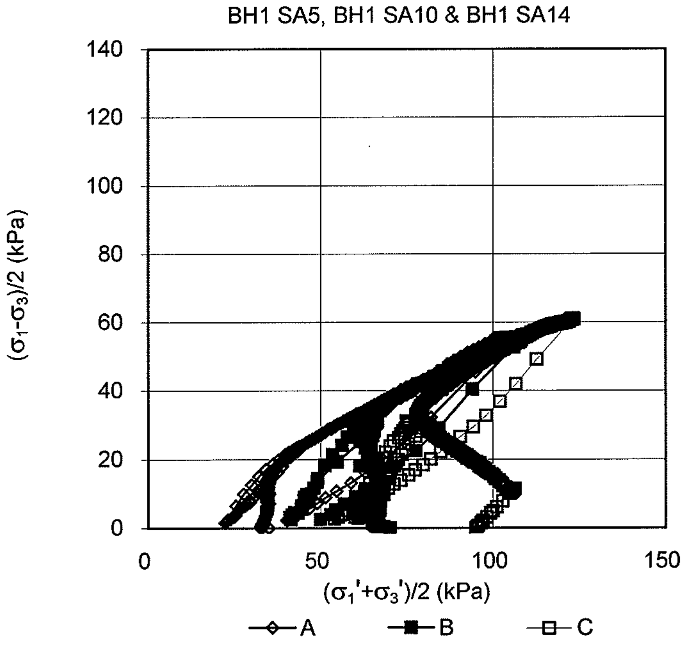
Prepared By LFG

Checked By: MM

CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 3 OF 4

FIGURE A6c





Date: 10/31/2006

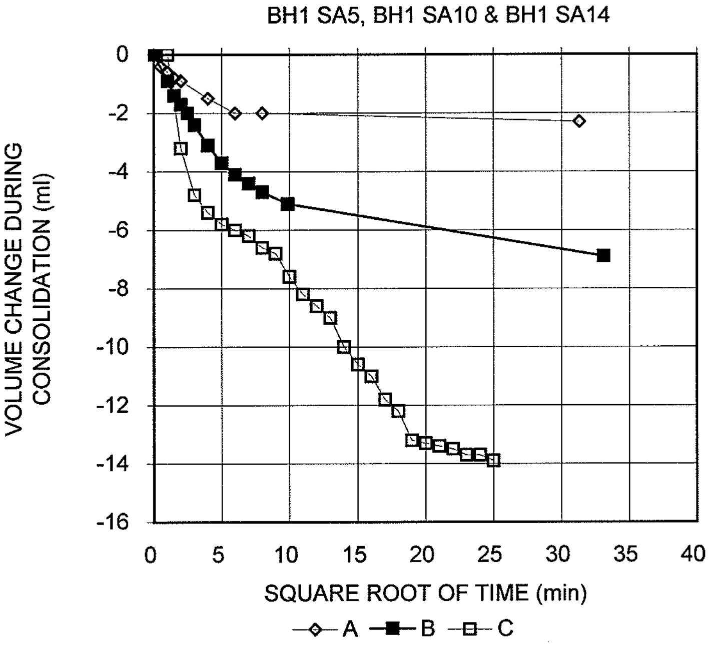
Project No. 04-1111-060

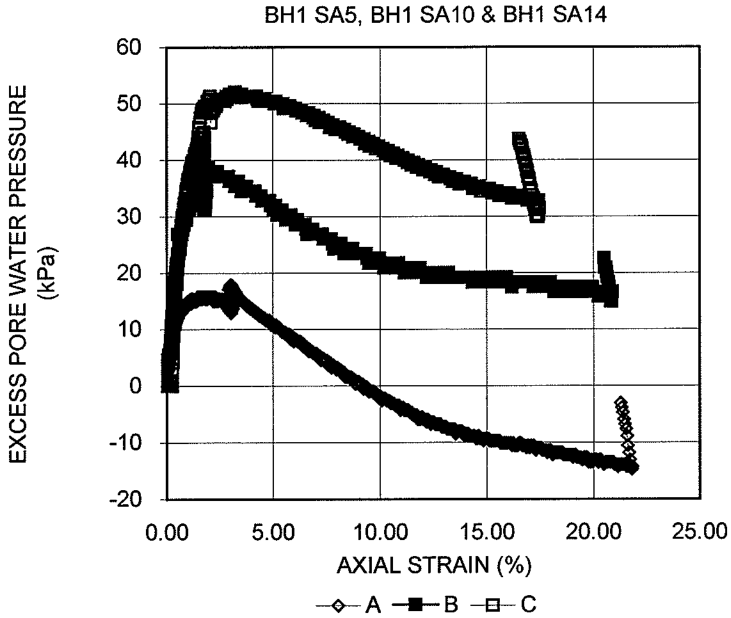
Golder Associates

Prepared By LFG

Checked By: MM

CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS SHEET 4 OF 4	FIGURE A6d





Date:	10/31/2006		Prepared By	LFG
Project No.	04-1111-060	**Golder Associates**	Checked By:	MM

CARBONATE TEST DETERMINATION

Borehole Number		1	1		
Sample Number		14	14		
Depth, m		19.13-19.74	19.13-19.74		

TEST DATA ENTRY

Sample Weight, g	(A)	1.724	1.709		
First Reading, ml	(B)	36.0	31.0		
Second Reading, ml	(C)	108.0	103.0		
Room Temperature, c	(D)	23.5	22.7		
Flask Temperature, c	(E)	23.5	23.5		
Barometer, kPa	(F)	101.8	101.8		
Flask Temp. / Barometer Correction	(G)	1.0372	1.0372		

TEST CALCULATIONS

CORRECTED READINGS				
First Reading, B*G	37.34	32.15		
Second Reading, C*G	112.02	106.83		
Dolomite, C*G-B*G	74.68	74.68		
Calcite, (B*G)-0.04((C*G)-(B*G))	34.35	29.17		

CARBONATE PERCENTAGES FROM TABLES

Dolomite, %	(H)	17.10	17.10		
Calcite, %	(I)	7.90	6.70		
Total, %	(H+I)	25.00	23.80		
Ratio,	(I/H)	0.46	0.39		

CARBONATE TEST DETERMINATION FORM

Project Number	04-1111-060	Tested By	AH
Date of Testing	07-02-09	Entered By	RO
Remarks		Checked By	RO

CARBONATE TEST DETERMINATION

Borehole Number		1	1	1	1
Sample Number		5	5	10	10
Depth, m		6.94-7.55	6.94-7.55	13.64-14.25	13.64-14.25

TEST DATA ENTRY

Sample Weight, g	(A)	1.702	1.734	1.700	1.704
First Reading, ml	(B)	45.0	48.0	43.0	48.0
Second Reading, ml	(C)	110.0	110.0	105.0	106.0
Room Temperature, c	(D)	23.0	23.5	23.2	22.8
Flask Temperature, c	(E)	23.8	23.5	23.5	23.0
Barometer, kPa	(F)	101.9	101.9	101.8	101.4
Flask Temp. / Barometer Correction	(G)	1.0372	1.0400	1.0372	1.0371

TEST CALCULATIONS

CORRECTED READINGS				
First Reading, B*G	46.68	49.92	44.60	49.78
Second Reading, C*G	114.10	114.40	108.91	109.94
Dolomite, C*G-B*G	67.42	64.48	64.31	60.15
Calcite, (B*G)-0.04((C*G)-(B*G))	43.98	47.34	42.03	47.38

CARBONATE PERCENTAGES FROM TABLES

Dolomite, %	(H)	15.40	14.80	14.70	13.90
Calcite, %	(I)	10.20	11.00	9.80	11.00
Total, %	(H+I)	25.60	25.80	24.50	24.90
Ratio,	(I/H)	0.66	0.74	0.67	0.79

CARBONATE TEST DETERMINATION FORM

Project Number	04-1111-060	Tested By	AH
Date of Testing	07-02-08	Entered By	RO
Remarks		Checked By	RO

UNCONFINED COMPRESSION TEST (UC)

SAMPLE IDENTIFICATION

PROJECT NUMBER	04-1111-060	SAMPLE NUMBER	1
BOREHOLE NUMBER	1	SAMPLE DEPTH, m	34.3-34.6

TEST CONDITIONS

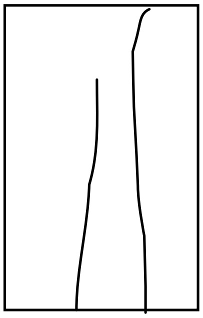
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.30

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.81	WATER CONTENT, (specimen) %	0.26
SAMPLE DIAMETER, cm	4.70	UNIT WEIGHT, kN/m^3	24.06
SAMPLE AREA, cm^2	17.35	DRY UNIT WT., kN/m^3	23.99
SAMPLE VOLUME, cm^3	187.55	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	460.22	VOID RATIO	0.10
DRY WEIGHT, g	459.03		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

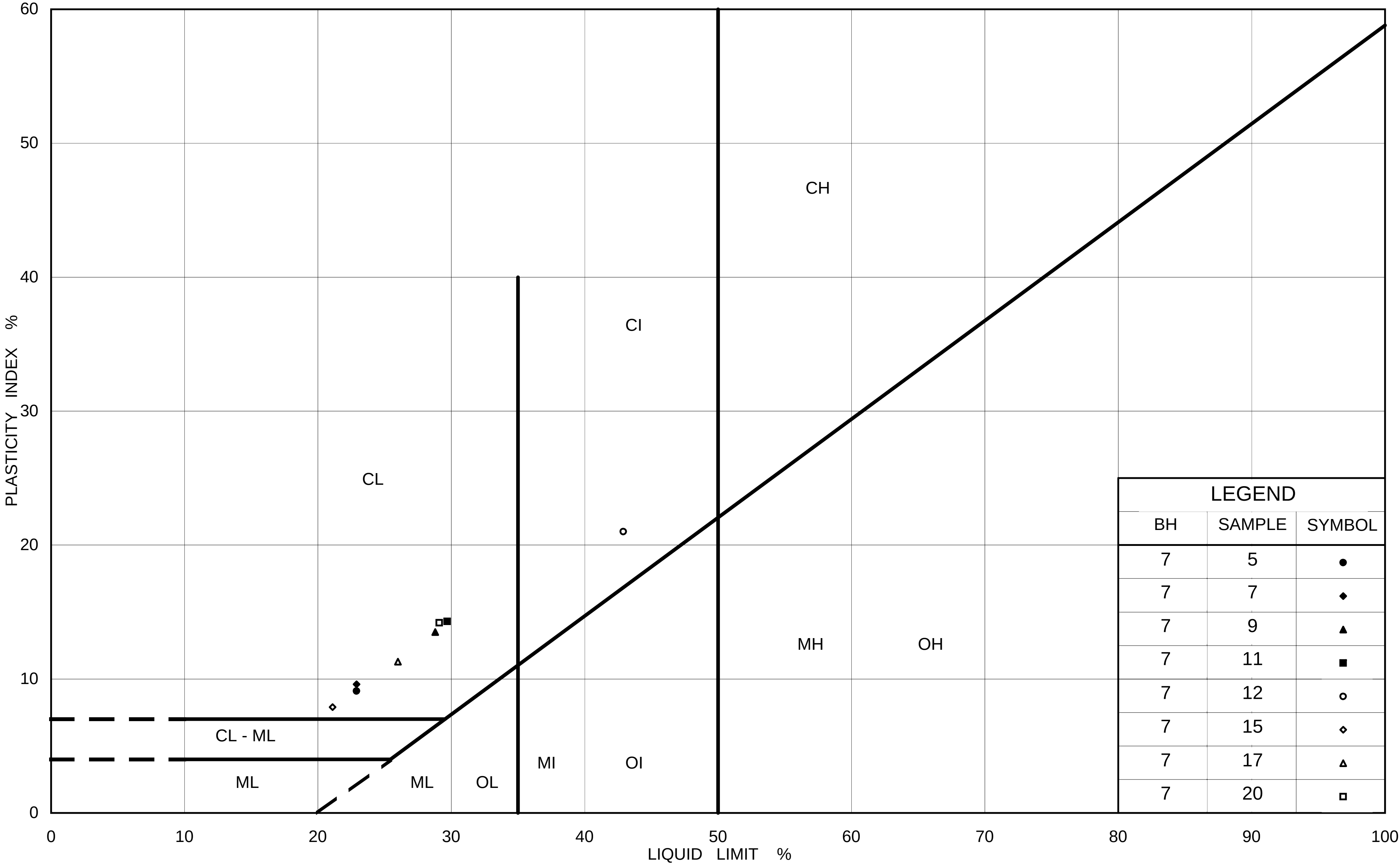
STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	49.2

REMARKS:

DATE: 09/02/2007

Golder Associates

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
7	5	●
7	7	◆
7	9	▲
7	11	■
7	12	○
7	15	◇
7	17	△
7	20	□

Ministry of Transportation

Ontario

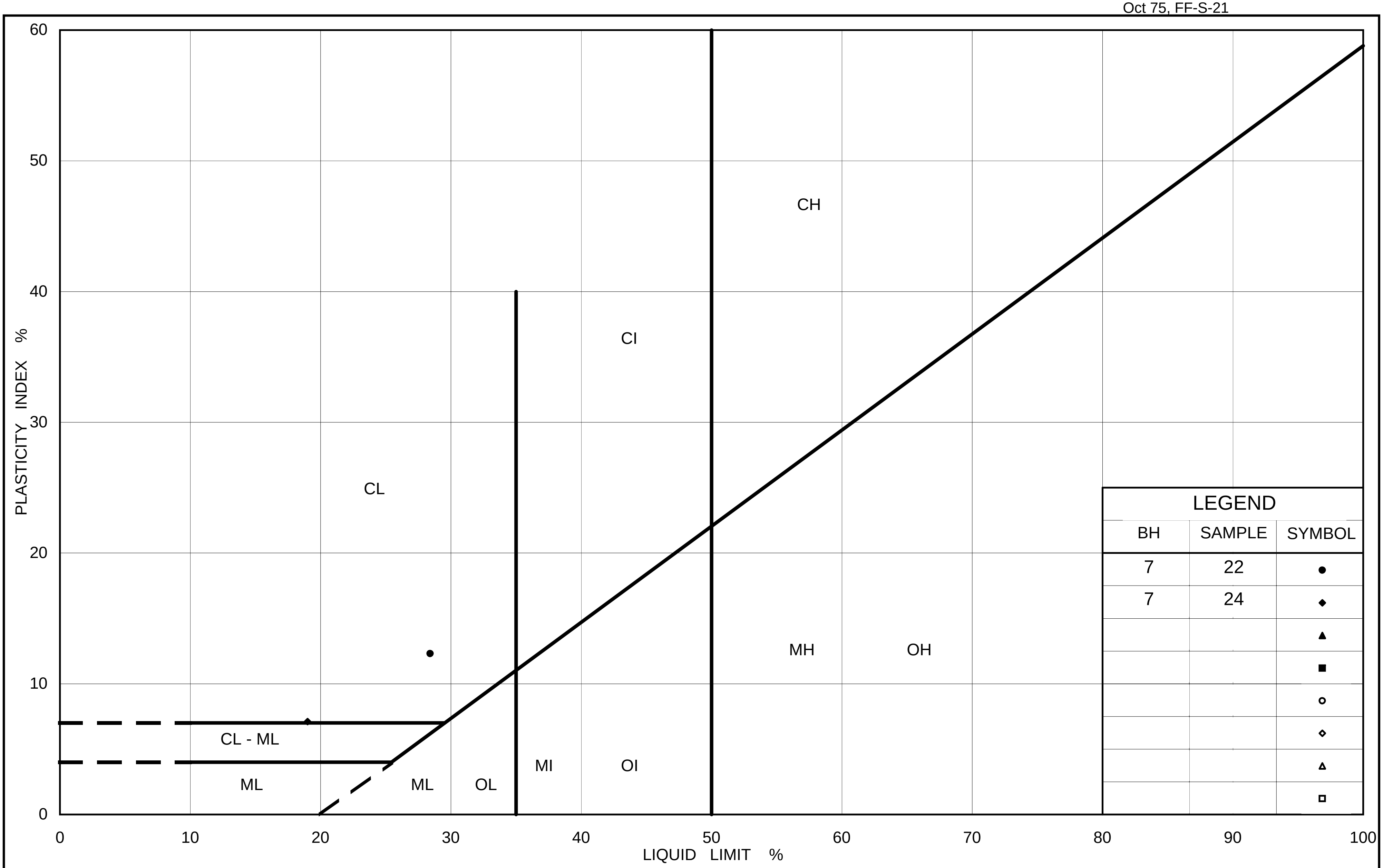
PLASTICITY CHART

FIG No. A7a

Project No. 04-1111-060

Checked By: SJB

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART

FIG No. A8b

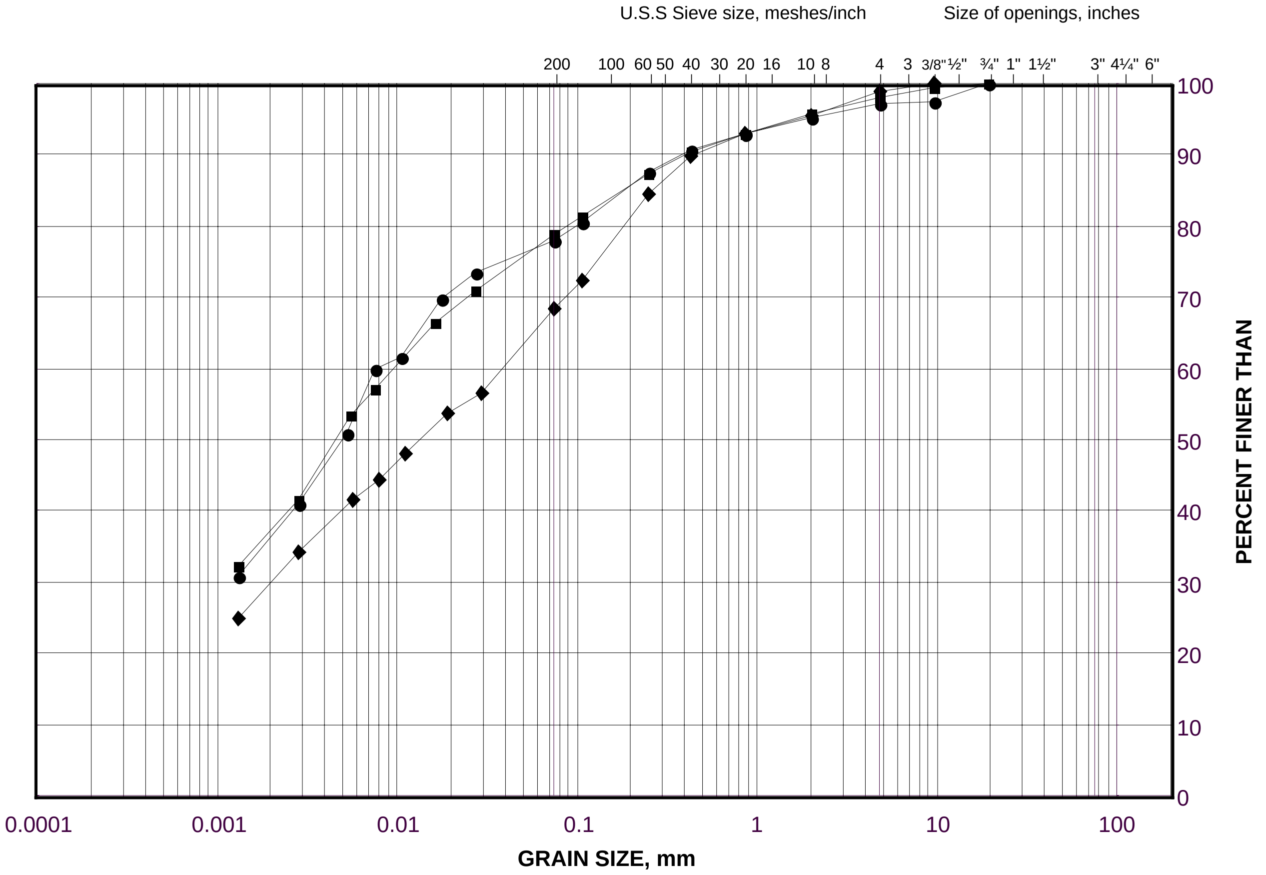
Project No. 04-1111-060

Checked By: SJB

GRAIN SIZE DISTRIBUTION

Clayey Silt to Silty Clay Deposit

FIGURE A9



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	7	11	14.60 - 15.00
■	7	20	25.90 - 26.50
◆	7	7	8.50 - 8.90

Project Number: 04-1111-060

Checked By: SJB

Golder Associates

Date: 06-Jun-07

OEDOMETER CONSOLIDATION SUMMARY

SAMPLE IDENTIFICATION

Project Number	04-1111-060	Sample Number	7
Borehole Number	7	Sample Depth, m	8.5-8.9

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	5		
Date Started	11/13/2006		
Date Completed	11/25/2006		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.91	Unit Weight, kN/m^3	21.44
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	18.35
Area, cm^2	31.65	Specific Gravity, measured	2.73
Volume, cm^3	60.45	Solids Height, cm	1.309
Water Content, %	16.88	Volume of Solids, cm ^{3}	41.43
Wet Mass, g	132.19	Volume of Voids, cm ^{3}	19.02
Dry Mass, g	113.10	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	1.910	0.459	1.910				
4.70	1.875	0.432	1.893	8	9.49E-02	3.90E-03	3.63E-05
9.54	1.865	0.425	1.870	7	1.06E-01	1.08E-03	1.12E-05
19.26	1.853	0.416	1.859	43	1.70E-02	6.46E-04	1.08E-06
38.70	1.837	0.403	1.845	46	1.57E-02	4.31E-04	6.63E-07
77.44	1.819	0.390	1.828	53	1.34E-02	2.43E-04	3.19E-07
154.87	1.794	0.371	1.807	76	9.10E-03	1.69E-04	1.51E-07
309.20	1.757	0.342	1.776	94	7.11E-03	1.26E-04	8.75E-08
618.55	1.711	0.307	1.734	124	5.14E-03	7.79E-05	3.92E-08
1241.52	1.660	0.268	1.686	68	8.86E-03	4.29E-05	3.72E-08
2478.24	1.608	0.228	1.634	146	3.88E-03	2.20E-05	8.36E-09
1241.52	1.614	0.233	1.611				
309.20	1.633	0.248	1.624				
77.44	1.659	0.267	1.646				
19.29	1.691	0.292	1.675				
4.85	1.717	0.312	1.704				

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

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.72	Unit Weight, kN/m^3	23.31
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	20.41
Area, cm^2	31.65	Specific Gravity, measured	2.73
Volume, cm^3	54.34	Solids Height, cm	1.309
Water Content, %	14.20	Volume of Solids, cm ^{3}	41.43
Wet Mass, g	129.16	Volume of Voids, cm ^{3}	12.91
Dry Mass, g	113.1		

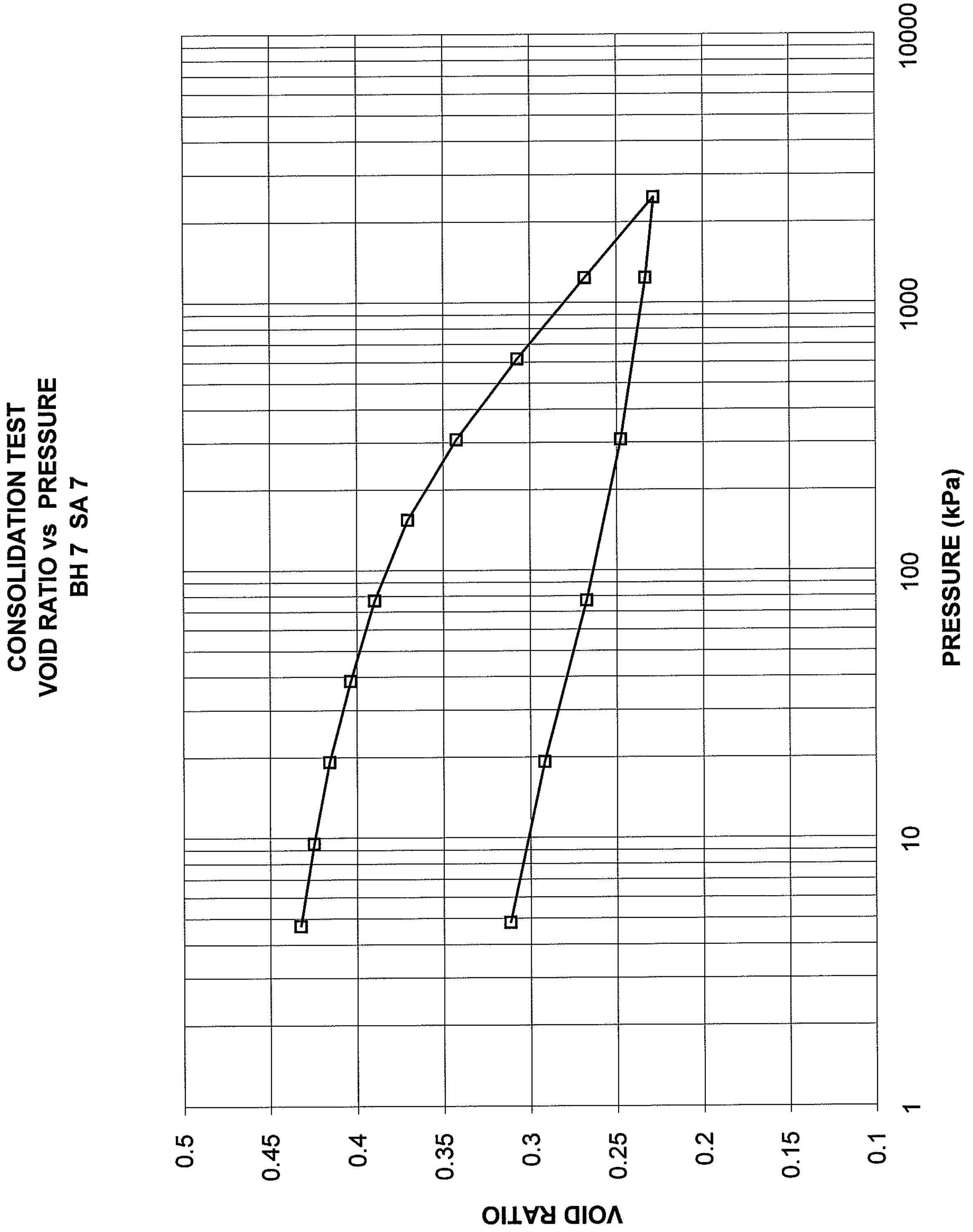
Prepared By: LFG

Golder Associates

Checked By: MM

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE A9a



Project No. 04-1111-060

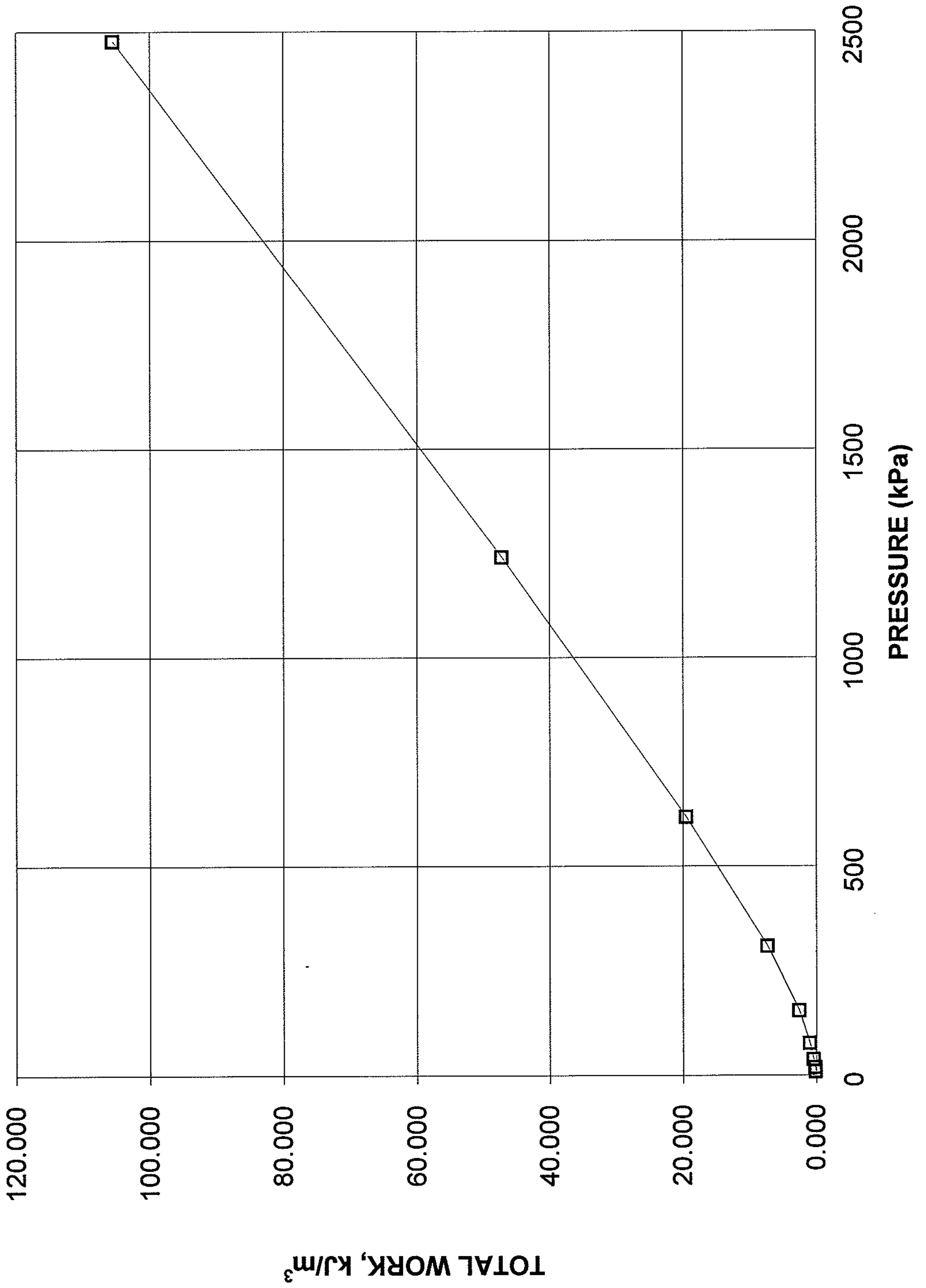
Prepared By: LFG

Golder Associates

Checked By: MM

FIGURE A9b

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



Project No. 04-1111-060

Golder Associates

Prepared By: LFG

Checked By: MM

OEDOMETER CONSOLIDATION SUMMARY

SAMPLE IDENTIFICATION

Project Number	04-1111-060	Sample Number	11
Borehole Number	7	Sample Depth, m	14.6-15.0

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	7		
Date Started	11/13/2006		
Date Completed	11/24/2006		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m^3	20.68
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	16.99
Area, cm^2	31.65	Specific Gravity, measured	2.76
Volume, cm^3	60.13	Solids Height, cm	1.193
Water Content, %	21.71	Volume of Solids, cm ^{3}	37.75
Wet Mass, g	126.80	Volume of Voids, cm ^{3}	22.39
Dry Mass, g	104.18	Degree of Saturation, %	101.0

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	0.593	1.900				
4.83	1.897	0.591	1.899	8	9.55E-02	3.27E-04	3.06E-06
9.55	1.890	0.585	1.894	13	5.85E-02	7.81E-04	4.47E-06
19.51	1.876	0.573	1.883	23	3.27E-02	7.40E-04	2.37E-06
38.91	1.855	0.555	1.866	23	3.21E-02	5.70E-04	1.79E-06
77.57	1.826	0.531	1.841	28	2.56E-02	3.95E-04	9.92E-07
154.67	1.791	0.502	1.809	124	5.59E-03	2.39E-04	1.31E-07
309.92	1.732	0.452	1.762	271	2.43E-03	2.00E-04	4.76E-08
619.27	1.670	0.400	1.701	40	1.53E-02	1.05E-04	1.59E-07
1237.90	1.593	0.336	1.632	15	3.76E-02	6.55E-05	2.42E-07
2475.99	1.508	0.264	1.551	34	1.50E-02	3.61E-05	5.31E-08
1237.90	1.522	0.276	1.515				
309.92	1.549	0.299	1.536				
77.57	1.588	0.331	1.569				
19.51	1.634	0.370	1.611				
4.83	1.663	0.394	1.649				

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

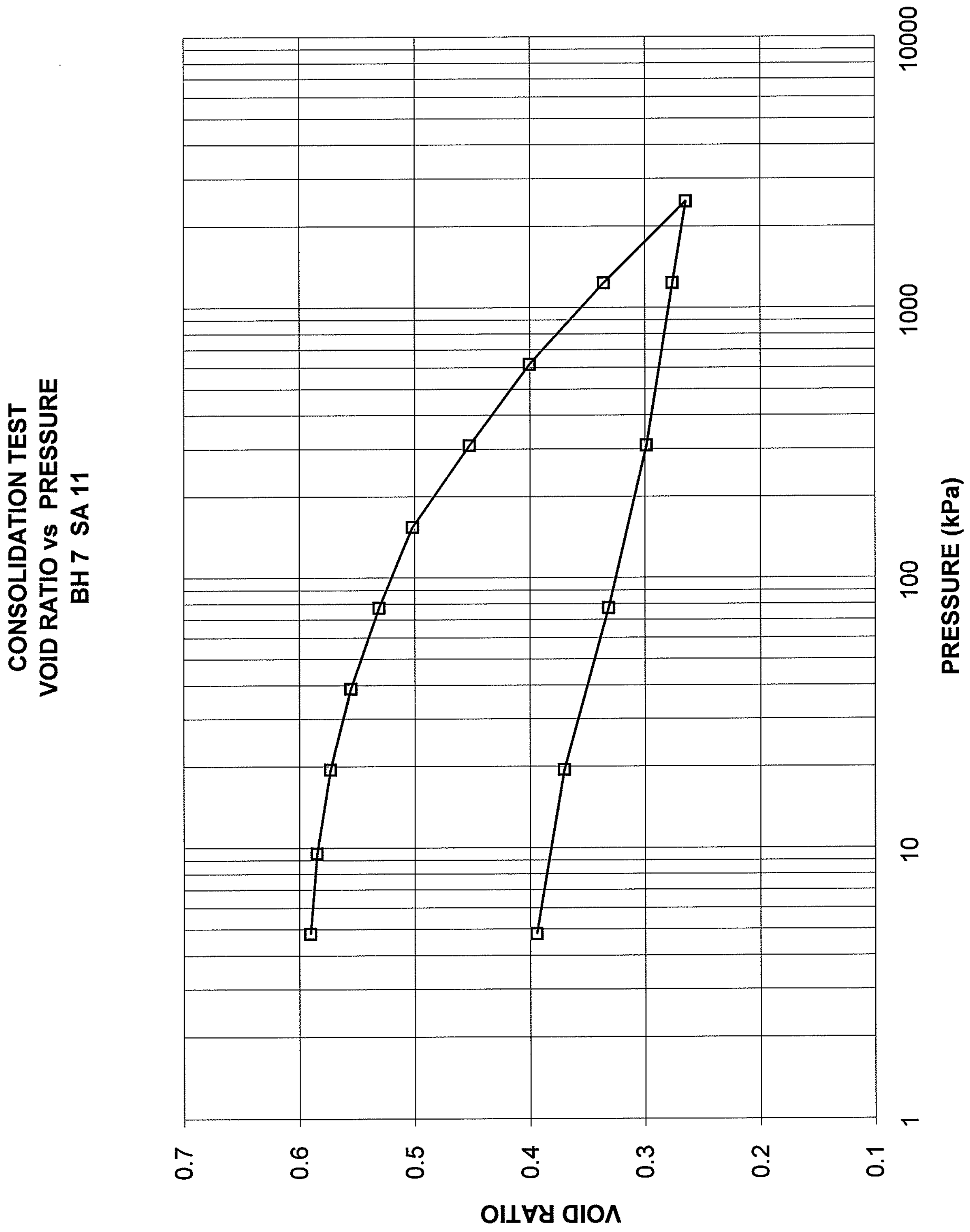
SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.66	Unit Weight, kN/m^3	22.90
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	19.41
Area, cm^2	31.65	Specific Gravity, measured	2.76
Volume, cm^3	52.63	Solids Height, cm	1.193
Water Content, %	18.00	Volume of Solids, cm ^{3}	37.75
Wet Mass, g	122.93	Volume of Voids, cm ^{3}	14.89
Dry Mass, g	104.18		

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

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE A10a



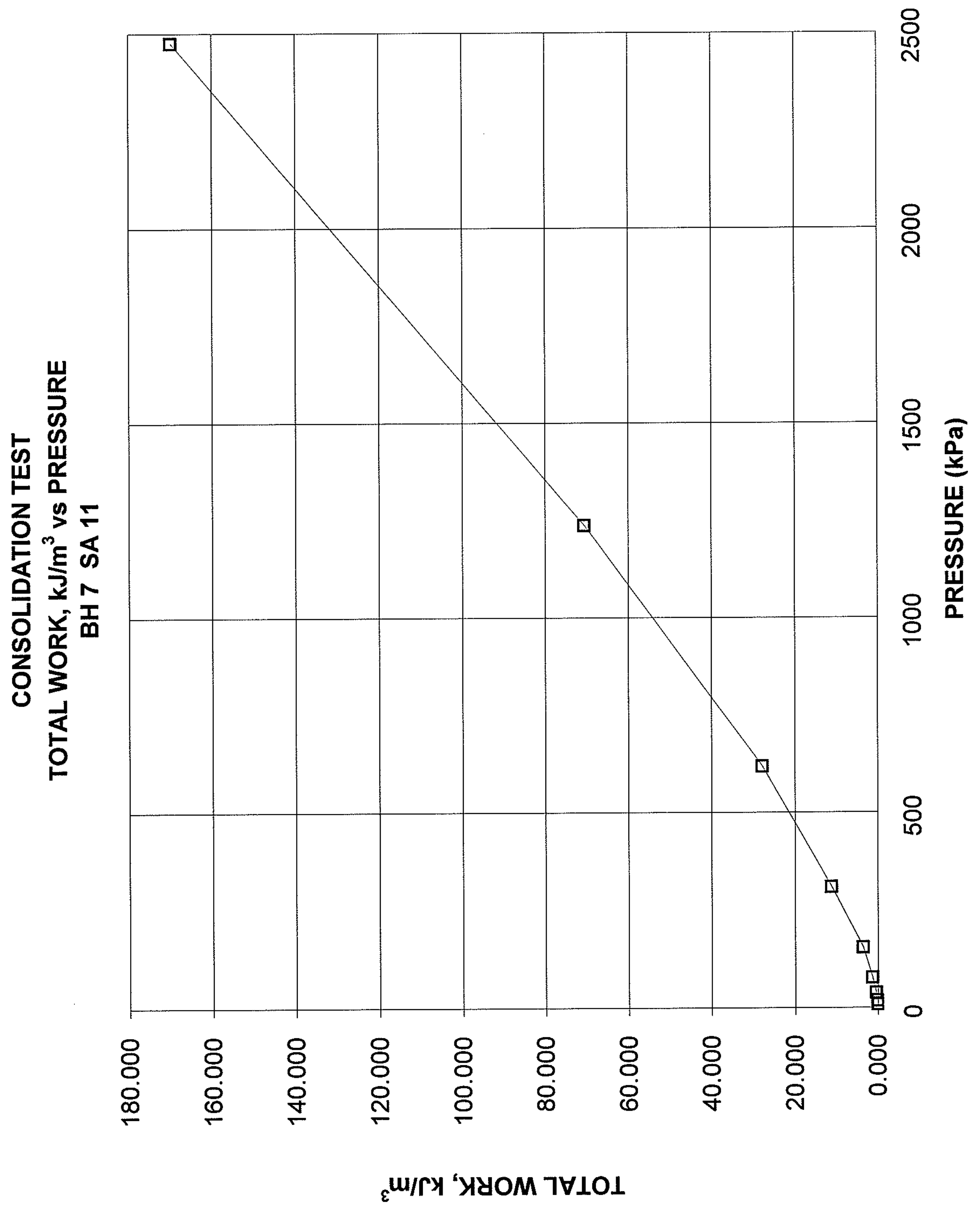
Project No. 04-1111-060

Prepared By: LFG

Golder Associates

Checked By: MM

FIGURE A10b



CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs PRESSURE
BH 7 SA 11

Project No. 04-1111-060

Golder Associates

Prepared By: LFG

Checked By: MM

OEDOMETER CONSOLIDATION SUMMARY

SAMPLE IDENTIFICATION

Project Number	04-1111-060	Sample Number	20
Borehole Number	7	Sample Depth, m	25.9-26.5

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	8		
Date Started	11/14/2006		
Date Completed	11/29/2006		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.92	Unit Weight, kN/m^3	20.98
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	17.54
Area, cm^2	31.67	Specific Gravity, measured	2.74
Volume, cm^3	60.65	Solids Height, cm	1.250
Water Content, %	19.57	Volume of Solids, cm ^{3}	39.60
Wet Mass, g	129.73	Volume of Voids, cm ^{3}	21.05
Dry Mass, g	108.50	Degree of Saturation, %	100.9

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	1.915	0.532	1.915				
4.85	1.912	0.529	1.914	7	1.11E-01	3.23E-04	3.51E-06
9.50	1.907	0.525	1.910	19	4.07E-02	5.61E-04	2.24E-06
19.40	1.894	0.515	1.901	15	5.10E-02	6.86E-04	3.43E-06
38.64	1.876	0.500	1.885	20	3.77E-02	4.89E-04	1.80E-06
77.43	1.849	0.479	1.863	11	6.69E-02	3.63E-04	2.38E-06
154.57	1.801	0.440	1.825	12	5.88E-02	3.25E-04	1.87E-06
309.12	1.746	0.396	1.774	17	3.92E-02	1.86E-04	7.14E-07
618.28	1.678	0.342	1.712	68	9.14E-03	1.15E-04	1.03E-07
1236.63	1.609	0.287	1.644	158	3.62E-03	5.83E-05	2.07E-08
2474.00	1.533	0.226	1.571	84	6.23E-03	3.21E-05	1.96E-08
1236.63	1.543	0.234	1.538				
309.12	1.570	0.256	1.557				
77.43	1.611	0.288	1.591				
19.40	1.656	0.324	1.634				
4.85	1.692	0.353	1.674				

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

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.69	Unit Weight, kN/m^3	23.20
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	19.86
Area, cm^2	31.67	Specific Gravity, measured	2.74
Volume, cm^3	53.58	Solids Height, cm	1.250
Water Content, %	16.81	Volume of Solids, cm ^{3}	39.60
Wet Mass, g	126.74	Volume of Voids, cm ^{3}	13.99
Dry Mass, g	108.5		

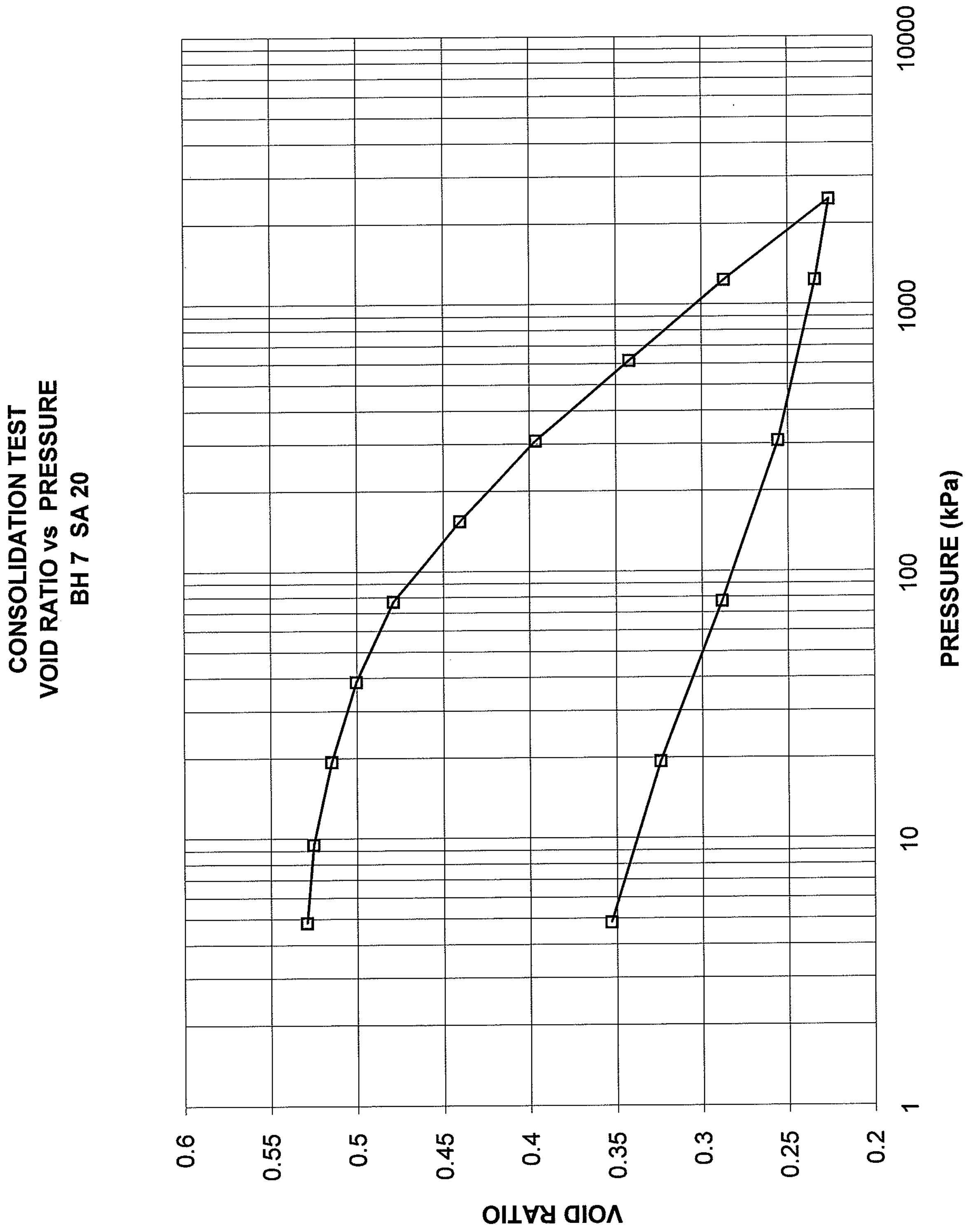
Prepared By: LFG

Golder Associates

Checked By: MM

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE A11a



Project No. 04-1111-060

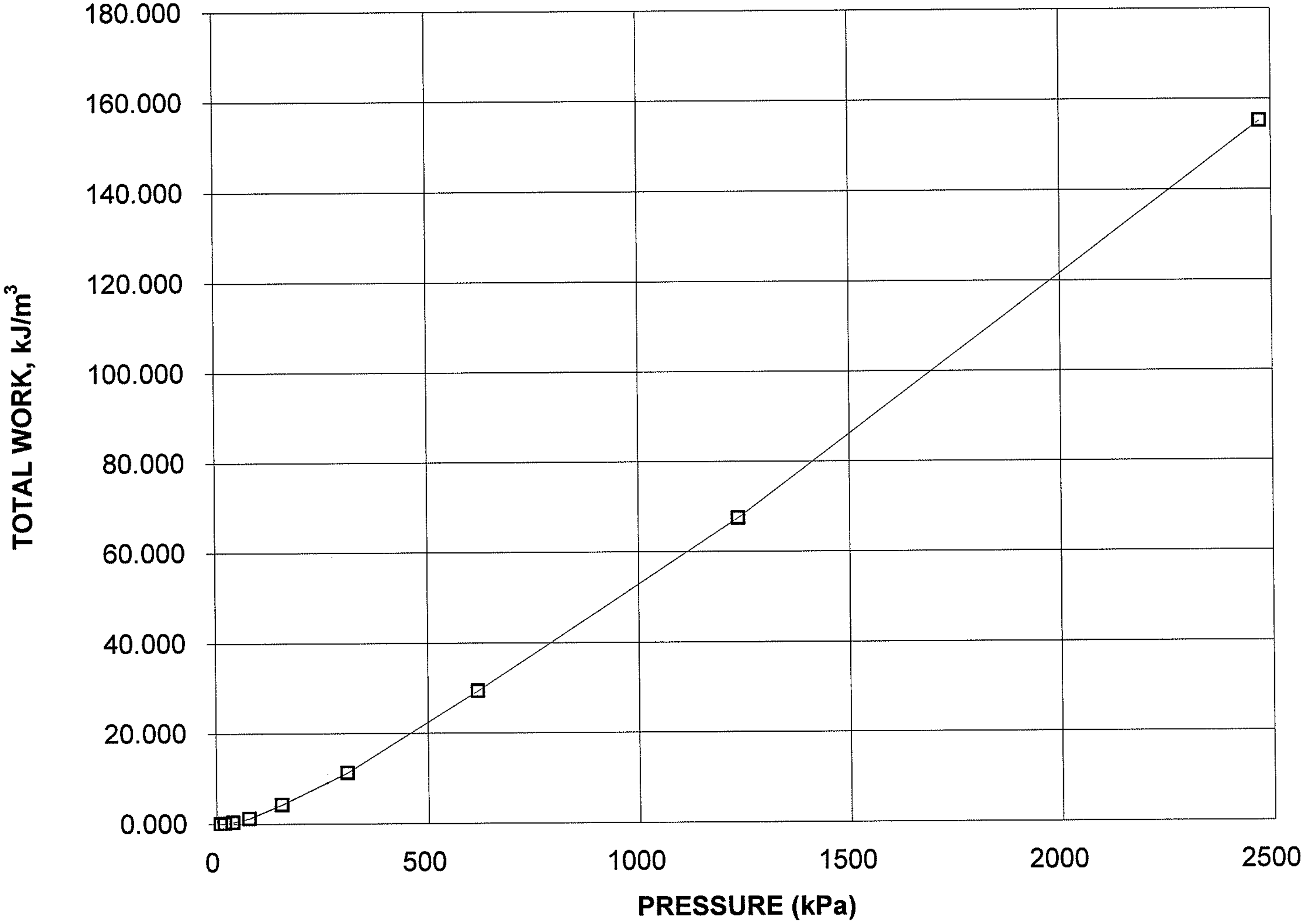
Prepared By: LFG

Golder Associates

Checked By: MM

FIGURE A11b

CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs PRESSURE
BH 7 SA 20



Project No. 04-1111-060

Prepared By: LFG

Golder Associates

Checked By: MM

CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 1 OF 4

FIGURE A12a

TEST STAGE	A	B	C
BOREHOLE NUMBER	7	7	7
SAMPLE	7	11	20
SPECIMEN DIAMETER, cm	4.98	4.96	4.96
SPECIMEN HEIGHT, cm	10.13	10.10	10.11
WATER CONTENT BEFORE CONSOLIDATION, %	20.4	23.0	22.0
CELL PRESSURE, σ_3, kPa	248.0	208.0	265.0
BACK PRESSURE, kPa	205.0	135.0	135.0
PORE PRESSURE PARAMETER "B"	0.96	0.96	0.96
CONSOLIDATION PRESSURE, σc, kPa	43.0	73.0	130.0
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	2.1	1.3	3.9
WATER CONTENT AFTER CONSOLIDATION, %	19.2	22.2	19.7
AVERAGE RATE OF STRAIN, %/hr	0.5	0.5	0.5
TIME TO FAILURE, DAYS	1	1	1
WATER CONTENT AFTER TEST, %	18.4	21.8	19.6
MAX. DEVIATOR STRESS, $(\sigma_1-\sigma_3)$, kPa	101.9	116.2	159.4
AXIAL STRAIN AT $(\sigma_1-\sigma_3)$ MAXIMUM, %	9.6	5.1	12.0
MAX EFFECTIVE PRINCIPAL STRESS RATIO, (σ_1/σ_3) MAXIMUM	4.1	3.4	3.1
DEVIATOR STRESS AT (σ_1/σ_3) MAXIMUM, kPa	71.3	105.3	158.9
AXIAL STRAIN AT (σ_1/σ_3) MAXIMUM, %	2.3	2.2	10.1
PORE PRESSURE PARAMETER, Af, AT $(\sigma_1-\sigma_3)$ MAXIMUM	-0.02	0.19	0.33
PORE PRESSURE PARAMETER, Af, AT (σ_1/σ_3) MAXIMUM	0.28	0.28	0.33
NATURAL WATER CONTENT, %	18.2	21.9	19.0
DRY DENSITY, Mg/m^3	1.83	1.71	1.75
FILTER DRAINS USED, y/n	y	y	y
TEST NOTES:			
CHANGED RATE OF STRAIN, %/hr	-	-	-
AXIAL STRAIN WHERE RATE OF STRAIN WAS CHANGED, %	-	-	-
FAILURE PLANE NUMBER	-	-	-
ANGLE OF FAILURE, DEGREES	bulged	bulged	bulged

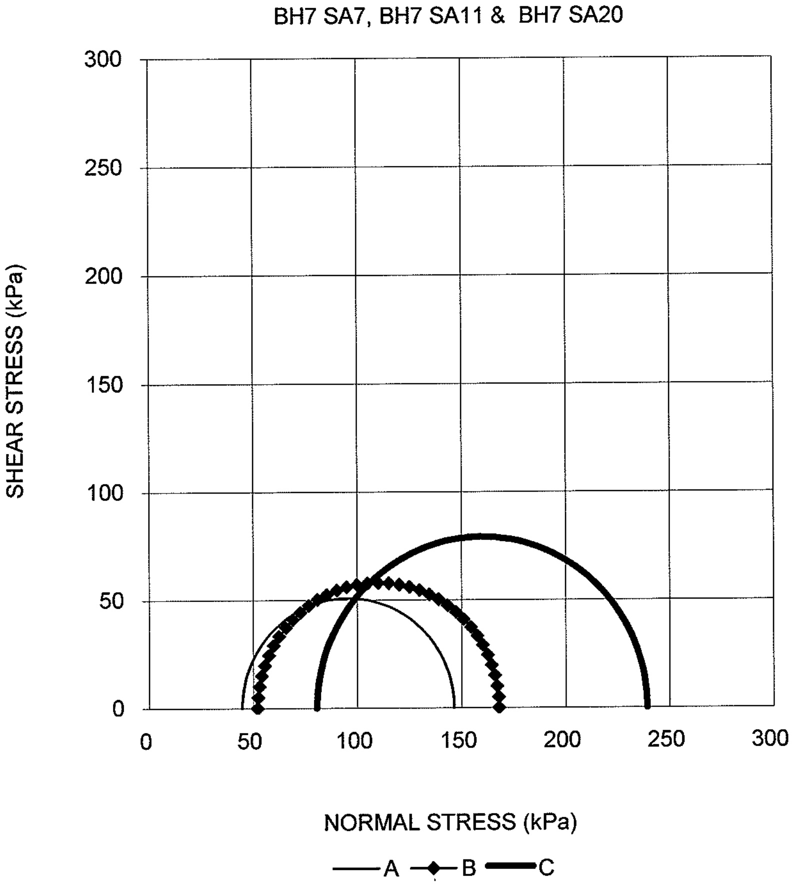
Date: 11/10/2006
Project No. 04-1111-060

Golder Associates

Prepared By LFG
Checked By: SJB

CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 2 OF 4

FIGURE A12b



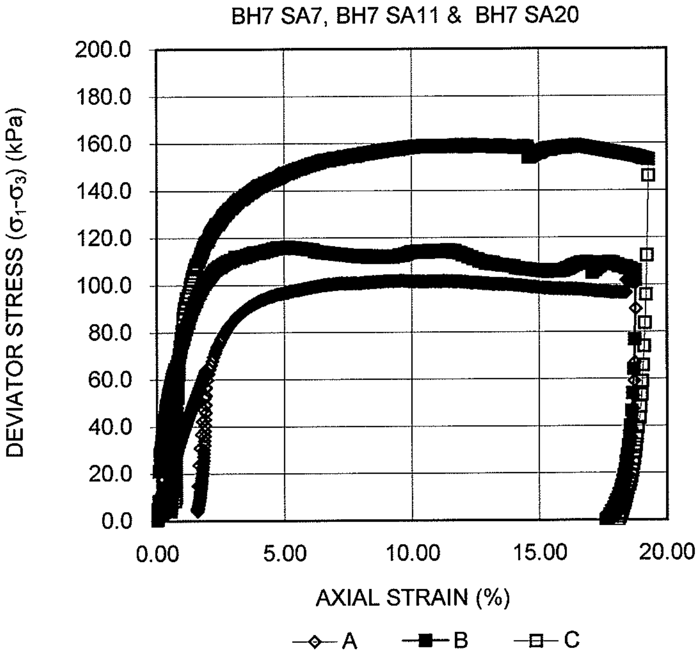
Date: 11/10/2006
Project No. 04-1111-060

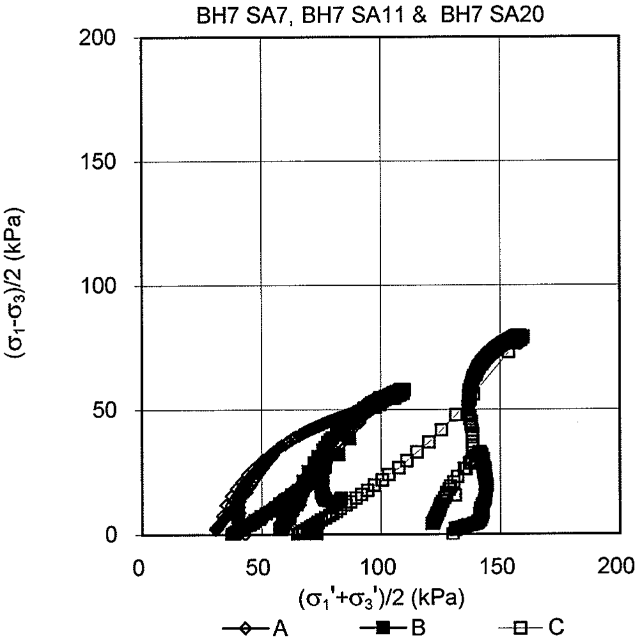
Golder Associates

Prepared By LFG
Checked By: MM

CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 3 OF 4

FIGURE A12c





Date: 11/10/2006

Project No. 04-1111-060

Golder Associates

Prepared By LFG

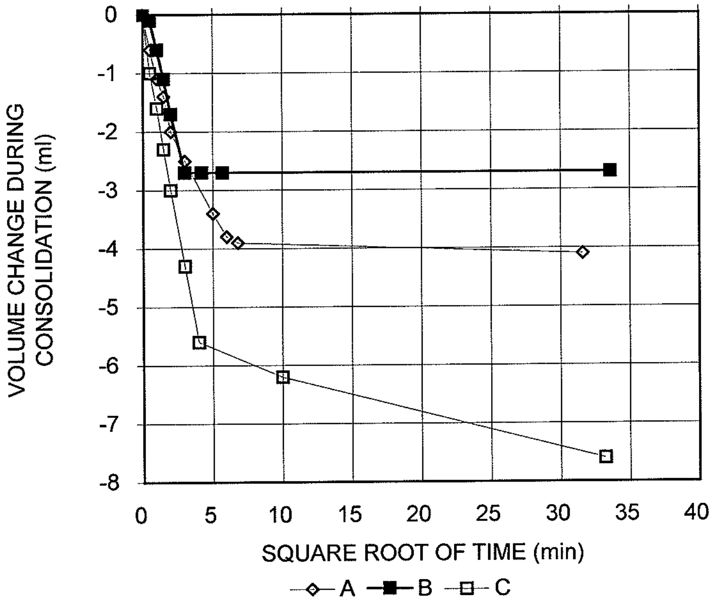
Checked By: SJB

CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS

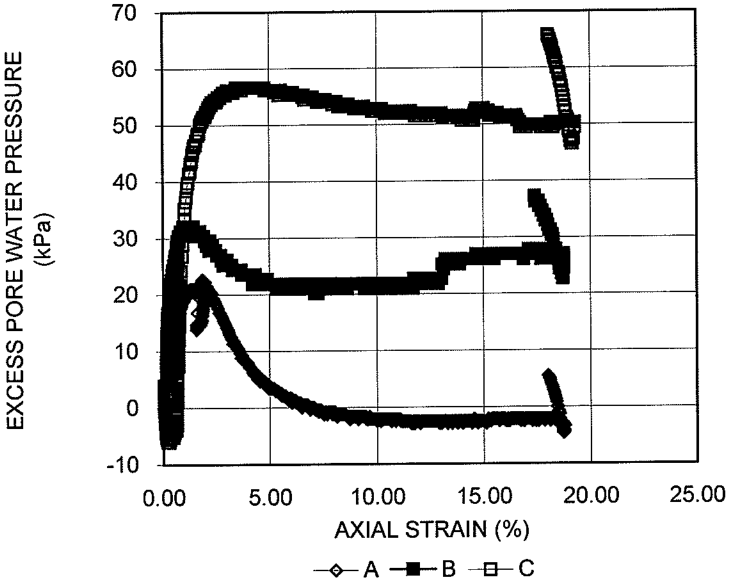
SHEET 4 OF 4

FIGURE A12d

BH7 SA7, BH7 SA11 & BH7 SA20



BH7 SA7, BH7 SA11 & BH7 SA20



Date: 11/10/2006

Project No. 04-1111-060

Golder Associates

Prepared By LFG

Checked By: SJB

CARBONATE TEST DETERMINATION

Borehole Number		7	7	7	
Sample Number		7	11	20	
Depth, m		8.5-8.9	14.6-15.0	25.9-26.5	
TEST DATA ENTRY					
Sample Weight, g	(A)	1.70	1.73	1.73	
First Reading, ml	(B)	48.00	39.00	50.20	
Second Reading, ml	(C)	109.00	102.00	117.00	
Room Temperature, °C	(D)	23.40	24.00	24.20	
Flask Temperature, °C	(E)	24.90	25.20	25.20	
Barometer, kPa	(F)	101.07	101.07	101.07	
Flask Temp. / Barometer Correction	(G)	1.02608	1.02608	1.02608	
TEST CALCULATIONS					
CORRECTED READINGS					
First Reading, BxG		49.25	40.02	51.51	
Second Reading, CxG		111.84	104.66	120.05	
Dolomite, CxG-BxG	(E)	62.59	64.64	68.54	
Calcite, (BxG)-0.04((CxG)-(BxG))	(F)	46.75	37.43	48.77	
CARBONATE PERCENTAGES FROM TABLES					
Dolomite, %	(H)	14.40	14.90	15.80	
Calcite, %	(I)	10.90	8.60	11.30	
Total, %	(H+I)	25.30	23.50	27.10	
Ratio	(I/H)	0.76	0.58	0.72	

Project Number	04-1111-060	Tested By	Angela
Date of Testing	1/18/2007	Entered By	LG
Remarks		Checked By	SJB

UNCONFINED COMPRESSION TEST (UC)

SAMPLE IDENTIFICATION

PROJECT NUMBER	04-1111-060	SAMPLE NUMBER	2
BOREHOLE NUMBER	7	SAMPLE DEPTH, m	37.2-37.4

TEST CONDITIONS

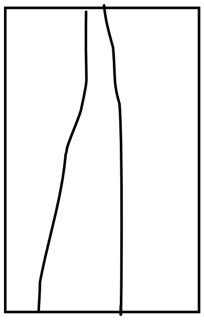
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.28

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.72	WATER CONTENT, (specimen) %	0.35
SAMPLE DIAMETER, cm	4.70	UNIT WEIGHT, kN/m^3	23.32
SAMPLE AREA, cm^2	17.35	DRY UNIT WT., kN/m^3	23.24
SAMPLE VOLUME, cm^3	185.99	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	442.47	VOID RATIO	0.14
DRY WEIGHT, g	440.93		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

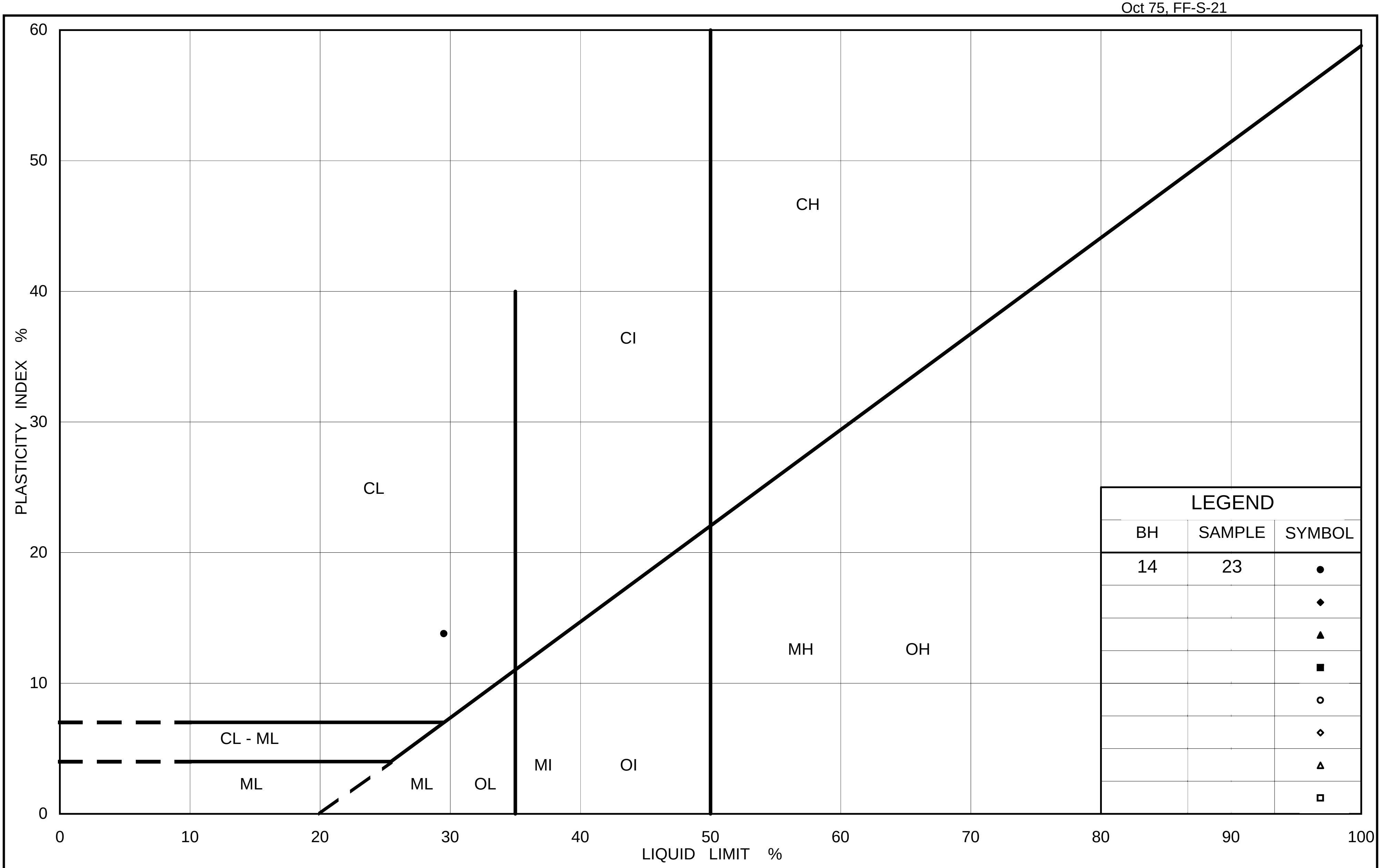
STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	33.3

REMARKS:

DATE: 09/02/2007

Golder Associates

Oct 75, FF-S-21



Ministry of Transportation

Ontario

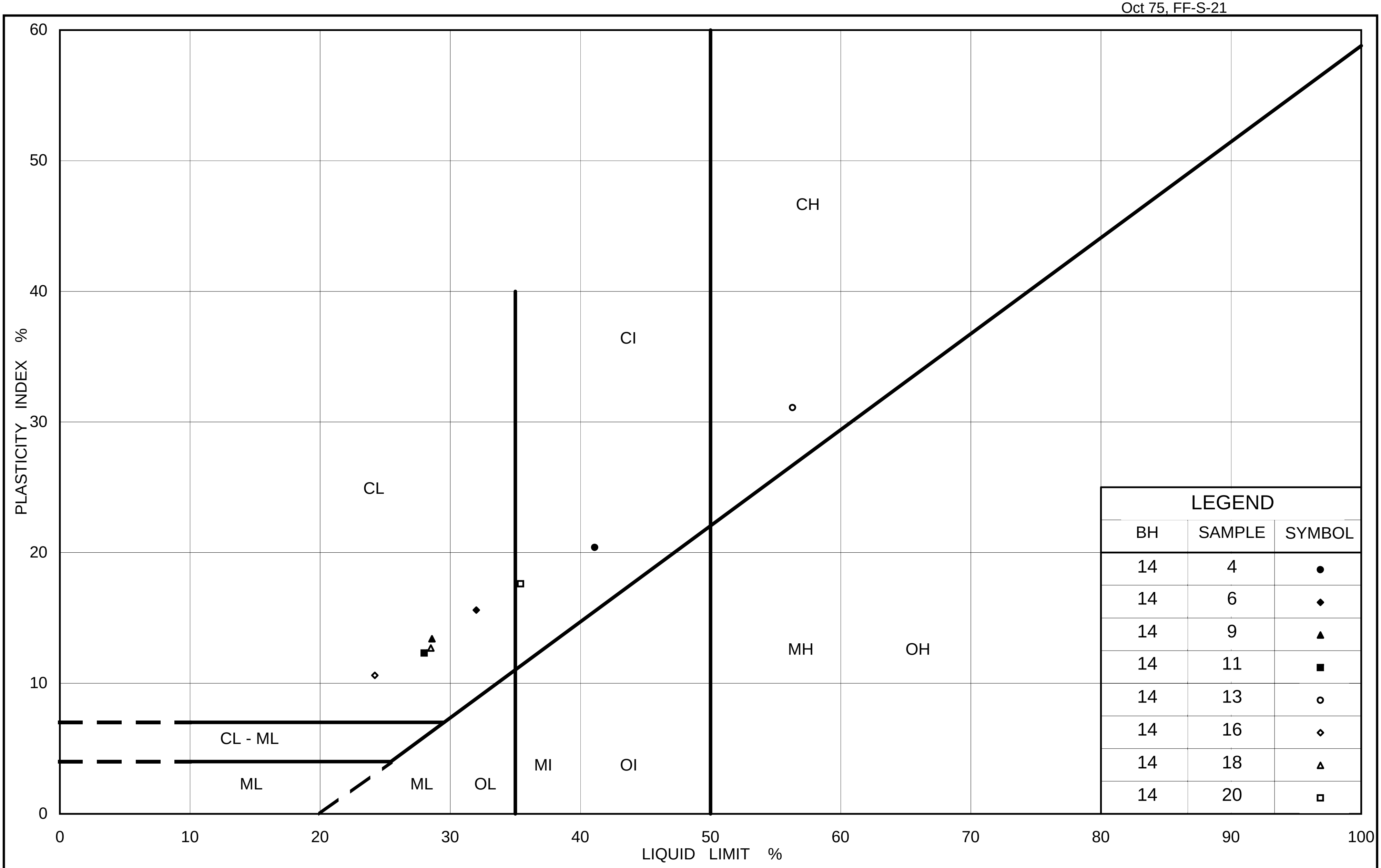
PLASTICITY CHART

FIG No. A13b

Project No. 04-1111-060

Checked By: SJB

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
14	4	●
14	6	◆
14	9	▲
14	11	■
14	13	○
14	16	◇
14	18	△
14	20	□

Ministry of Transportation

Ontario

PLASTICITY CHART

FIG No. A13a

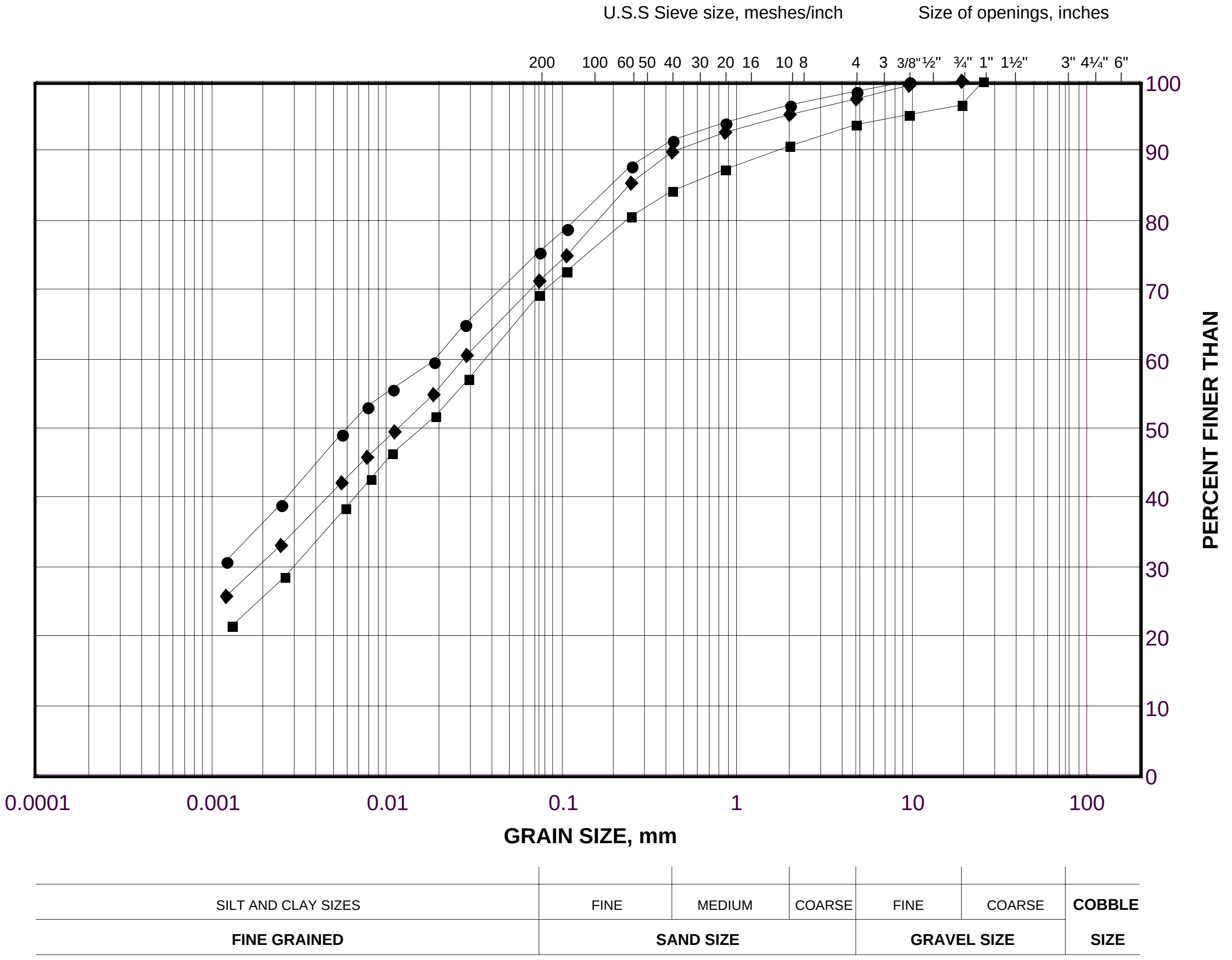
Project No. 04-1111-060

Checked By: SJB

GRAIN SIZE DISTRIBUTION

Clayey Silt to Silty Clay Deposit

FIGURE A14



LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	14	11	13.10 - 13.60
■	14	16	18.60 - 19.20
◆	14	9	10.10 - 10.50

Project Number: 04-1111-060

Checked By: SJB

Golder Associates

Date: 06-Jun-07

OEDOMETER CONSOLIDATION SUMMARY

SAMPLE IDENTIFICATION

Project Number	04-1111-060	Sample Number	9
Borehole Number	14	Sample Depth, m	10.1-10.5

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	6		
Date Started	11/16/2006		
Date Completed	11/29/2006		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m^3	20.99
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	17.47
Area, cm^2	31.67	Specific Gravity, measured	2.75
Volume, cm^3	60.17	Solids Height, cm	1.231
Water Content, %	20.15	Volume of Solids, cm ^{3}	38.98
Wet Mass, g	128.80	Volume of Voids, cm ^{3}	21.19
Dry Mass, g	107.20	Degree of Saturation, %	101.9

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	0.544	1.900				
4.75	1.894	0.539	1.897	10	7.63E-02	6.65E-04	4.97E-06
9.54	1.888	0.534	1.891	53	1.43E-02	6.59E-04	9.24E-07
19.25	1.876	0.524	1.882	76	9.88E-03	6.50E-04	6.30E-07
38.68	1.857	0.509	1.867	94	7.86E-03	5.15E-04	3.96E-07
77.38	1.835	0.491	1.846	184	3.93E-03	2.99E-04	1.15E-07
154.68	1.808	0.469	1.822	108	6.51E-03	1.84E-04	1.17E-07
309.02	1.768	0.436	1.788	124	5.47E-03	1.36E-04	7.31E-08
618.89	1.717	0.395	1.743	53	1.21E-02	8.66E-05	1.03E-07
1238.01	1.657	0.346	1.687	103	5.86E-03	5.10E-05	2.93E-08
2475.42	1.592	0.293	1.625	158	3.54E-03	2.76E-05	9.59E-09
1238.01	1.605	0.304	1.599				
309.02	1.629	0.323	1.617				
77.38	1.664	0.352	1.647				
19.25	1.710	0.389	1.687				
4.75	1.747	0.419	1.729				

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

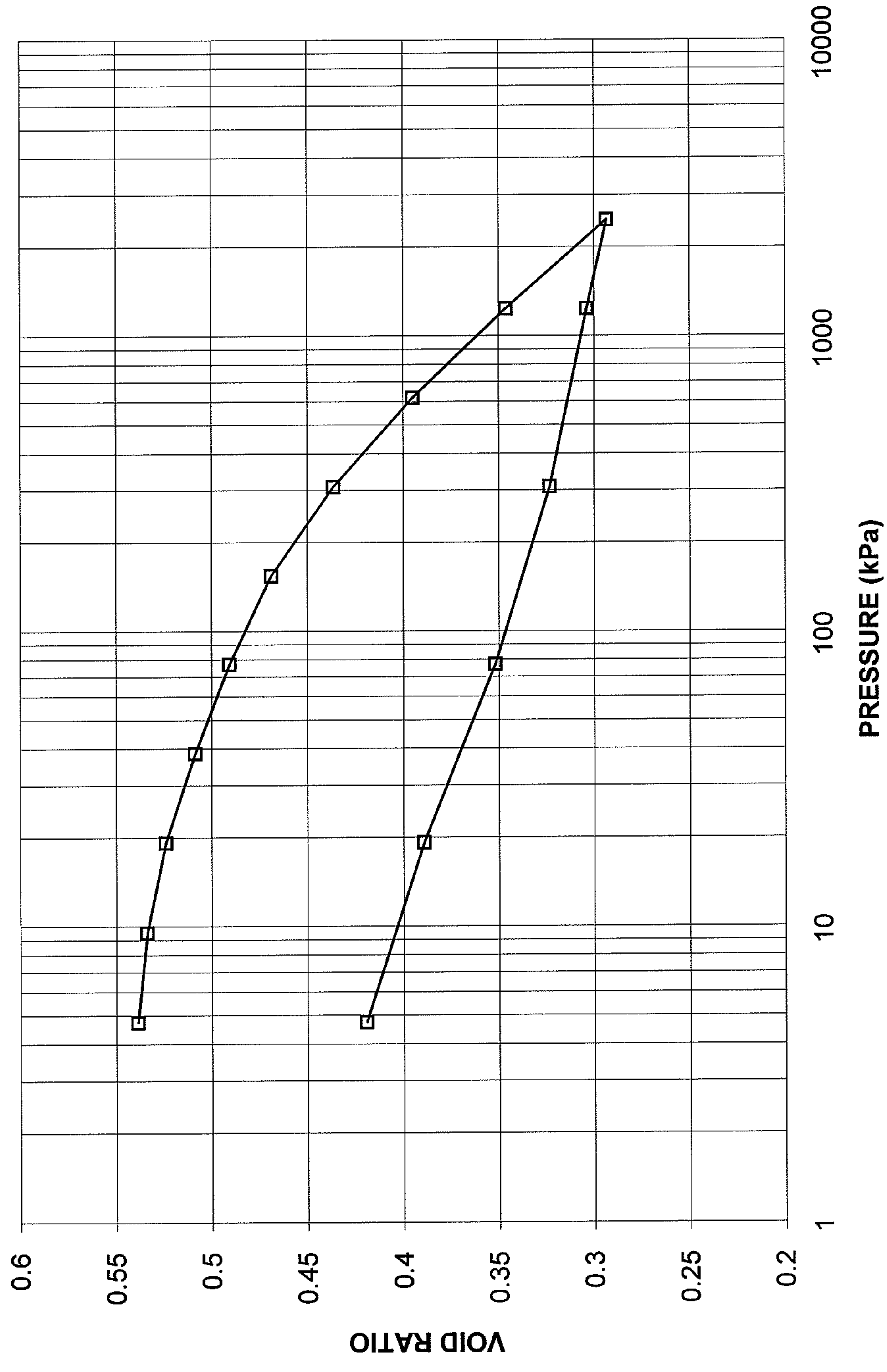
SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.75	Unit Weight, kN/m^3	22.40
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	19.00
Area, cm^2	31.67	Specific Gravity, measured	2.75
Volume, cm^3	55.33	Solids Height, cm	1.231
Water Content, %	17.90	Volume of Solids, cm ^{3}	38.98
Wet Mass, g	126.39	Volume of Voids, cm ^{3}	16.34
Dry Mass, g	107.2		

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

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE A15a



CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH 14 SA 9

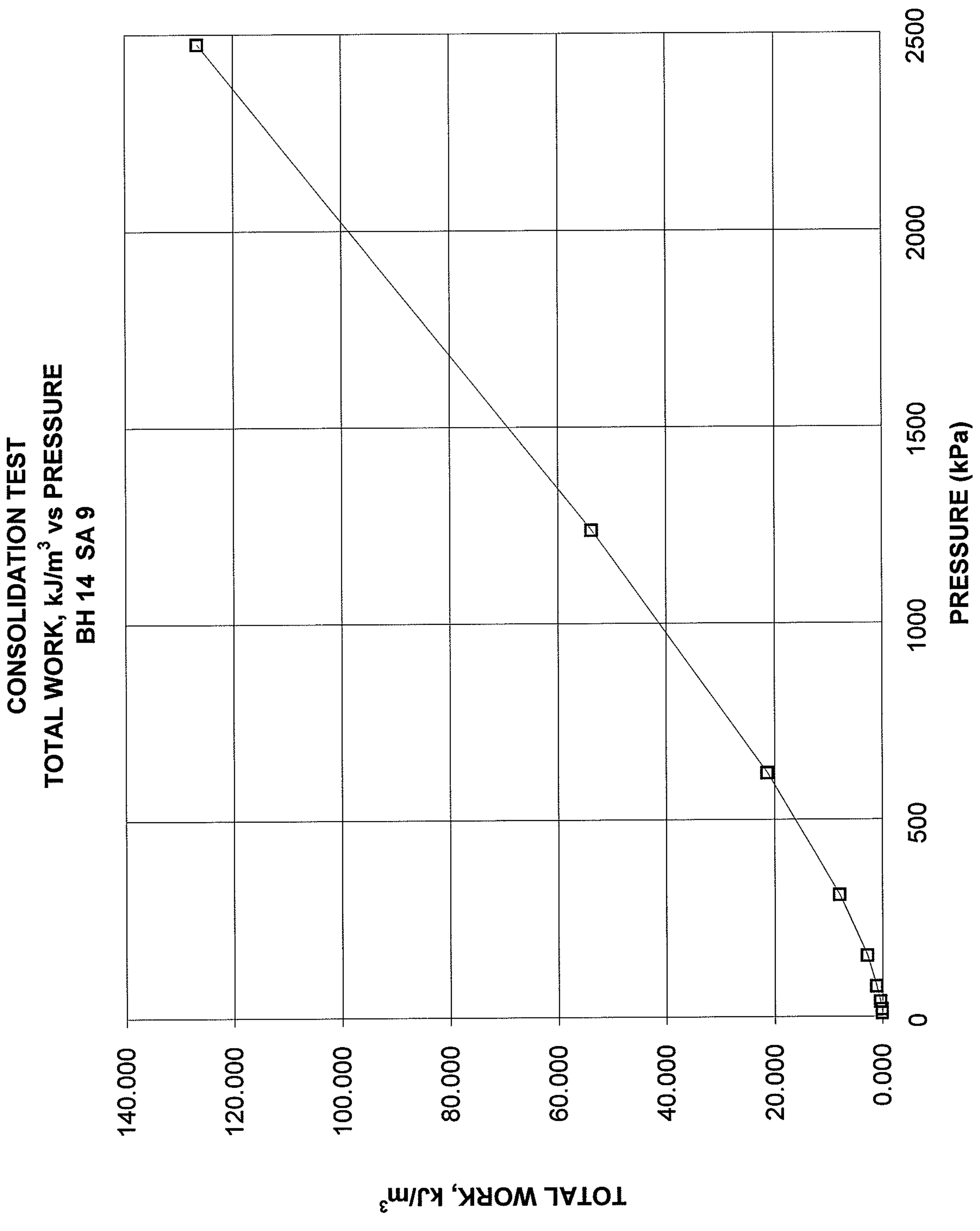
Project No. 04-1111-060

Prepared By: LFG

Golder Associates

Checked By: MM

FIGURE A15b


CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs PRESSURE
BH 14 SA 9

Project No. 04-1111-060

Golder Associates

Prepared By: LFG

Checked By: MM

OEDOMETER CONSOLIDATION SUMMARY

SAMPLE IDENTIFICATION

Project Number	04-1111-060	Sample Number	11
Borehole Number	14	Sample Depth, m	13.1-13.6

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	9		
Date Started	11/20/2006		
Date Completed	12/03/2006		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.92	Unit Weight, kN/m^3	20.18
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	16.38
Area, cm^2	31.52	Specific Gravity, measured	2.76
Volume, cm^3	60.36	Solids Height, cm	1.159
Water Content, %	23.17	Volume of Solids, cm ^{3}	36.53
Wet Mass, g	124.18	Volume of Voids, cm ^{3}	23.83
Dry Mass, g	100.82	Degree of Saturation, %	98.0

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	1.915	0.652	1.915				
4.87	1.911	0.649	1.913	2	3.88E-01	4.29E-04	1.63E-05
9.55	1.906	0.645	1.909	4	1.93E-01	5.58E-04	1.06E-05
19.50	1.895	0.635	1.901	26	2.95E-02	5.77E-04	1.67E-06
38.82	1.878	0.620	1.887	60	1.26E-02	4.59E-04	5.66E-07
77.80	1.854	0.600	1.866	60	1.23E-02	3.22E-04	3.88E-07
155.52	1.821	0.571	1.838	60	1.19E-02	2.22E-04	2.59E-07
310.67	1.777	0.533	1.799	197	3.48E-03	1.48E-04	5.05E-08
621.30	1.721	0.485	1.749	76	8.53E-03	9.41E-05	7.87E-08
1243.24	1.660	0.432	1.691	197	3.08E-03	5.12E-05	1.54E-08
2486.79	1.594	0.375	1.627	68	8.25E-03	2.77E-05	2.24E-08
1243.24	1.605	0.385	1.600				
1243.24	1.605	0.385	1.605				
310.67	1.631	0.407	1.618				
77.80	1.670	0.441	1.651				
19.50	1.712	0.477	1.691				
4.87	1.741	0.502	1.727				

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

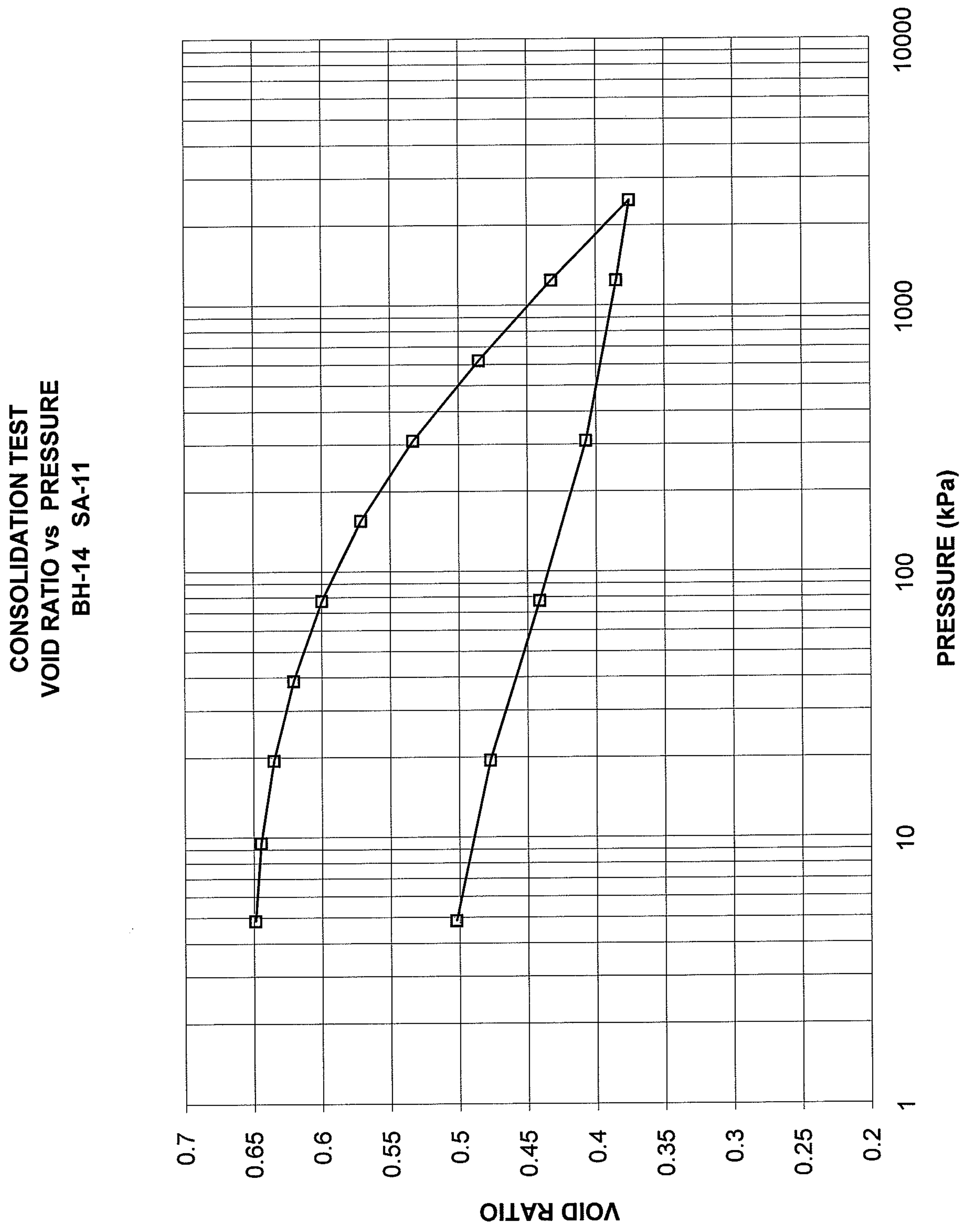
SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.71	Unit Weight, kN/m^3	21.65
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	18.32
Area, cm^2	31.52	Specific Gravity, measured	2.76
Volume, cm^3	53.96	Solids Height, cm	1.159
Water Content, %	18.14	Volume of Solids, cm ^{3}	36.53
Wet Mass, g	119.11	Volume of Voids, cm ^{3}	17.43
Dry Mass, g	100.82		

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

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE A16a



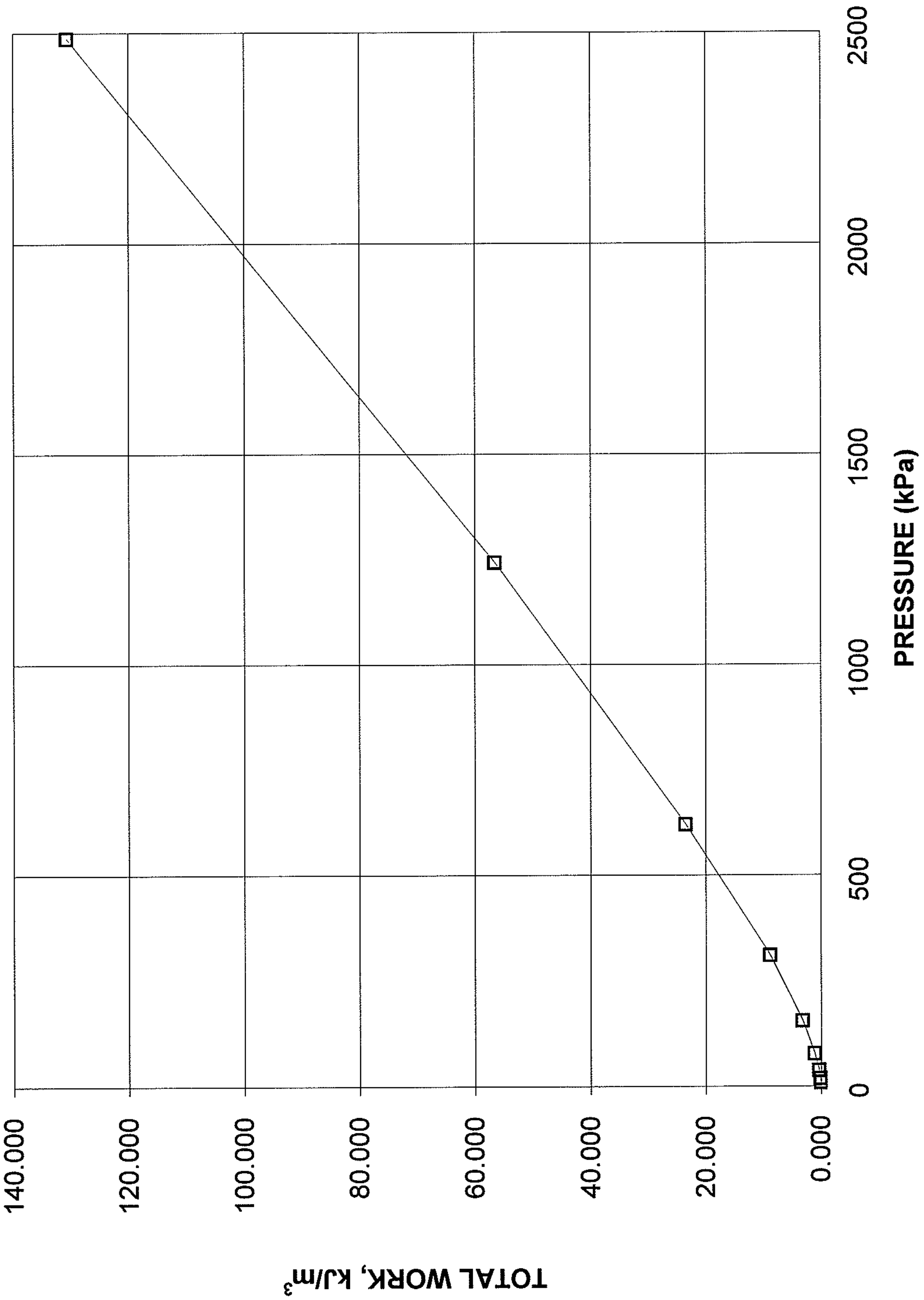
Project No. 04-1111-060
Prepared By: LFG

Golder Associates

Checked By: MM

FIGURE A16b

CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs PRESSURE
BH-14 SA-11



Project No. 04-1111-060
Prepared By: LFG

Golder Associates

Checked By: MM

OEDOMETER CONSOLIDATION SUMMARY

SAMPLE IDENTIFICATION

Project Number	04-1111-060	Sample Number	16
Borehole Number	14	Sample Depth, m	18.6-19.2

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	7		
Date Started	11/24/2006		
Date Completed	12/07/2006		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m^3	21.36
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	18.26
Area, cm^2	31.65	Specific Gravity, measured	2.73
Volume, cm^3	60.13	Solids Height, cm	1.296
Water Content, %	17.00	Volume of Solids, cm ^{3}	41.00
Wet Mass, g	130.97	Volume of Voids, cm ^{3}	19.13
Dry Mass, g	111.94	Degree of Saturation, %	99.5

TEST COMPUTATIONS

Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	1.900	0.467	1.900				
4.83	1.888	0.457	1.894	540	1.41E-03	1.31E-03	1.80E-07
9.46	1.885	0.455	1.887	124	6.08E-03	3.41E-04	2.03E-07
19.51	1.868	0.442	1.877	184	4.06E-03	8.90E-04	3.54E-07
38.91	1.851	0.429	1.860	475	1.54E-03	4.61E-04	6.98E-08
77.57	1.829	0.412	1.840	709	1.01E-03	3.00E-04	2.97E-08
154.88	1.801	0.390	1.815	475	1.47E-03	1.91E-04	2.75E-08
309.33	1.768	0.365	1.785	304	2.22E-03	1.12E-04	2.45E-08
619.10	1.732	0.337	1.750	184	3.53E-03	6.12E-05	2.12E-08
1239.01	1.686	0.301	1.709	85	7.28E-03	3.91E-05	2.79E-08
2477.23	1.638	0.264	1.662	85	6.89E-03	2.04E-05	1.38E-08
1239.01	1.646	0.270	1.642				
309.33	1.665	0.285	1.656				
77.57	1.688	0.303	1.677				
19.51	1.710	0.320	1.699				
4.83	1.733	0.338	1.722				

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

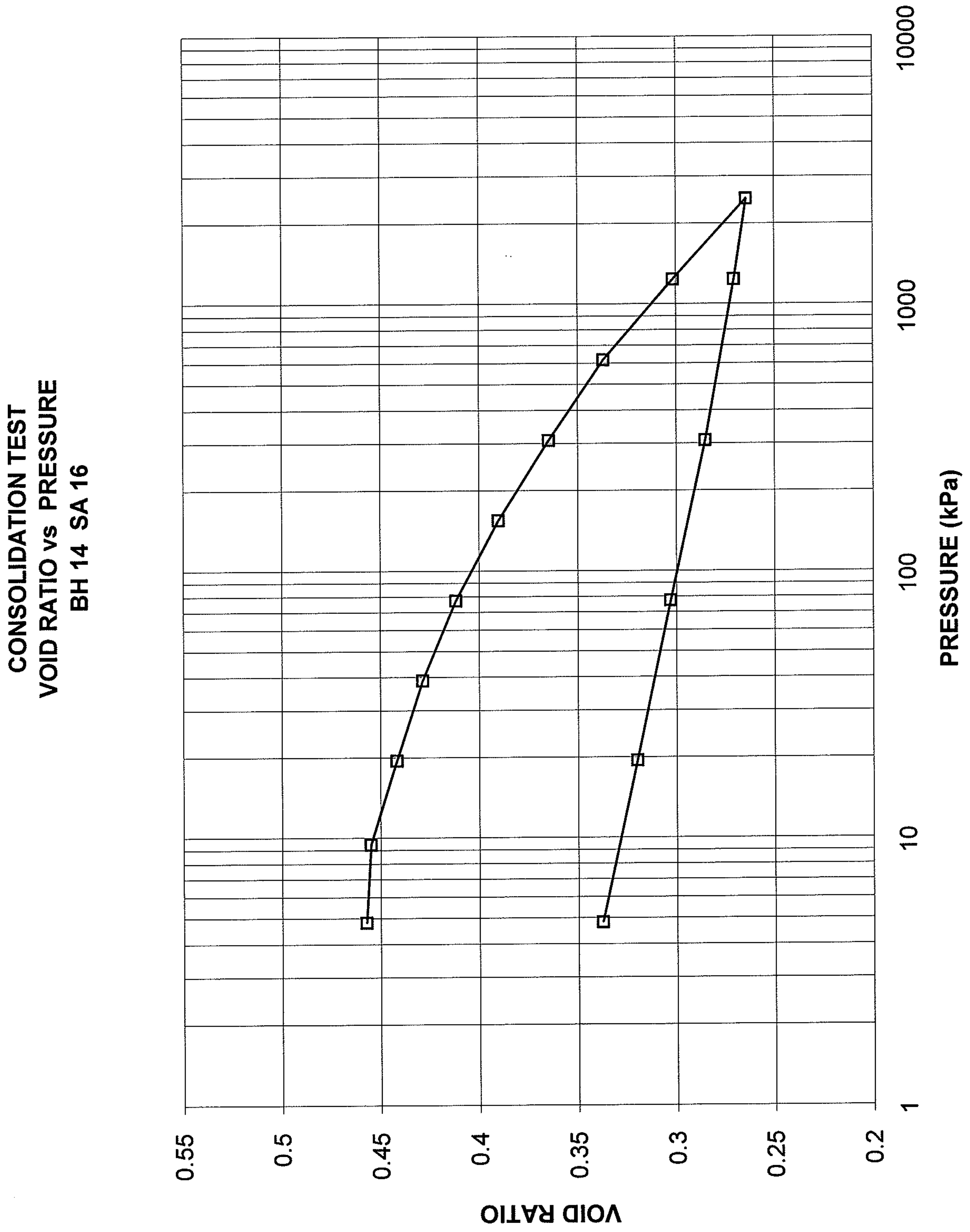
SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.73	Unit Weight, kN/m^3	22.70
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	20.01
Area, cm^2	31.65	Specific Gravity, measured	2.73
Volume, cm^3	54.85	Solids Height, cm	1.296
Water Content, %	13.40	Volume of Solids, cm ^{3}	41.00
Wet Mass, g	126.94	Volume of Voids, cm ^{3}	13.84
Dry Mass, g	111.94		

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

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE A17a



Project No. 04-1111-060

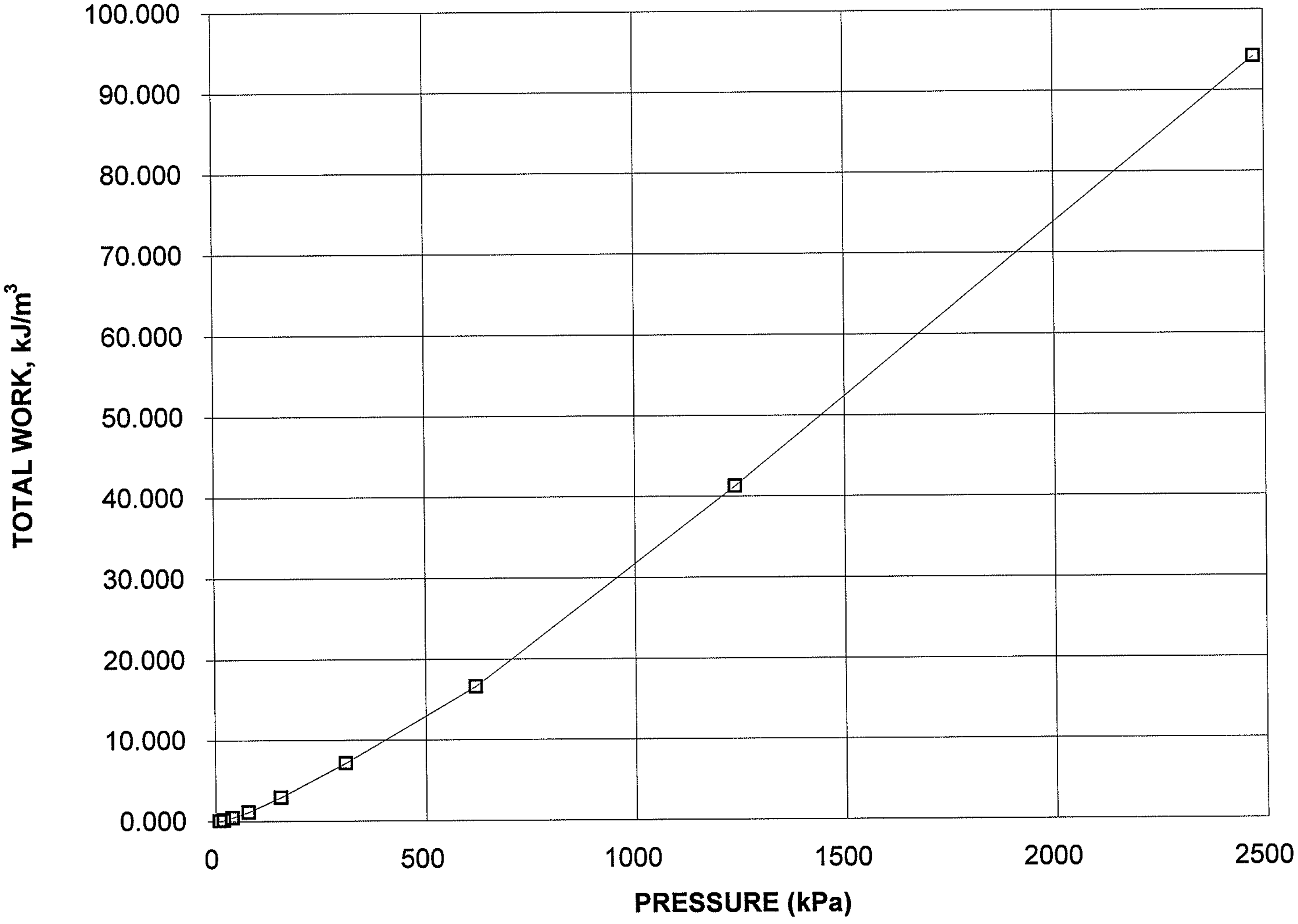
Prepared By: LFG

Golder Associates

Checked By: MM

FIGURE A17b

CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs PRESSURE
BH 14 SA 16



Project No. 04-1111-060

Prepared By: LFG

Golder Associates

Checked By: MM

CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 1 OF 4

FIGURE A18a

TEST STAGE	A	B	C
BOREHOLE NUMBER	14	14	14
SAMPLE	9	11	16
SPECIMEN DIAMETER, cm	4.97	4.98	4.92
SPECIMEN HEIGHT, cm	10.16	10.19	10.10
WATER CONTENT BEFORE CONSOLIDATION, %	23.5	24.1	18.5
CELL PRESSURE, σ_3, kPa	255.0	271.0	228.0
BACK PRESSURE, kPa	205.0	205.0	135.0
PORE PRESSURE PARAMETER "B"	0.96	0.97	0.97
CONSOLIDATION PRESSURE, σc, kPa	50.0	66.0	93.0
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	1.7	2.5	3.9
WATER CONTENT AFTER CONSOLIDATION, %	22.5	22.6	16.4
AVERAGE RATE OF STRAIN, %/hr	0.5	0.5	0.5
TIME TO FAILURE, DAYS	1	1	1
WATER CONTENT AFTER TEST, %	21.3	22.6	15.8
MAX. DEVIATOR STRESS, (σ_1-σ_3), kPa	100.4	97.2	132.8
AXIAL STRAIN AT (σ_1-σ_3) MAXIMUM, %	8.5	5.5	14.7
MAX EFFECTIVE PRINCIPAL STRESS RATIO, (σ_1/σ_3) MAXIMUM	3.9	3.4	3.3
DEVIATOR STRESS AT (σ_1/σ_3) MAXIMUM, kPa	83.6	90.2	123.0
AXIAL STRAIN AT (σ_1/σ_3) MAXIMUM, %	1.9	2.9	7.2
PORE PRESSURE PARAMETER, Af, AT (σ_1-σ_3) MAXIMUM	0.05	0.24	0.24
PORE PRESSURE PARAMETER, Af, AT (σ_1/σ_3) MAXIMUM	0.02	0.31	0.32
NATURAL WATER CONTENT, %	21.6	21.8	17.0
DRY DENSITY, Mg/m^3	1.71	1.68	1.84
FILTER DRAINS USED, y/n	y	y	y
TEST NOTES:			
CHANGED RATE OF STRAIN, %/hr	-	-	-
AXIAL STRAIN WHERE RATE OF STRAIN WAS CHANGED, %	-	-	-
FAILURE PLANE NUMBER	-	1.0	1.0
ANGLE OF FAILURE, DEGREES	-	60.0	55.0

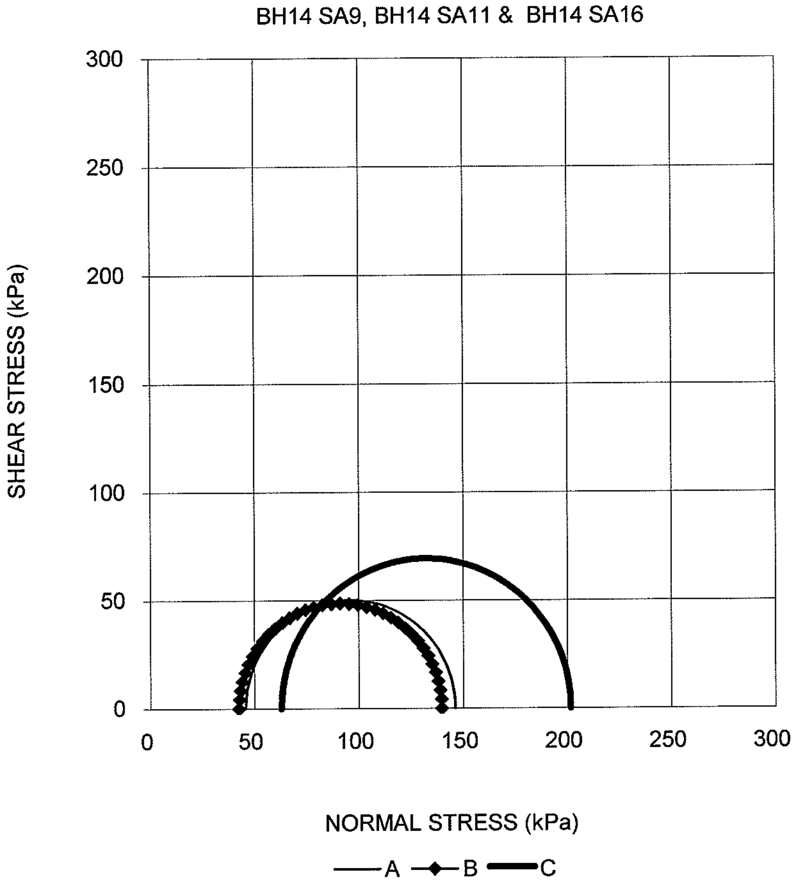
Date: 11/30/2006
Project No. 04-1111-060

Golder Associates

Prepared By LFG
Checked By: SJB

CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 2 OF 4

FIGURE A18b



Date: 11/10/2006

Project No. 04-1111-060

Golder Associates

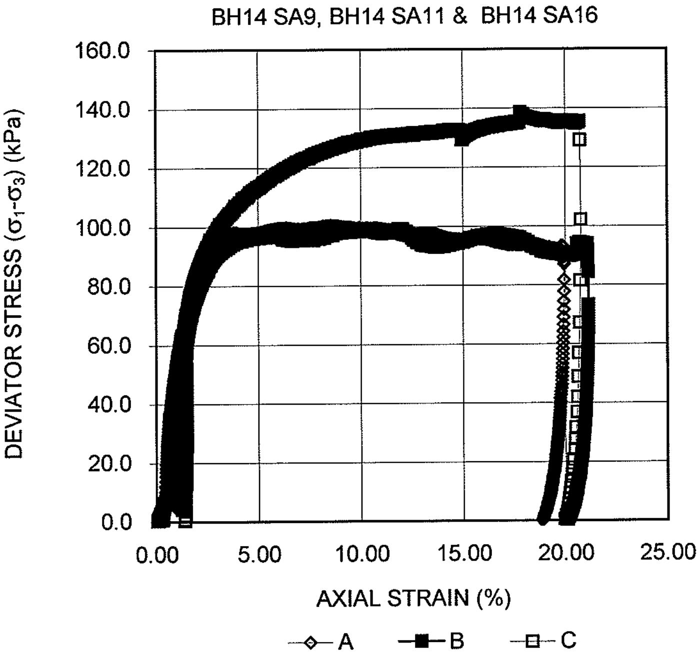
Prepared By LFG

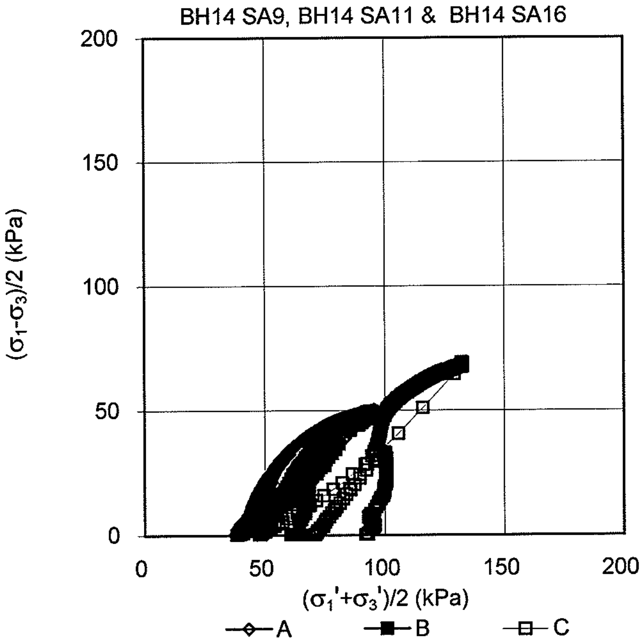
Checked By: MM

CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS

SHEET 3 OF 4

FIGURE A18c





Date: 11/10/2006

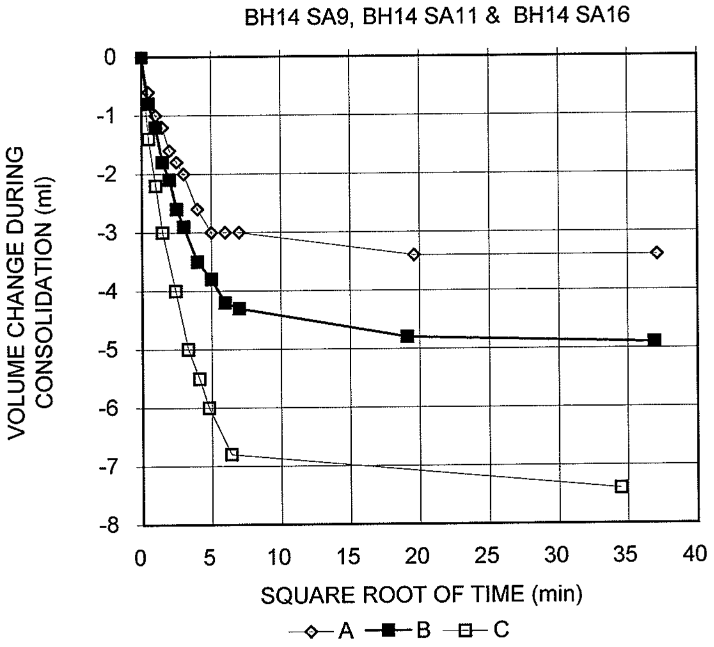
Project No. 04-1111-060

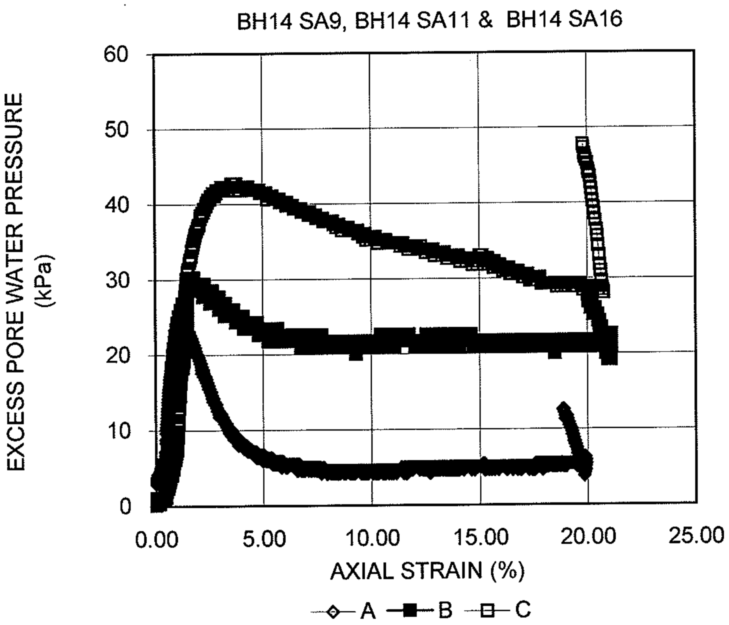
Golder Associates

Prepared By LFG

Checked By: SJB

CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS SHEET 4 OF 4	FIGURE A18d





Date: 11/10/2006

Project No. 04-1111-060

Golder Associates

Prepared By LFG

Checked By: SJB

CARBONATE TEST DETERMINATION

Borehole Number		14	14	14	
Sample Number		9	11	16	
Depth, m		10.1-10.5	13.1-13.6	18.6-19.2	
TEST DATA ENTRY					
Sample Weight, g	(A)	1.71	1.72	1.70	
First Reading, ml	(B)	49.00	50.40	59.00	
Second Reading, ml	(C)	101.00	100.00	137.00	
Room Temperature, °C	(D)	23.60	24.30	23.30	
Flask Temperature, °C	(E)	25.20	25.00	25.00	
Barometer, kPa	(F)	101.07	101.07	101.07	
Flask Temp. / Barometer Correction	(G)	1.02608	1.02608	1.02608	
TEST CALCULATIONS					
CORRECTED READINGS					
First Reading, BxG		50.28	51.71	60.54	
Second Reading, CxG		103.63	102.61	140.57	
Dolomite, CxG-BxG	(E)	53.36	50.89	80.03	
Calcite, (BxG)-0.04((CxG)-(BxG))	(F)	48.14	49.68	57.34	
CARBONATE PERCENTAGES FROM TABLES					
Dolomite, %	(H)	12.30	11.80	18.30	
Calcite, %	(I)	11.20	11.50	13.30	
Total, %	(H+I)	23.50	23.30	31.60	
Ratio	(I/H)	0.91	0.97	0.73	

Project Number	04-1111-060	Tested By	Angela
Date of Testing	1/18/2007	Entered By	LG
Remarks		Checked By	SJB

Golder Associates

UNCONFINED COMPRESSION TEST (UC)

SAMPLE IDENTIFICATION

PROJECT NUMBER	04-1111-060	SAMPLE NUMBER	3
BOREHOLE NUMBER	14	SAMPLE DEPTH, m	37.0-37.2

TEST CONDITIONS

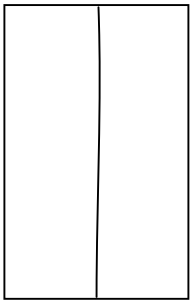
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.25

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.58	WATER CONTENT, (specimen) %	0.17
SAMPLE DIAMETER, cm	4.70	UNIT WEIGHT, kN/m^3	24.24
SAMPLE AREA, cm^2	17.35	DRY UNIT WT., kN/m^3	24.20
SAMPLE VOLUME, cm^3	183.56	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	453.96	VOID RATIO	0.09
DRY WEIGHT, g	453.19		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

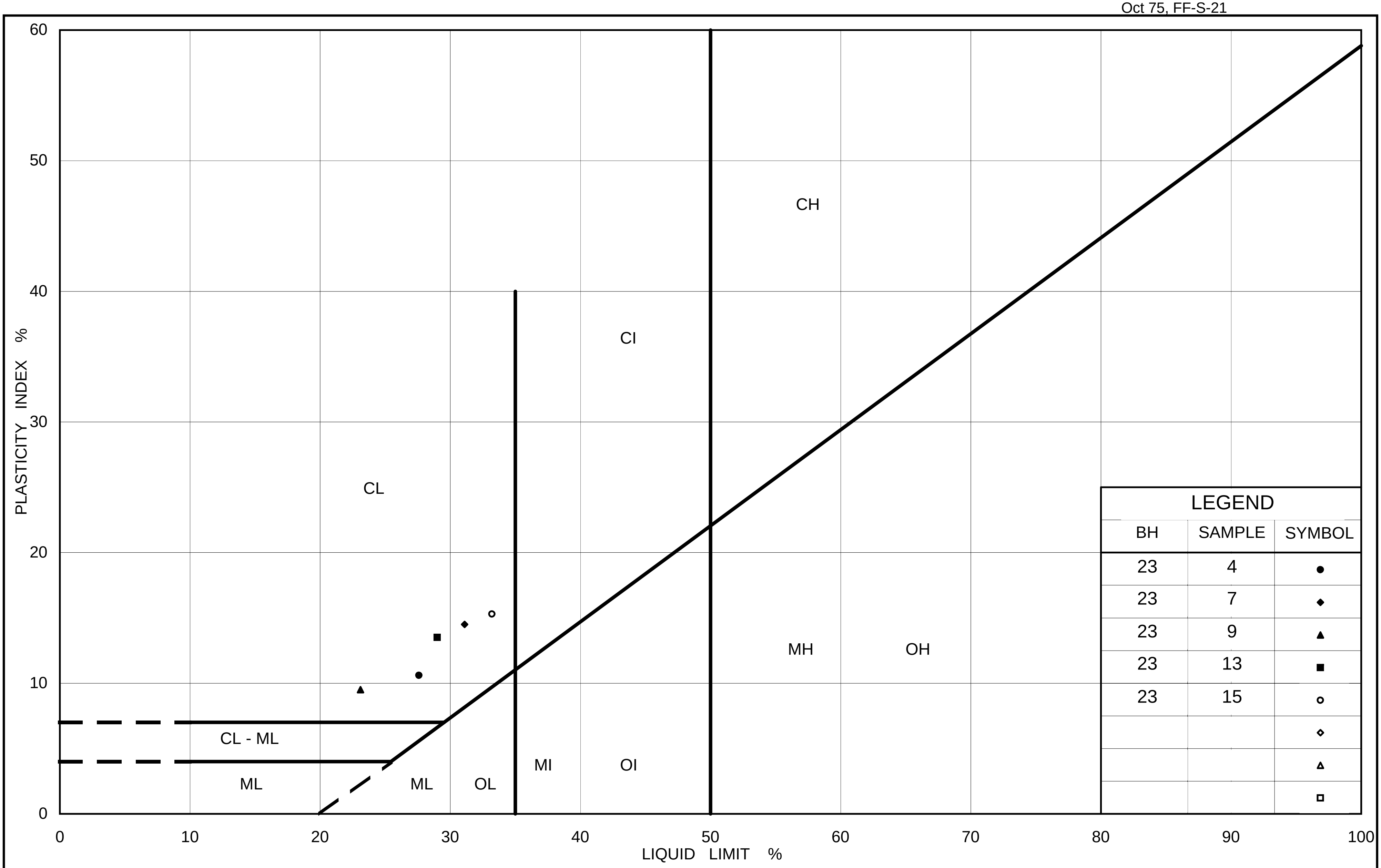
STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	36.4

REMARKS:

DATE: 09/02/2007

Golder Associates

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART

FIG No. A19a

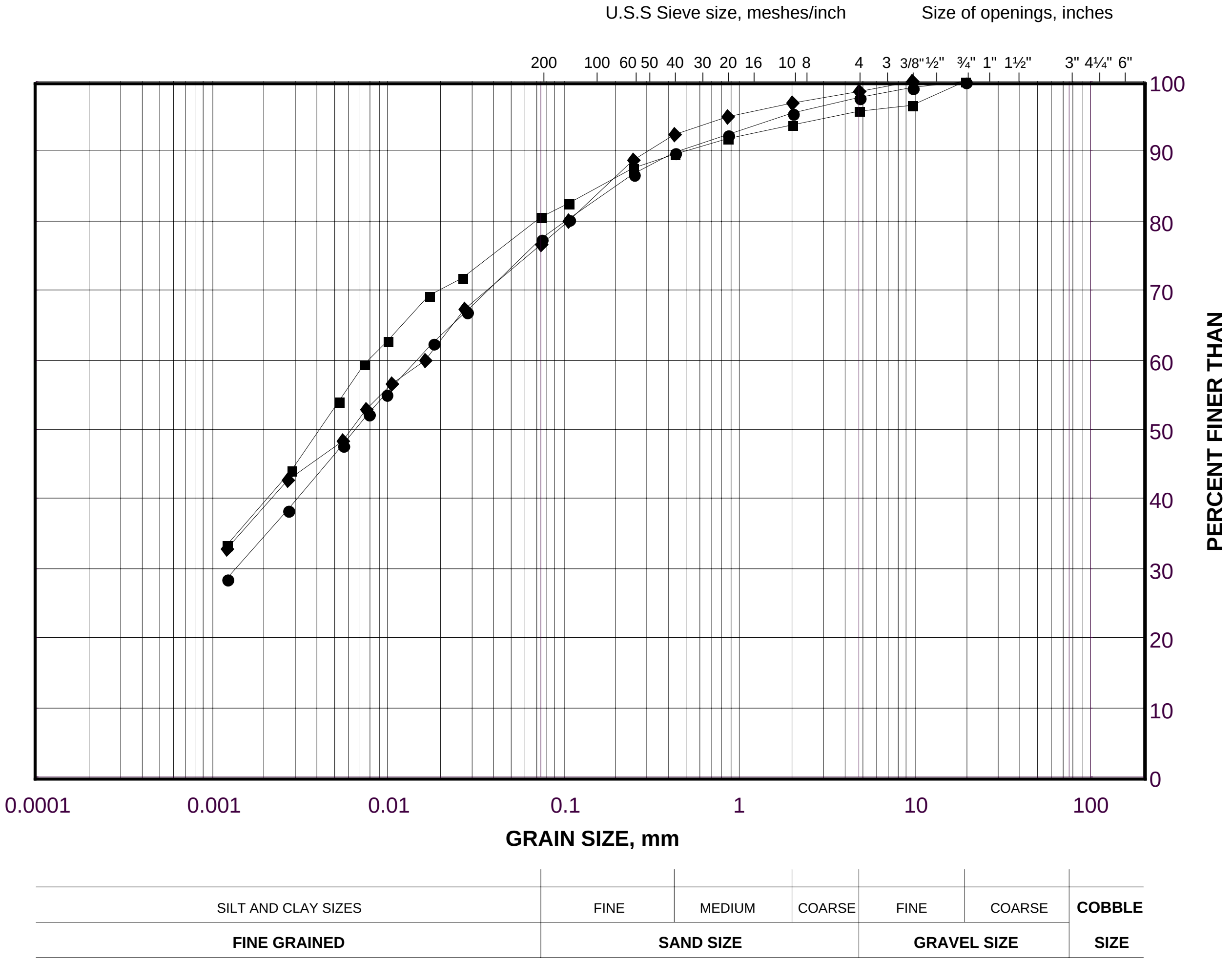
Project No. 04-1111-060

Checked By: SJB

GRAIN SIZE DISTRIBUTION

Clayey Silt to Silty Clay Deposit

FIGURE A20



LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH(m)
●	23	13	16.80 - 17.20
■	23	15	19.20 - 19.70
◆	23	7	8.50 - 9.00

Project Number: 04-1111-060

Checked By: SJB

Golder Associates

Date: 06-Jun-07

OEDOMETER CONSOLIDATION SUMMARY

SAMPLE IDENTIFICATION

Project Number	04-1111-060	Sample Number	7
Borehole Number	23	Sample Depth, m	8.5-9.0

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	5		
Date Started	11/27/2006		
Date Completed	12/10/2006		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.91	Unit Weight, kN/m^3	19.86
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	15.80
Area, cm^2	31.65	Specific Gravity, measured	2.75
Volume, cm^3	60.45	Solids Height, cm	1.119
Water Content, %	25.72	Volume of Solids, cm ^{3}	35.42
Wet Mass, g	122.45	Volume of Voids, cm ^{3}	25.03
Dry Mass, g	97.4	Degree of Saturation, %	100.1

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	1.910	0.707	1.910				
4.85	1.868	0.669	1.889	8	9.46E-02	4.53E-03	4.20E-05
9.54	1.863	0.665	1.866	34	2.17E-02	5.58E-04	1.19E-06
19.29	1.853	0.656	1.858	68	1.08E-02	5.37E-04	5.66E-07
38.71	1.838	0.642	1.846	80	9.03E-03	4.04E-04	3.58E-07
77.44	1.813	0.620	1.826	240	2.94E-03	3.38E-04	9.75E-08
154.88	1.767	0.579	1.790	240	2.83E-03	3.11E-04	8.63E-08
309.17	1.694	0.514	1.731	158	4.02E-03	2.48E-04	9.75E-08
618.53	1.624	0.451	1.659	540	1.08E-03	1.18E-04	1.25E-08
1237.69	1.556	0.390	1.590	394	1.36E-03	5.75E-05	7.67E-09
2479.12	1.493	0.334	1.525	184	2.68E-03	2.66E-05	6.97E-09
1237.29	1.499	0.339	1.496				
309.36	1.525	0.363	1.512				
77.44	1.563	0.397	1.544				
19.29	1.602	0.432	1.583				
4.82	1.634	0.460	1.618				

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

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.63	Unit Weight, kN/m^3	21.99
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	18.47
Area, cm^2	31.65	Specific Gravity, measured	2.75
Volume, cm^3	51.71	Solids Height, cm	1.119
Water Content, %	19.03	Volume of Solids, cm ^{3}	35.42
Wet Mass, g	115.94	Volume of Voids, cm ^{3}	16.30
Dry Mass, g	97.4		

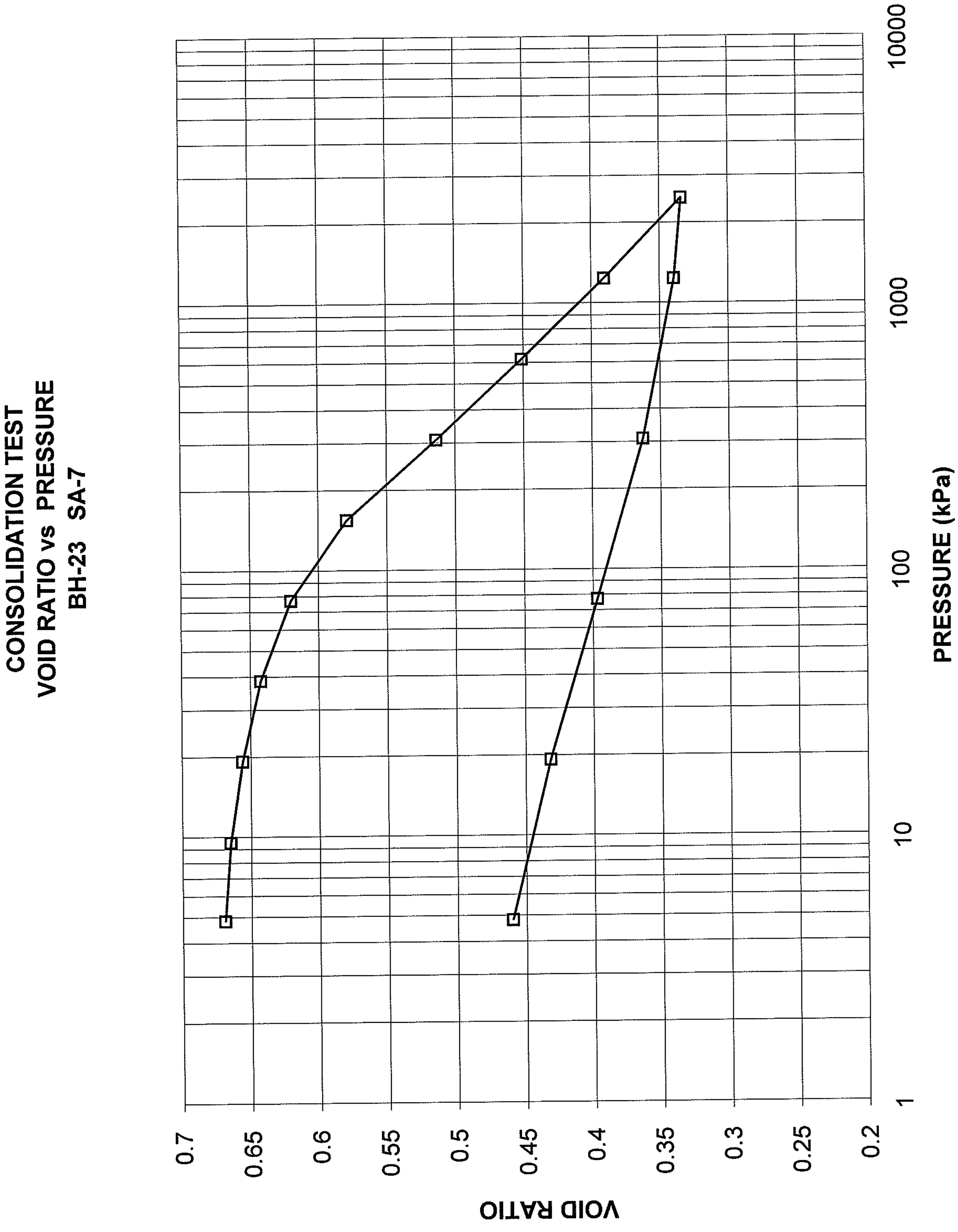
Prepared By: LFG

Golder Associates

Checked By: MM

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE A21a



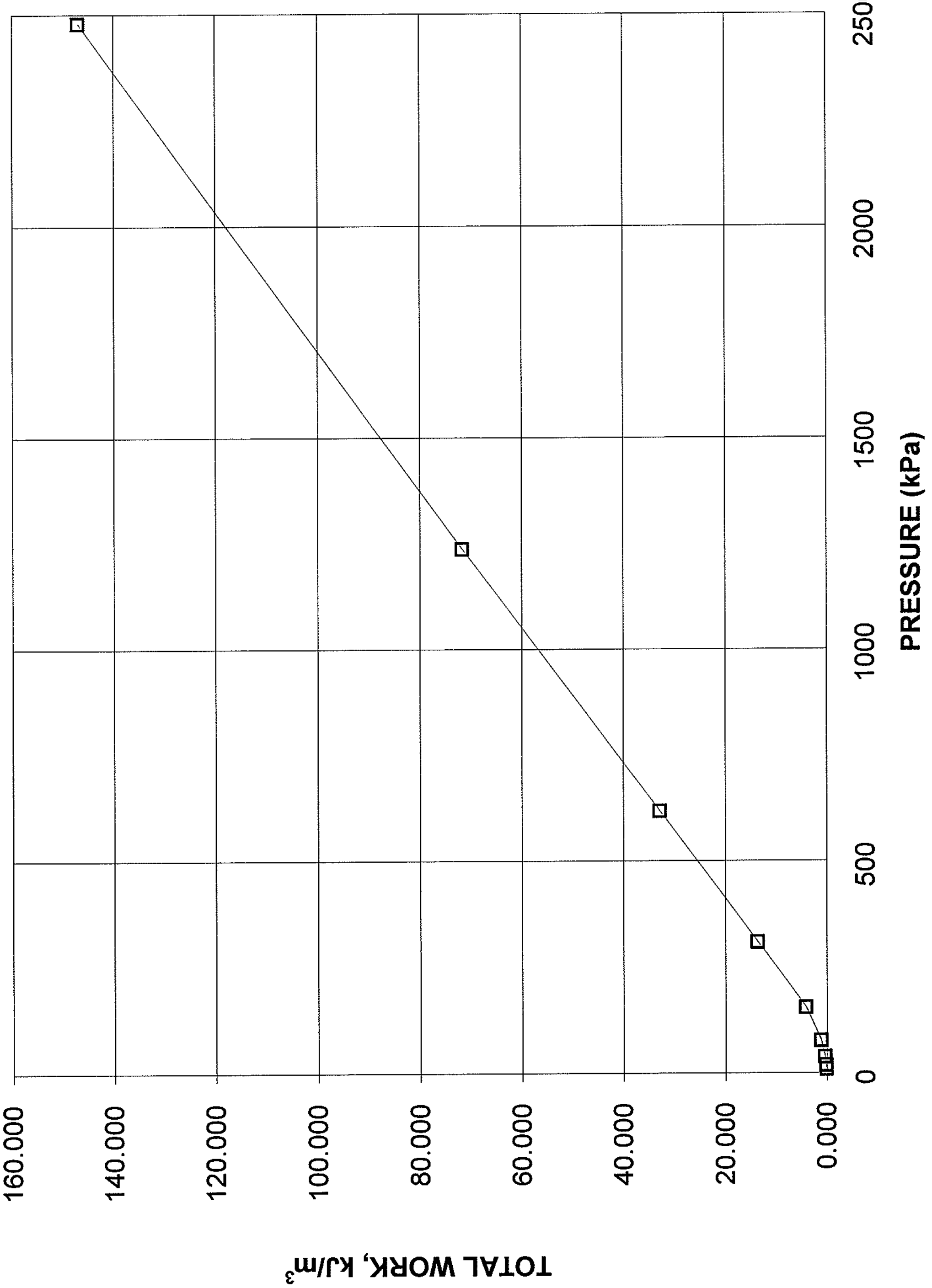
Project No. 04-1111-060
Prepared By: LFG

Golder Associates

Checked By: MM

FIGURE A21b

CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs PRESSURE
BH-23 SA-7



Project No. 04-1111-060
Prepared By: LFG

Golder Associates

Checked By: MM

OEDOMETER CONSOLIDATION SUMMARY

SAMPLE IDENTIFICATION

Project Number	04-1111-060	Sample Number	13
Borehole Number	23	Sample Depth, m	16.8-17.2

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	11		
Date Started	11/27/2006		
Date Completed	12/09/2006		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	2.54	Unit Weight, kN/m^3	21.20
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	17.78
Area, cm^2	31.67	Specific Gravity, measured	2.76
Volume, cm^3	80.44	Solids Height, cm	1.668
Water Content, %	19.24	Volume of Solids, cm ^{3}	52.83
Wet Mass, g	173.87	Volume of Voids, cm ^{3}	27.61
Dry Mass, g	145.82	Degree of Saturation, %	101.6

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	2.540	0.523	2.540				
4.82	2.523	0.512	2.532	60	2.26E-02	1.39E-03	3.08E-06
9.54	2.514	0.507	2.519	103	1.31E-02	7.51E-04	9.60E-07
19.25	2.497	0.497	2.506	103	1.29E-02	6.89E-04	8.73E-07
38.68	2.477	0.485	2.487	124	1.06E-02	4.05E-04	4.20E-07
77.38	2.450	0.469	2.464	85	1.51E-02	2.75E-04	4.07E-07
154.90	2.413	0.446	2.432	324	3.87E-03	1.88E-04	7.12E-08
309.64	2.361	0.415	2.387	454	2.66E-03	1.32E-04	3.45E-08
618.65	2.303	0.380	2.332	540	2.14E-03	7.39E-05	1.55E-08
1236.69	2.238	0.341	2.271	338	3.23E-03	4.14E-05	1.31E-08
2473.91	2.171	0.301	2.205	304	3.39E-03	2.13E-05	7.08E-09
1236.69	2.178	0.306	2.175				
309.64	2.205	0.322	2.192				
77.52	2.246	0.346	2.226				
18.97	2.286	0.370	2.266				
4.82	2.317	0.389	2.302				

Note:
k calculated using cv based on t $_{90}$ values.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

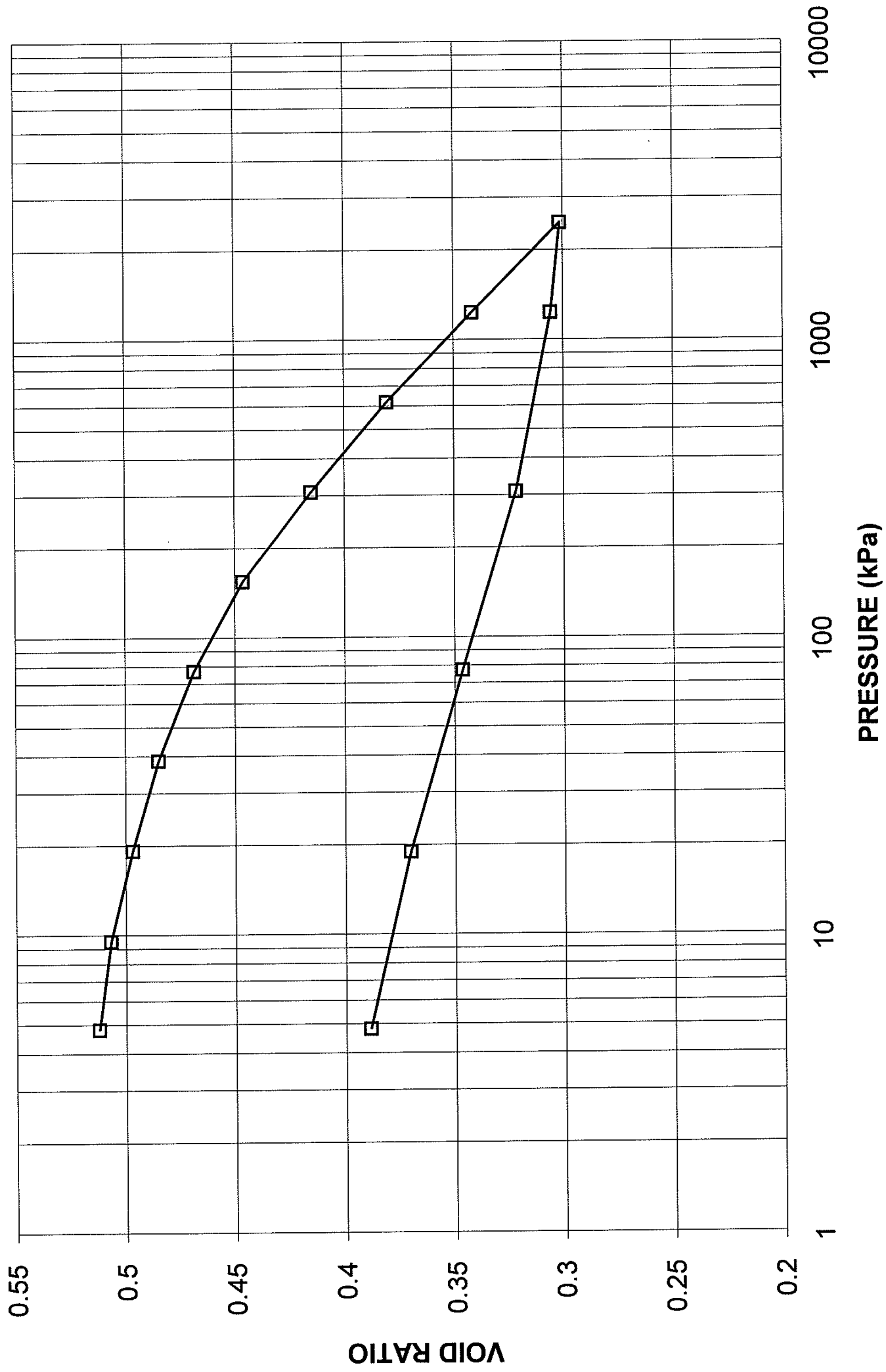
Sample Height, cm	2.32	Unit Weight, kN/m^3	22.59
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	19.49
Area, cm^2	31.67	Specific Gravity, measured	2.76
Volume, cm^3	73.38	Solids Height, cm	1.668
Water Content, %	15.90	Volume of Solids, cm ^{3}	52.83
Wet Mass, g	169.00	Volume of Voids, cm ^{3}	20.54
Dry Mass, g	145.82		

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

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE A22a

CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH-23 SA-13



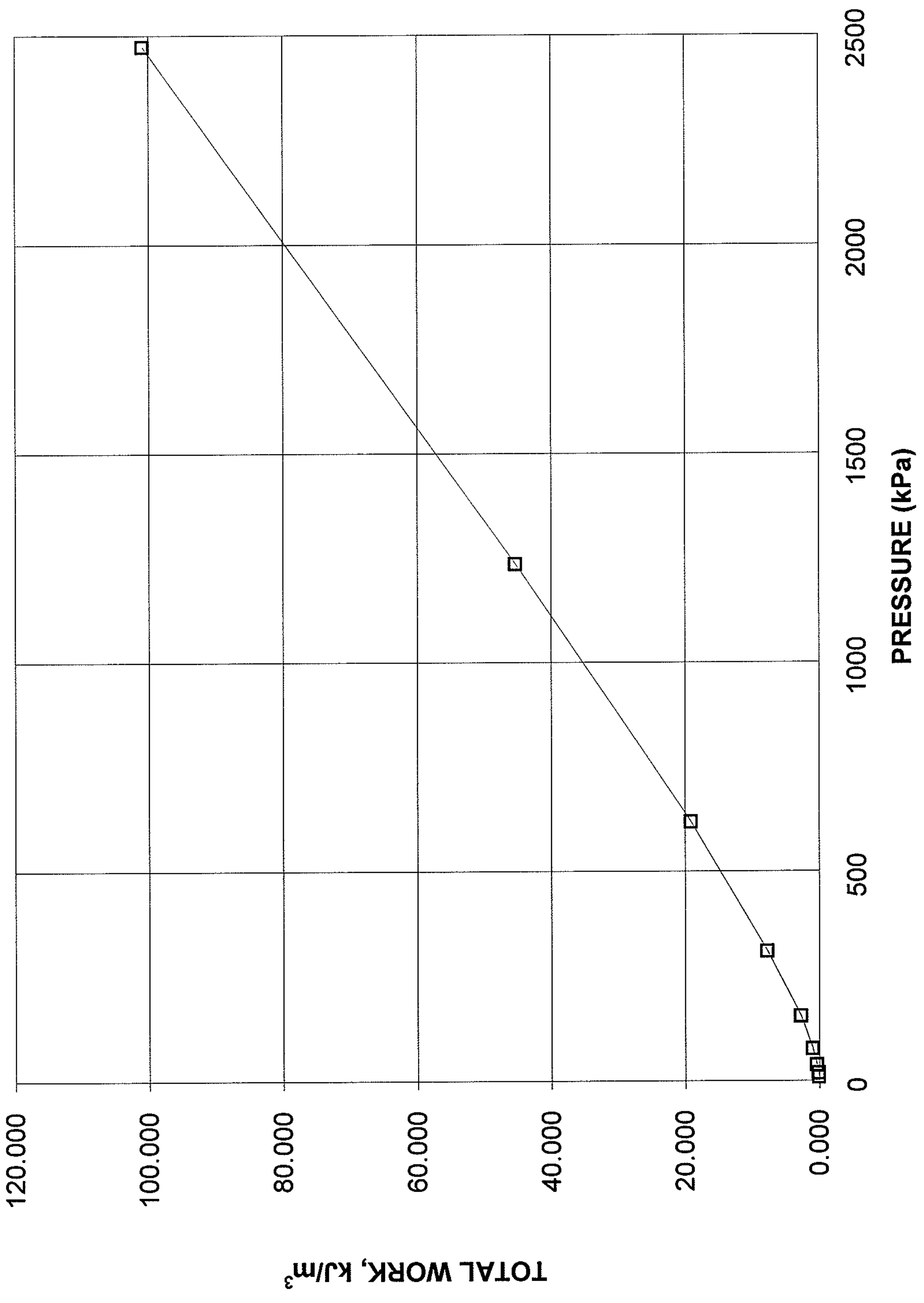
Project No. 04-1111-060
Prepared By: LFG

Golder Associates

Checked By: MM

FIGURE A22b

CONSOLIDATION TEST
TOTAL WORK, kJ/m^3 vs PRESSURE
BH-23 SA-13



Project No. 04-1111-060
Prepared By: LFG

Golder Associates

Checked By: MM

OEDOMETER CONSOLIDATION SUMMARY

SAMPLE IDENTIFICATION

Project Number	04-1111-060	Sample Number	15
Borehole Number	23	Sample Depth, m	19.2-19.7

TEST CONDITIONS

Test Type	Standard	Load Duration, hr	24
Oedometer Number	6		
Date Started	11/28/2006		
Date Completed	12/11/2006		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m^3	21.61
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	18.26
Area, cm^2	31.67	Specific Gravity, measured	2.75
Volume, cm^3	60.17	Solids Height, cm	1.286
Water Content, %	18.36	Volume of Solids, cm ^{3}	40.73
Wet Mass, g	132.58	Volume of Voids, cm ^{3}	19.44
Dry Mass, g	112.01	Degree of Saturation, %	105.8

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	0.477	1.900				
4.69	1.889	0.469	1.895	11	6.92E-02	1.23E-03	8.37E-06
9.53	1.879	0.461	1.884	23	3.27E-02	1.09E-03	3.49E-06
19.28	1.863	0.449	1.871	60	1.24E-02	8.64E-04	1.05E-06
38.69	1.843	0.433	1.853	53	1.37E-02	5.42E-04	7.30E-07
77.39	1.822	0.417	1.833	68	1.05E-02	2.86E-04	2.93E-07
154.78	1.796	0.396	1.809	34	2.04E-02	1.77E-04	3.54E-07
309.73	1.759	0.368	1.778	60	1.12E-02	1.26E-04	1.37E-07
619.39	1.714	0.333	1.737	40	1.60E-02	7.65E-05	1.20E-07
1238.36	1.662	0.292	1.688	60	1.01E-02	4.42E-05	4.36E-08
2479.37	1.601	0.245	1.632	124	4.55E-03	2.59E-05	1.15E-08
1238.36	1.614	0.255	1.608				
309.73	1.643	0.277	1.629				
77.39	1.681	0.307	1.662				
19.28	1.721	0.338	1.701				
4.69	1.756	0.365	1.739				

Note:
k calculated using cv based on t $_{90}$ values.

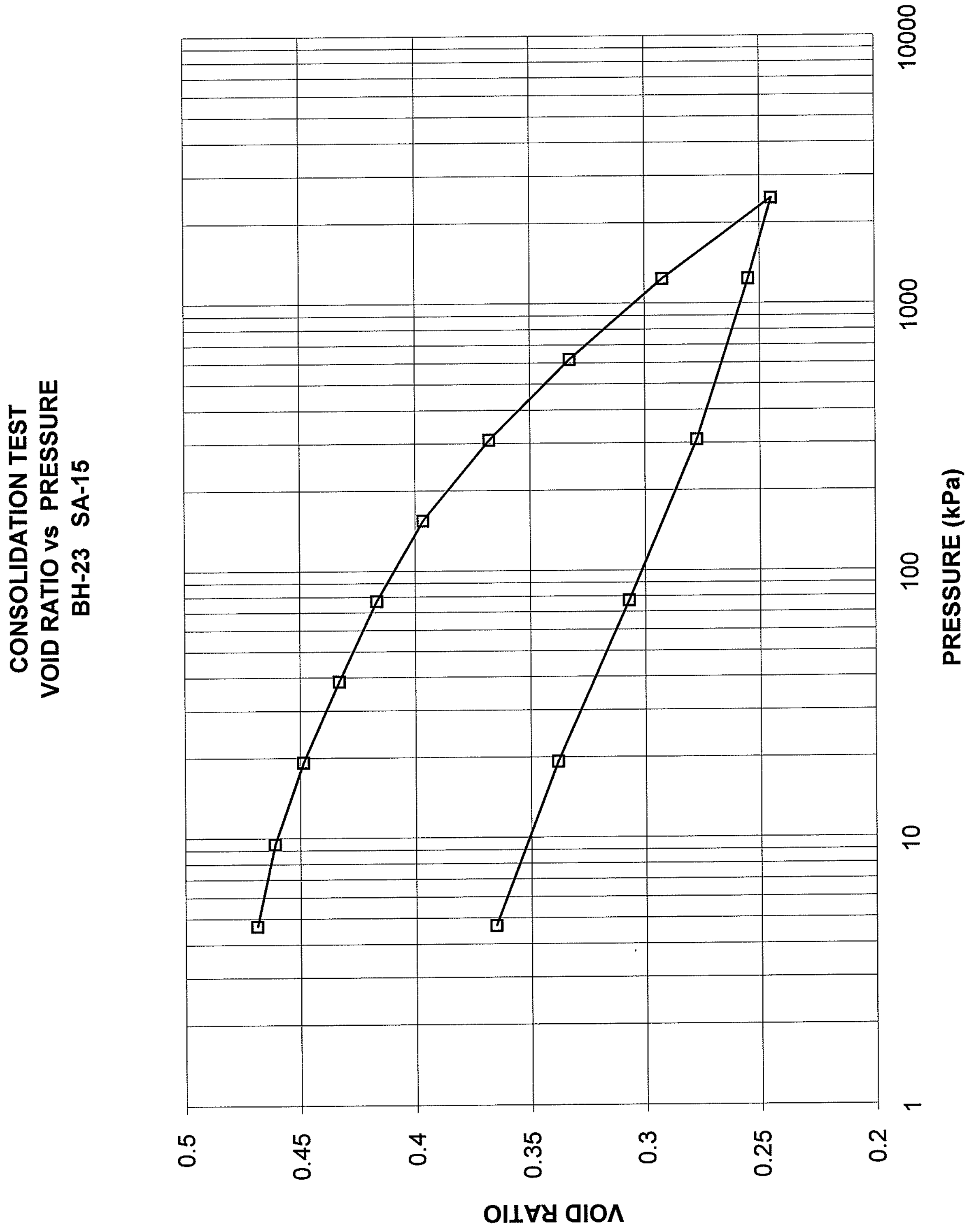
SAMPLE DIMENSIONS AND PROPERTIES - FINAL

Sample Height, cm	1.76	Unit Weight, kN/m^3	23.32
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m^3	19.75
Area, cm^2	31.67	Specific Gravity, measured	2.75
Volume, cm^3	55.61	Solids Height, cm	1.286
Water Content, %	18.07	Volume of Solids, cm ^{3}	40.73
Wet Mass, g	132.25	Volume of Voids, cm ^{3}	14.88
Dry Mass, g	112.01		

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

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE A23a



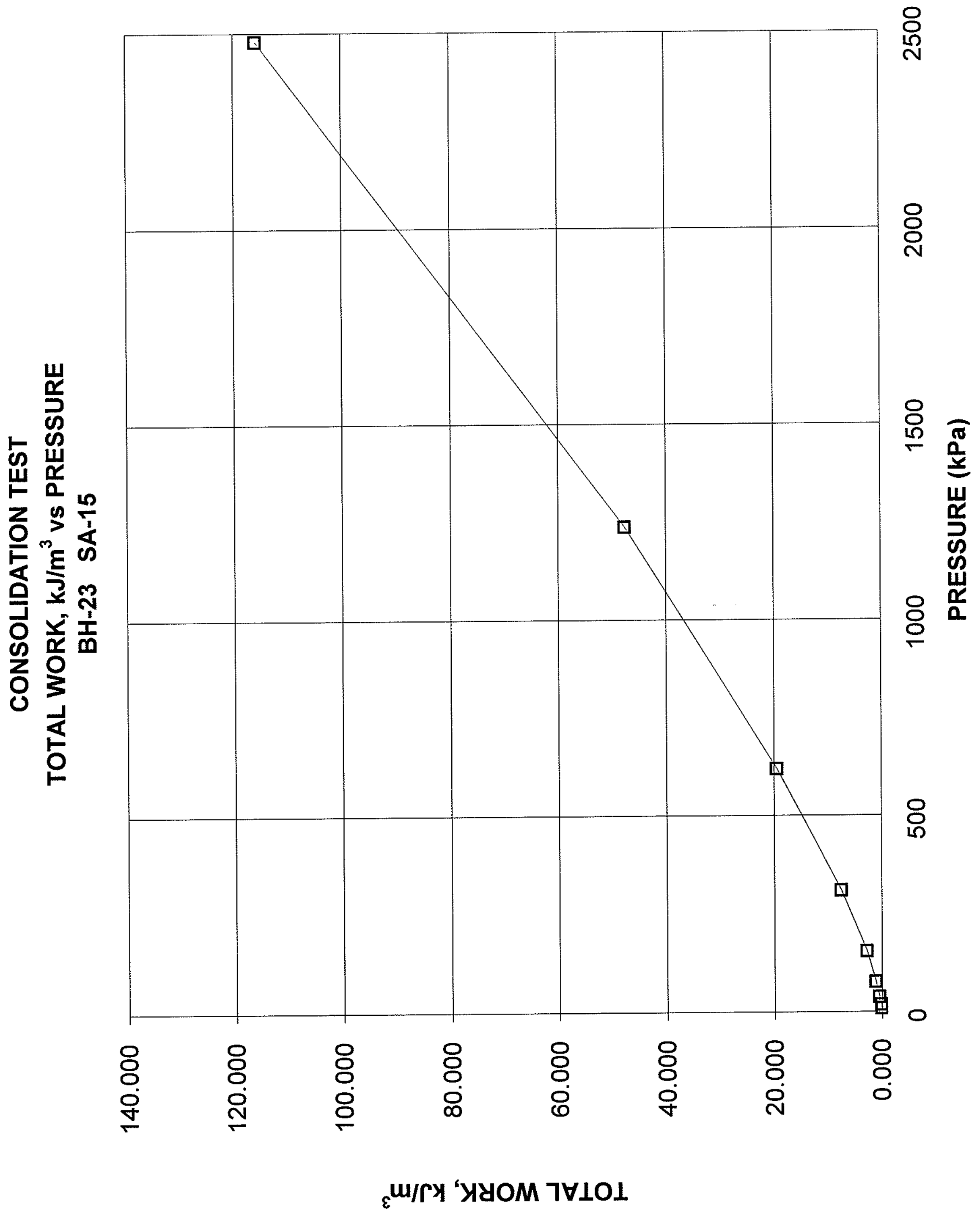
Project No. 04-1111-060

Prepared By: LFG

Golder Associates

Checked By: MM

FIGURE A23b



Project No. 04-1111-060
Prepared By: LFG

Golder Associates

Checked By: MM

CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 1 OF 4

FIGURE A24a

TEST STAGE	A	B	C
BOREHOLE NUMBER	23	23	23
SAMPLE	7	13	15
SPECIMEN DIAMETER, cm	5.13	4.95	5.04
SPECIMEN HEIGHT, cm	10.18	10.10	10.13
WATER CONTENT BEFORE CONSOLIDATION, %	29.6	20.1	22.1
CELL PRESSURE, σ_3, kPa	178.0	289.0	371.0
BACK PRESSURE, kPa	135.0	205.0	275.0
PORE PRESSURE PARAMETER "B"	0.96	0.97	0.96
CONSOLIDATION PRESSURE, σc, kPa	43.0	84.0	96.0
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	1.9	3.6	2.7
WATER CONTENT AFTER CONSOLIDATION, %	28.3	18.1	20.8
AVERAGE RATE OF STRAIN, %/hr	0.5	0.5	0.5
TIME TO FAILURE, DAYS	1	1	1
WATER CONTENT AFTER TEST, %	29.4	17.2	20.8
MAX. DEVIATOR STRESS, (σ_1-σ_3), kPa	56.1	155.0	189.1
AXIAL STRAIN AT (σ_1-σ_3) MAXIMUM, %	8.9	17.9	20.4
MAX EFFECTIVE PRINCIPAL STRESS RATIO, (σ_1/σ_3) MAXIMUM	3.6	3.6	3.0
DEVIATOR STRESS AT (σ_1/σ_3) MAXIMUM, kPa	53.1	117.6	133.5
AXIAL STRAIN AT (σ_1/σ_3) MAXIMUM, %	2.4	5.3	4.0
PORE PRESSURE PARAMETER, Af, AT (σ_1-σ_3) MAXIMUM	0.34	0.12	-0.09
PORE PRESSURE PARAMETER, Af, AT (σ_1/σ_3) MAXIMUM	0.43	0.33	0.22
NATURAL WATER CONTENT, %	31.6	18.8	20.5
DRY DENSITY, Mg/m^3	1.53	1.82	1.74
FILTER DRAINS USED, y/n	y	y	y
TEST NOTES:			
CHANGED RATE OF STRAIN, %/hr	-	-	-
AXIAL STRAIN WHERE RATE OF STRAIN WAS CHANGED, %	-	-	-
FAILURE PLANE NUMBER	1.0	2.0	1.0
ANGLE OF FAILURE, DEGREES	40.0	40.0	55.0

Date: 12/04/2006
Project No. 04-1111-060

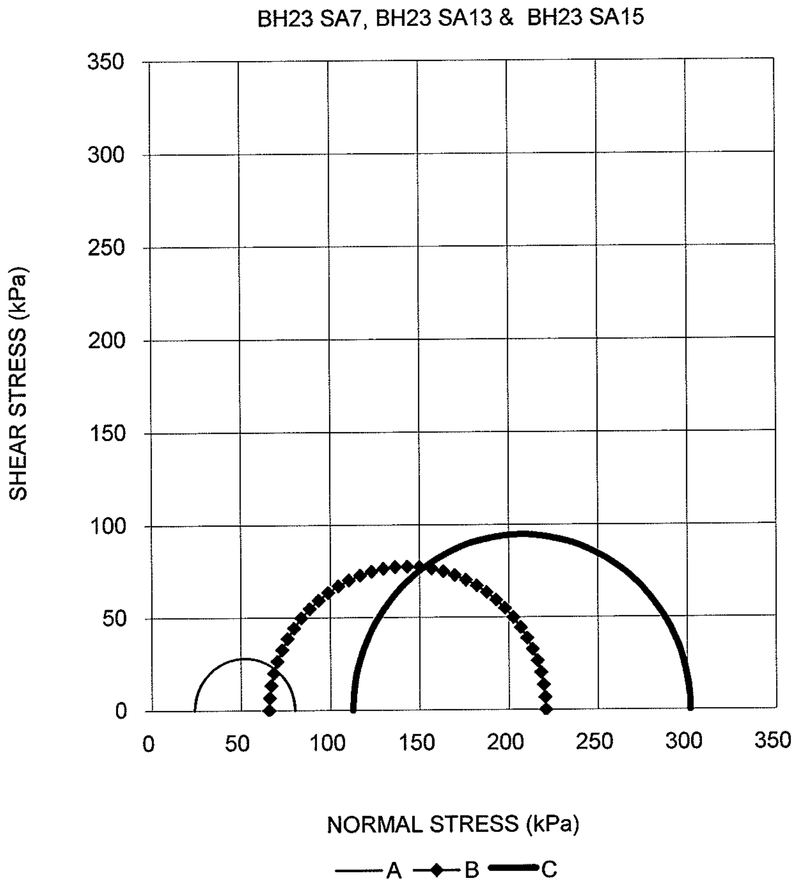
Golder Associates

Prepared By LFG
Checked By: SJB

CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS

SHEET 2 OF 4

FIGURE A24b



Date: 12/04/2006

Project No. 04-1111-060

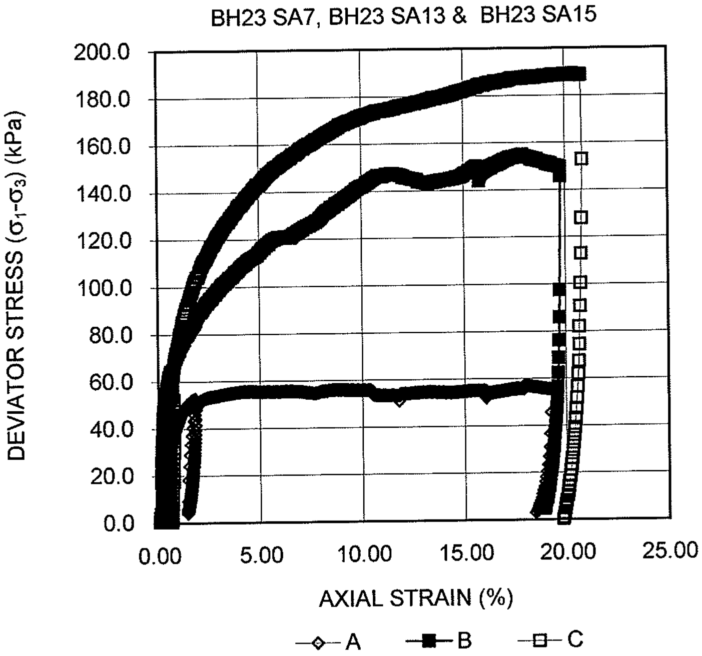
Golder Associates

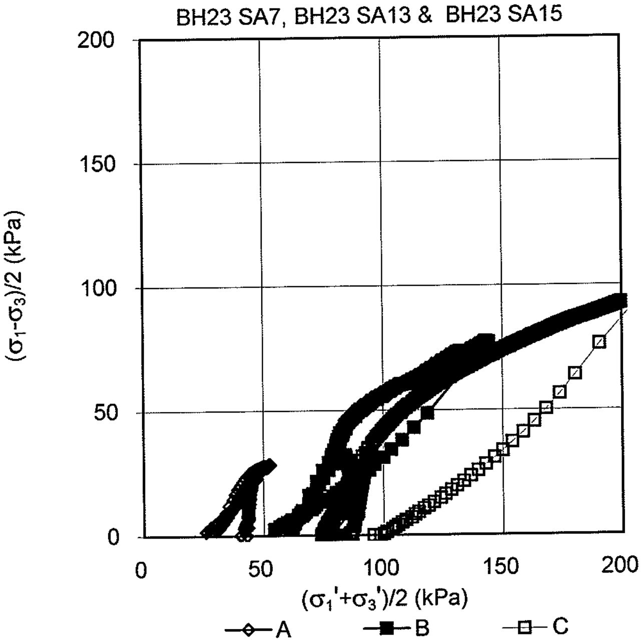
Prepared By LFG

Checked By: MM

CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 3 OF 4

FIGURE A24c





Date: 12/04/2006

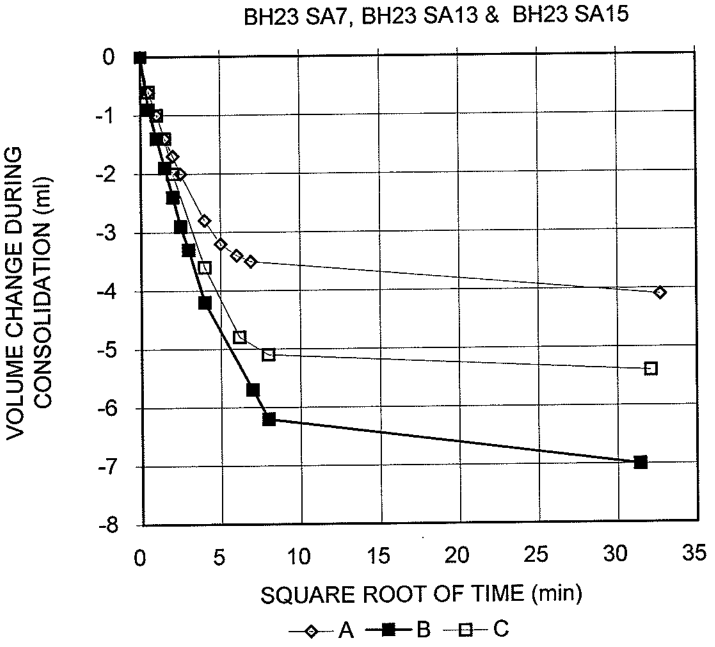
Project No. 04-1111-060

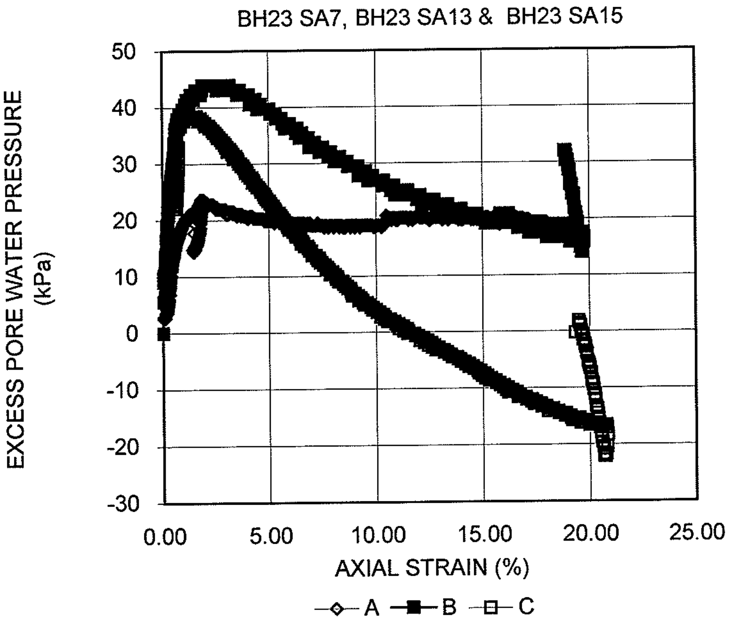
Golder Associates

Prepared By LFG

Checked By: SJB

CONSOLIDATED UNDRAINED TRIAXIAL
WITH PORE PRESSURE MEASUREMENTS
SHEET 4 OF 4





Date: 12/04/2006
Project No. 04-1111-060

Golder Associates

Prepared By LFG
Checked By: SJB

CARBONATE TEST DETERMINATION

Borehole Number		23	23	23	23 (repeat)
Sample Number		7	13	15	15
Depth, m		8.5-9.0	16.8-17.2	19.2-19.7	19.2-19.7
TEST DATA ENTRY					
Sample Weight, g	(A)	1.71	1.72	1.71	1.74
First Reading, ml	(B)	39.00	50.00	44.50	50.00
Second Reading, ml	(C)	84.00	115.00	104.00	106.00
Room Temperature, °C	(D)	22.80	24.10	22.80	23.00
Flask Temperature, °C	(E)	25.20	25.10	24.80	24.80
Barometer, kPa	(F)	101.07	101.07	101.07	101.07
Flask Temp. / Barometer Correction	(G)	1.02608	1.02608	1.02608	1.02608
TEST CALCULATIONS					
CORRECTED READINGS					
First Reading, BxG		40.02	51.30	45.66	51.30
Second Reading, CxG		86.19	118.00	106.71	108.76
Dolomite, CxG-BxG	(E)	46.17	66.70	61.05	57.46
Calcite, (BxG)-0.04((CxG)-(BxG))	(F)	38.17	48.64	43.22	49.01
CARBONATE PERCENTAGES FROM TABLES					
Dolomite, %	(H)	10.70	15.30	14.10	13.20
Calcite, %	(I)	8.80	11.30	10.00	11.40
Total, %	(H+I)	19.50	26.60	24.10	24.60
Ratio	(I/H)	0.82	0.74	0.71	0.86

Project Number	04-1111-060	Tested By	Angela
Date of Testing	1/18/2007	Entered By	LG
Remarks		Checked By SJB	

UNCONFINED COMPRESSION TEST (UC)

SAMPLE IDENTIFICATION

PROJECT NUMBER	04-1111-060	SAMPLE NUMBER	4
BOREHOLE NUMBER	23	SAMPLE DEPTH, m	24.5-24.7

TEST CONDITIONS

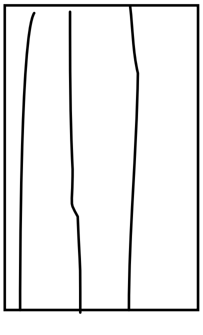
MACHINE SPEED, mm/min	-	TYPE OF SPECIMEN	Rock Core
DURATION OF TEST,min	>2 <15	L/D	2.28

SPECIMEN INFORMATION

SAMPLE HEIGHT, cm	10.72	WATER CONTENT, (specimen) %	0.43
SAMPLE DIAMETER, cm	4.70	UNIT WEIGHT, kN/m^3	24.90
SAMPLE AREA, cm^2	17.35	DRY UNIT WT., kN/m^3	24.79
SAMPLE VOLUME, cm^3	185.99	SPECIFIC GRAVITY, assumed	2.70
WET WEIGHT, g	472.37	VOID RATIO	0.07
DRY WEIGHT, g	470.35		

VISUAL INSPECTION

FAILURE SKETCH



TEST RESULTS

STRAIN AT FAILURE, %	-	COMPRESSIVE STRESS, MPa	55.4

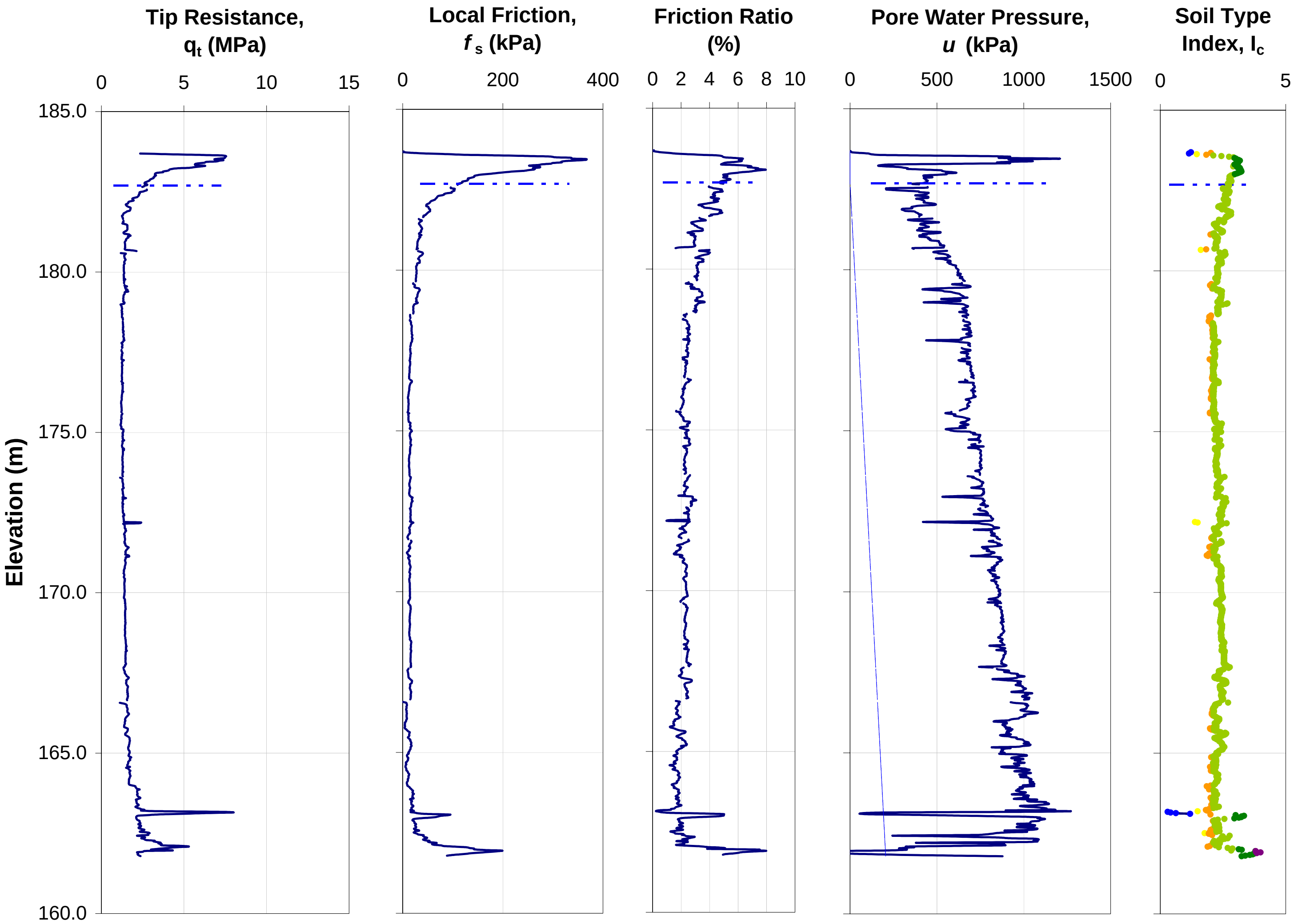
REMARKS:

DATE: 09/02/2007

Golder Associates

APPENDIX B

CONE PENETRATION TEST DATA



Notes:

1. Soil Index Type, Ic, is an inferred soil type and actual conditions may differ.
2. Soil Index Type based on "Guidelines for Geotechnical Design Using CPT and CPTU", PK Robertson and RG Campanella 1989, Robertson and Wride, 1998, and/or site specific data.
3. See report text for limitations of testing and interpretation of data

Legend:

Ic < 1.30	Gravelly Sand	🟦
1.30 < Ic < 1.80	Sand to Silty Sand	🟨
1.80 < Ic < 2.10	Silty Sand to Sandy Silt	🟧
2.10 < Ic < 2.95	Clayey Silt to Silty Clay	🟩
2.95 < Ic < 3.80	Clay	🟩
3.80 < Ic	Clay and Organics	🟪
Static Groundwater Surface	—·—··—	
Static Groundwater Pressure	··········	

Operator Initials:	CC	Easting - X Coordinate (m):	335,502.0
Cone Identification Number:	3806	Northing - Y Coordinate (m):	4,677,739.0
Correction Factor a	0.584	Ground Surface Elevation (m):	186.7
Correction Factor b	0.012	Depth to Groundwater (m):	4.0
		Predrilling Depth (m):	3.0

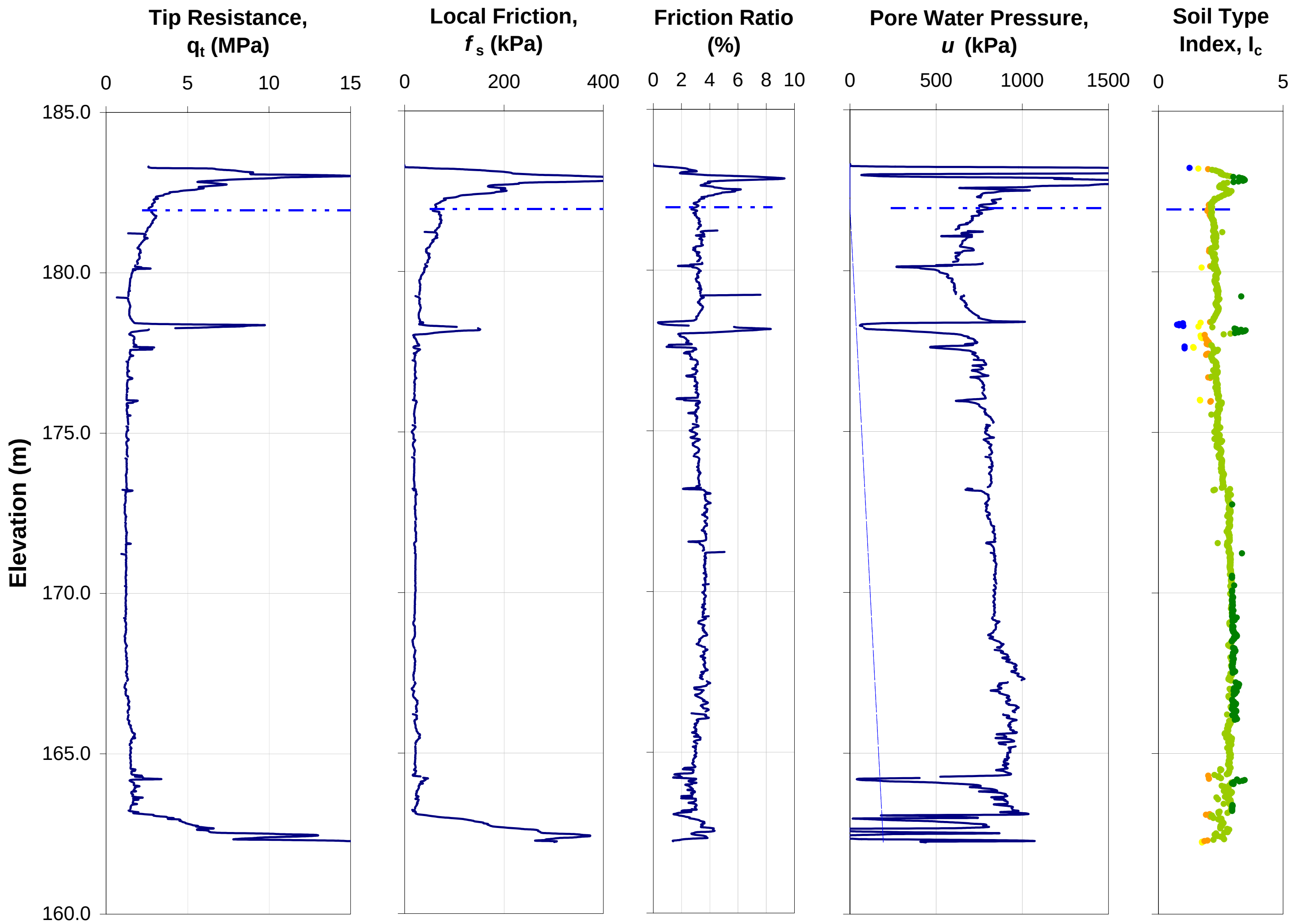
Golder Associates

Project No.:	04-1111-060
Date:	06/11/2006
Check:	SJB
Review:	FJH

Detroit River International Crossing

Cone Penetration Test

1



Notes:

1. Soil Index Type, Ic, is an inferred soil type and actual conditions may differ.
2. Soil Index Type based on "Guidelines for Geotechnical Design Using CPT and CPTU", PK Robertson and RG Campanella 1989, Robertson and Wride, 1998, and/or site specific data.
3. See report text for limitations of testing and interpretation of data

Legend:

	Ic <	1.30 Gravelly Sand	🟦
1.30	< Ic <	1.80 Sand to Silty Sand	🟨
1.80	< Ic <	2.10 Silty Sand to Sandy Silt	🟧
2.10	< Ic <	2.95 Clayey Silt to Silty Clay	🟩
2.95	< Ic <	3.60 Clay	🟩
3.60	< Ic	Clay and Organic Soils	🟪

Static Groundwater Surface — · · — · · —

Static Groundwater Pressure ··························

Operator Initials:	CC	Easting - X Coordinate (m):	335,185.0
Cone Identification Number:	3742	Northing - Y Coordinate (m):	4,677,841.0
Correction Factor a	0.6	Ground Surface Elevation (m):	186.4
Correction Factor b	0.013	Depth to Groundwater (m):	4.4
		Predrilling Depth (m):	3.1

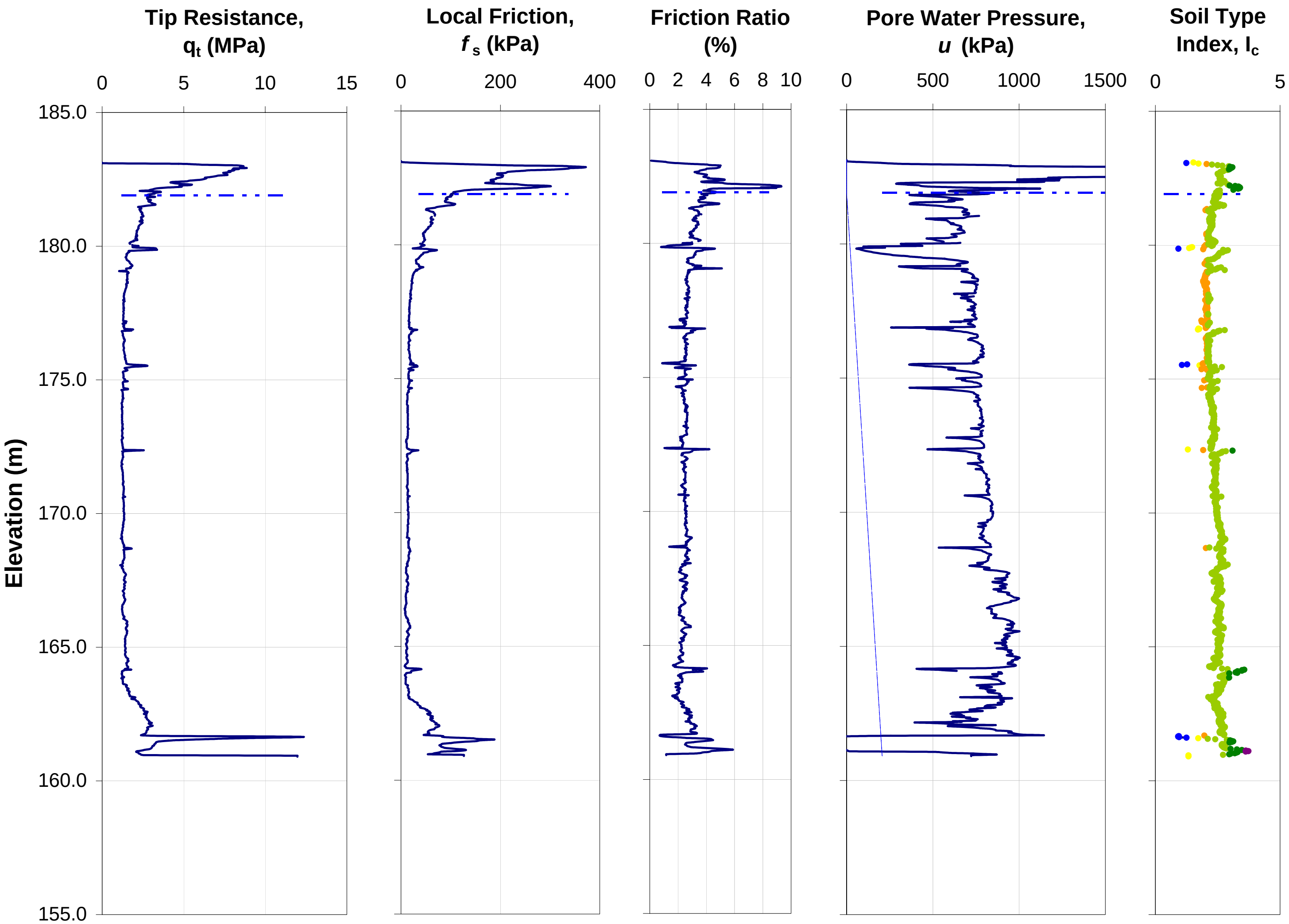
Golder Associates

Project No.: 04-1111-060
Date: 12/11/2006
Check: SJB
Review: FJH

Detroit River International Crossing

Cone Penetration Test

2



Notes:

1. Soil Index Type, Ic, is an inferred soil type and actual conditions may differ.
2. Soil Index Type based on "Guidelines for Geotechnical Design Using CPT and CPTU", PK Robertson and RG Campanella 1989, Robertson and Wride, 1998, and/or site specific data.
3. See report text for limitations of testing and interpretation of data

Legend:

		Ic <	1.30	Gravelly Sand	🟦
1.30	<	Ic <	1.80	Sand to Silty Sand	🟨
1.80	<	Ic <	2.10	Silty Sand to Sandy Silt	🟧
2.10	<	Ic <	2.95	Clayey Silt to Silty Clay	🟩
2.95	<	Ic <	3.60	Clay	🟩
3.60	<	Ic		Clay and Organic Soils	🟪

Static Groundwater Surface — · — · —

Static Groundwater Pressure ··············

Operator Initials:	CC	Easting - X Coordinate (m):	334,957.0
Cone Identification Number:	3742	Northing - Y Coordinate (m):	4,678,022.0
Correction Factor a	0.6	Ground Surface Elevation (m):	185.9
Correction Factor b	0.013	Depth to Groundwater (m):	4.0
		Predrilling Depth (m):	2.8

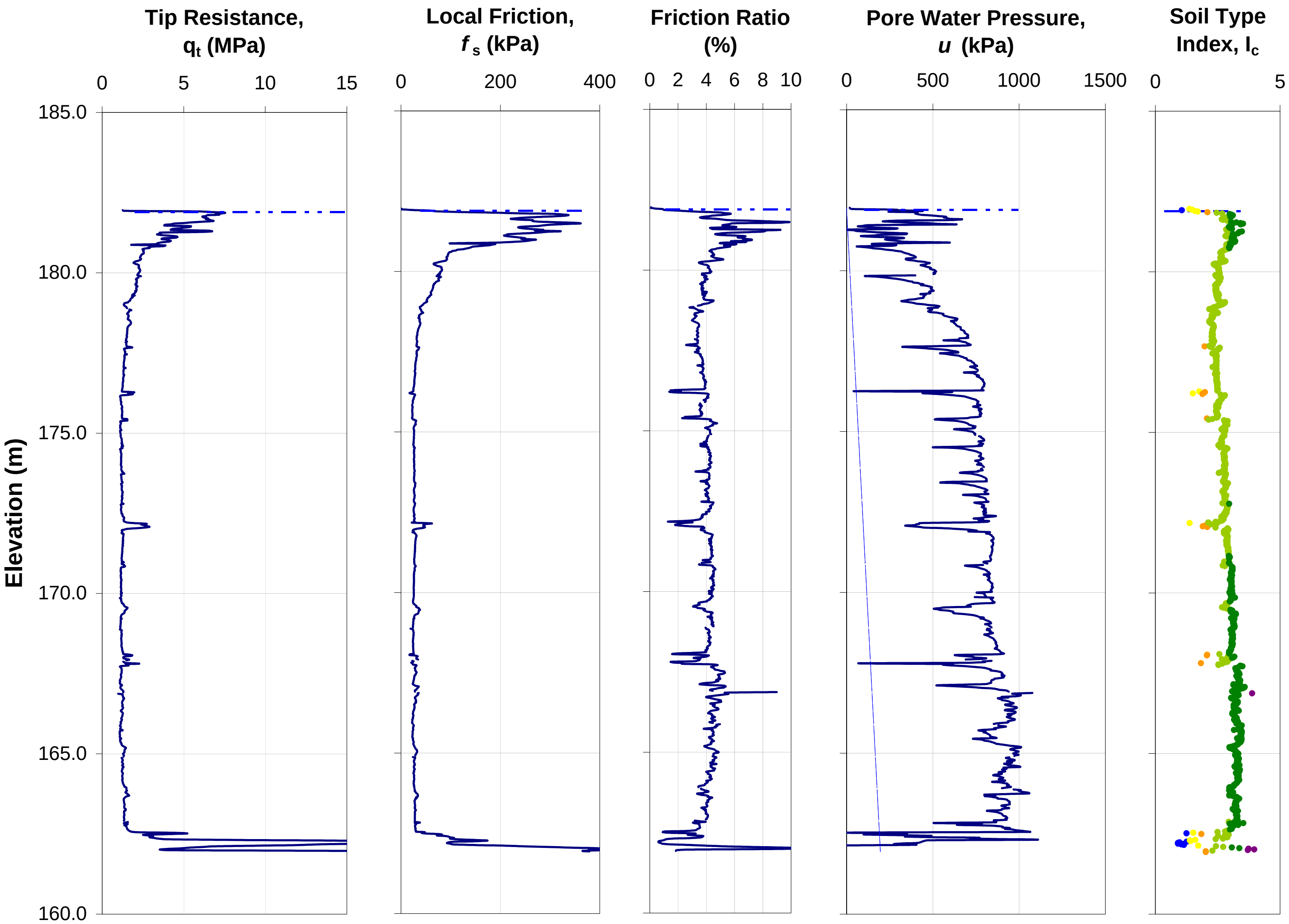
Golder Associates

Detroit River International Crossing

Project No.:	04-1111-060
Date:	13/11/2006
Check:	SJB
Review:	FJH

Cone Penetration Test

3



Notes:

1. Soil Index Type, Ic, is an inferred soil type and actual conditions may differ.
2. Soil Index Type based on "Guidelines for Geotechnical Design Using CPT and CPTU", PK Robertson and RG Campanella 1989, Robertson and Wride, 1998, and/or site specific data.
3. See report text for limitations of testing and interpretation of data

Legend:

	Ic <	1.30	Gravelly Sand	🟦
1.30	< Ic <	1.80	Sand to Silty Sand	🟨
1.80	< Ic <	2.10	Silty Sand to Sandy Silt	🟧
2.10	< Ic <	2.95	Clayey Silt to Silty Clay	🟩
2.95	< Ic <	3.60	Clay	🟩
3.60	< Ic		Clay and Organic Soils	🟪

Static Groundwater Surface -·-·-·-

Static Groundwater Pressure ··········

Operator Initials:	CC	Easting - X Coordinate (m):	334,516.0
Cone Identification Number:	3742	Northing - Y Coordinate (m):	4,678,208.0
Correction Factor a	0.6	Ground Surface Elevation (m):	185.1
Correction Factor b	0.013	Depth to Groundwater (m):	3.2
		Predrilling Depth (m):	3.1

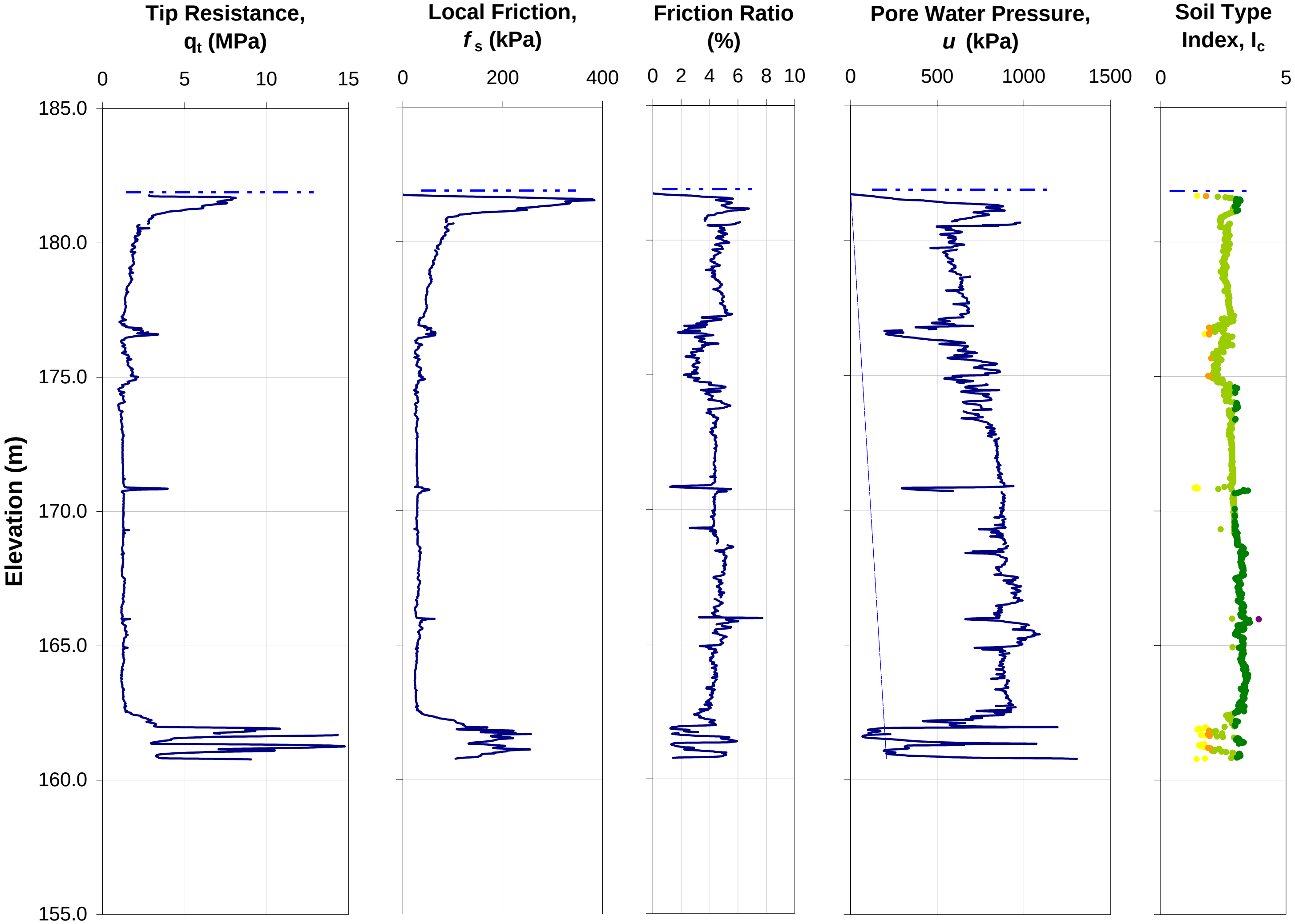
Golder Associates

Detroit River International Crossing

Project No.:	04-1111-060
Date:	14/11/2006
Check:	SJB
Review:	FJH

Cone Penetration Test

4



Notes:

1. Soil Index Type, Ic, is an inferred soil type and actual conditions may differ.
2. Soil Index Type based on "Guidelines for Geotechnical Design Using CPT and CPTU", PK Robertson and RG Campanella 1989, Robertson and Wride, 1998, and/or site specific data.
3. See report text for limitations of testing and interpretation of data

Legend:

Ic <	1.30 Gravelly Sand	🟦
1.30 < Ic <	1.80 Sand to Silty Sand	🟨
1.80 < Ic <	2.10 Silty Sand to Sandy Silt	🟧
2.10 < Ic <	2.95 Clayey Silt to Silty Clay	🟩
2.95 < Ic <	3.60 Clay	🟩
3.60 < Ic	Clay and Organic Soils	🟪

Static Groundwater Surface — · · — · · —

Static Groundwater Pressure ·························

Operator Initials:	CC	Easting - X Coordinate (m):	334,220.0
Cone Identification Number:	3806	Northing - Y Coordinate (m):	4,678,413.0
Correction Factor a	0.584	Ground Surface Elevation (m):	184.7
Correction Factor b	0.012	Depth to Groundwater (m):	2.8
		Predrilling Depth (m):	2.9

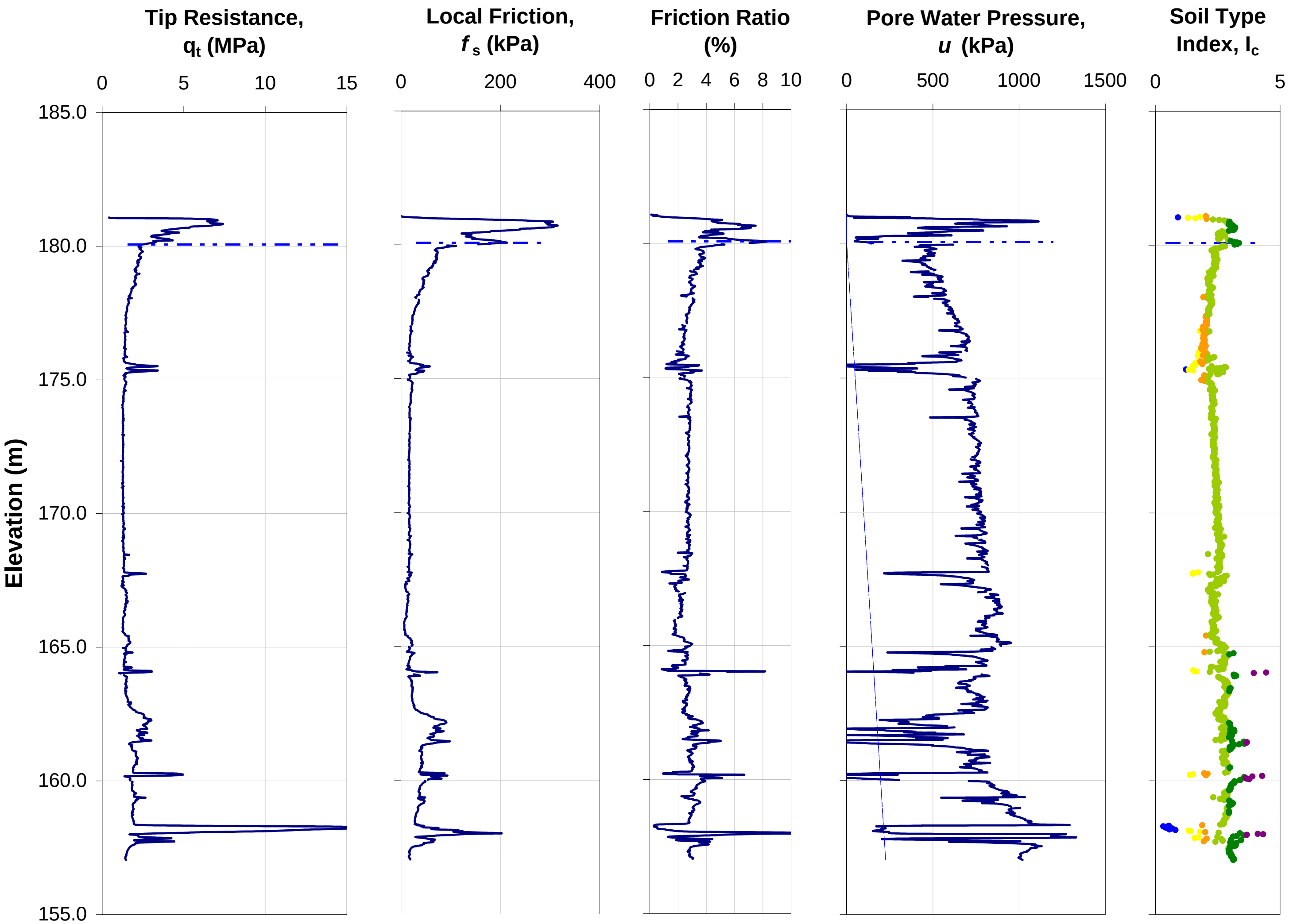
Golder Associates

Project No.:	04-1111-060
Date:	13/11/2006
Check:	SJB
Review:	FJH

Detroit River International Crossing

Cone Penetration Test

5



Notes:

1. Soil Index Type, Ic, is an inferred soil type and actual conditions may differ.
2. Soil Index Type based on "Guidelines for Geotechnical Design Using CPT and CPTU", PK Robertson and RG Campanella 1989, Robertson and Wride, 1998, and/or site specific data.
3. See report text for limitations of testing and interpretation of data

Legend:

	Ic <	1.30	Gravelly Sand	🟦
1.30	< Ic <	1.80	Sand to Silty Sand	🟨
1.80	< Ic <	2.10	Silty Sand to Sandy Silt	🟧
2.10	< Ic <	2.95	Clayey Silt to Silty Clay	🟩
2.95	< Ic <	3.60	Clay	🟩
3.60	< Ic		Clay and Organic Soils	🟪

Static Groundwater Surface —·—··—·—··—

Static Groundwater Pressure ·····················

Operator Initials: CC

Cone Identification Number: 3742

Correction Factor a 0.6

Correction Factor b 0.013

Easting - X Coordinate (m): 333,844.0

Northing - Y Coordinate (m): 4,678,621.0

Ground Surface Elevation (m): 184.1

Depth to Groundwater (m): 4.0

Predrilling Depth (m): 3.0

Golder Associates

Detroit River International Crossing

Project No.: 04-1111-060

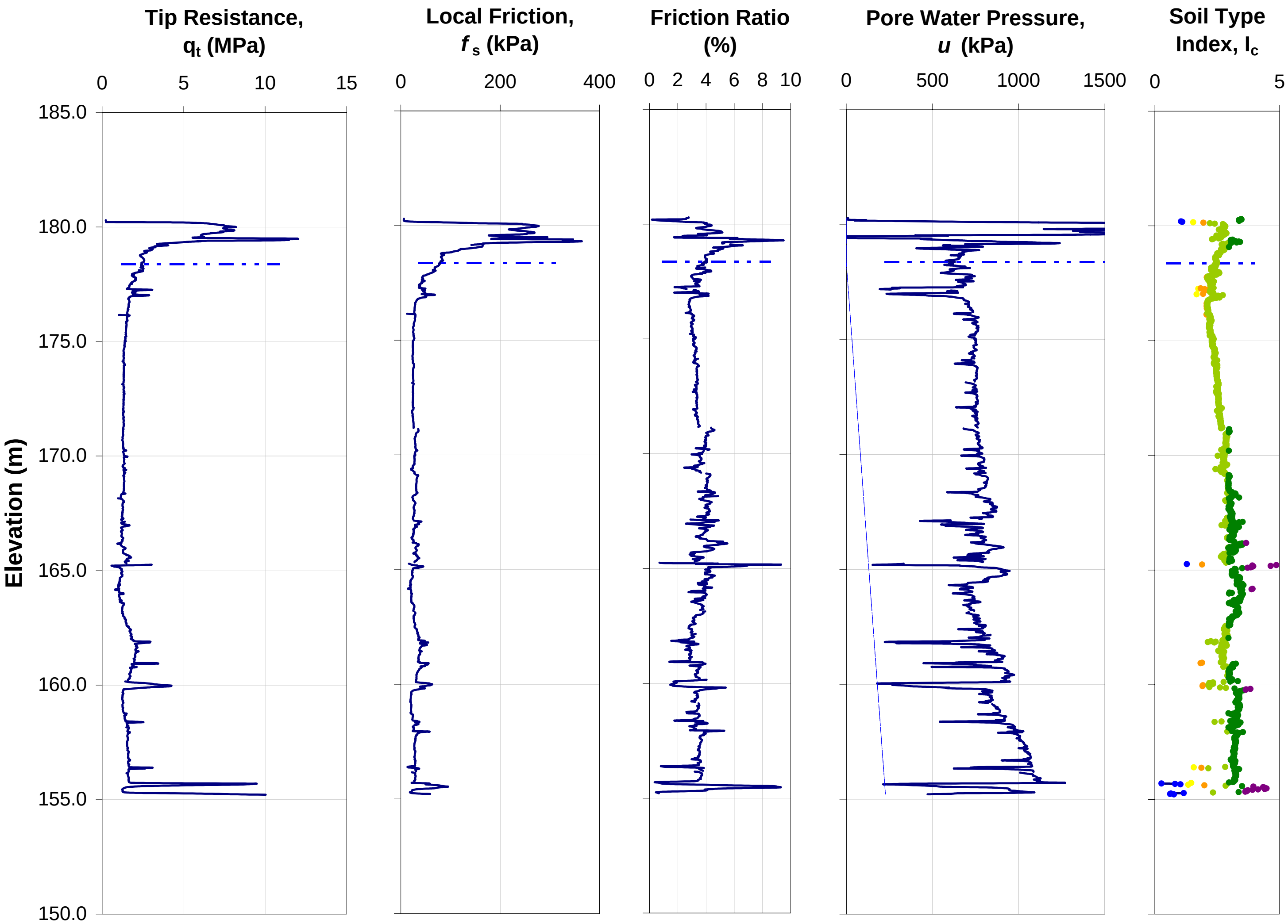
Date: 13/11/2006

Check: SJB

Review: FJH

Cone Penetration Test

6



Notes:

1. Soil Index Type, Ic, is an inferred soil type and actual conditions may differ.
2. Soil Index Type based on "Guidelines for Geotechnical Design Using CPT and CPTU", PK Robertson and RG Campanella 1989, Robertson and Wride, 1998, and/or site specific data.
3. See report text for limitations of testing and interpretation of data

Legend:

	Ic <	1.30	Gravelly Sand	🟦
1.30	< Ic <	1.80	Sand to Silty Sand	🟨
1.80	< Ic <	2.10	Silty Sand to Sandy Silt	🟧
2.10	< Ic <	2.95	Clayey Silt to Silty Clay	🟩
2.95	< Ic <	3.60	Clay	🟩
3.60	< Ic		Clay and Organic Soils	🟪

Static Groundwater Surface -·-·-·-

Static Groundwater Pressure ··········

Operator Initials:	CC	Easting - X Coordinate (m):	333,327.0
Cone Identification Number:	3806	Northing - Y Coordinate (m):	4,678,844.0
Correction Factor a	0.584	Ground Surface Elevation (m):	183.2
Correction Factor b	0.012	Depth to Groundwater (m):	4.8
		Predrilling Depth (m):	2.9

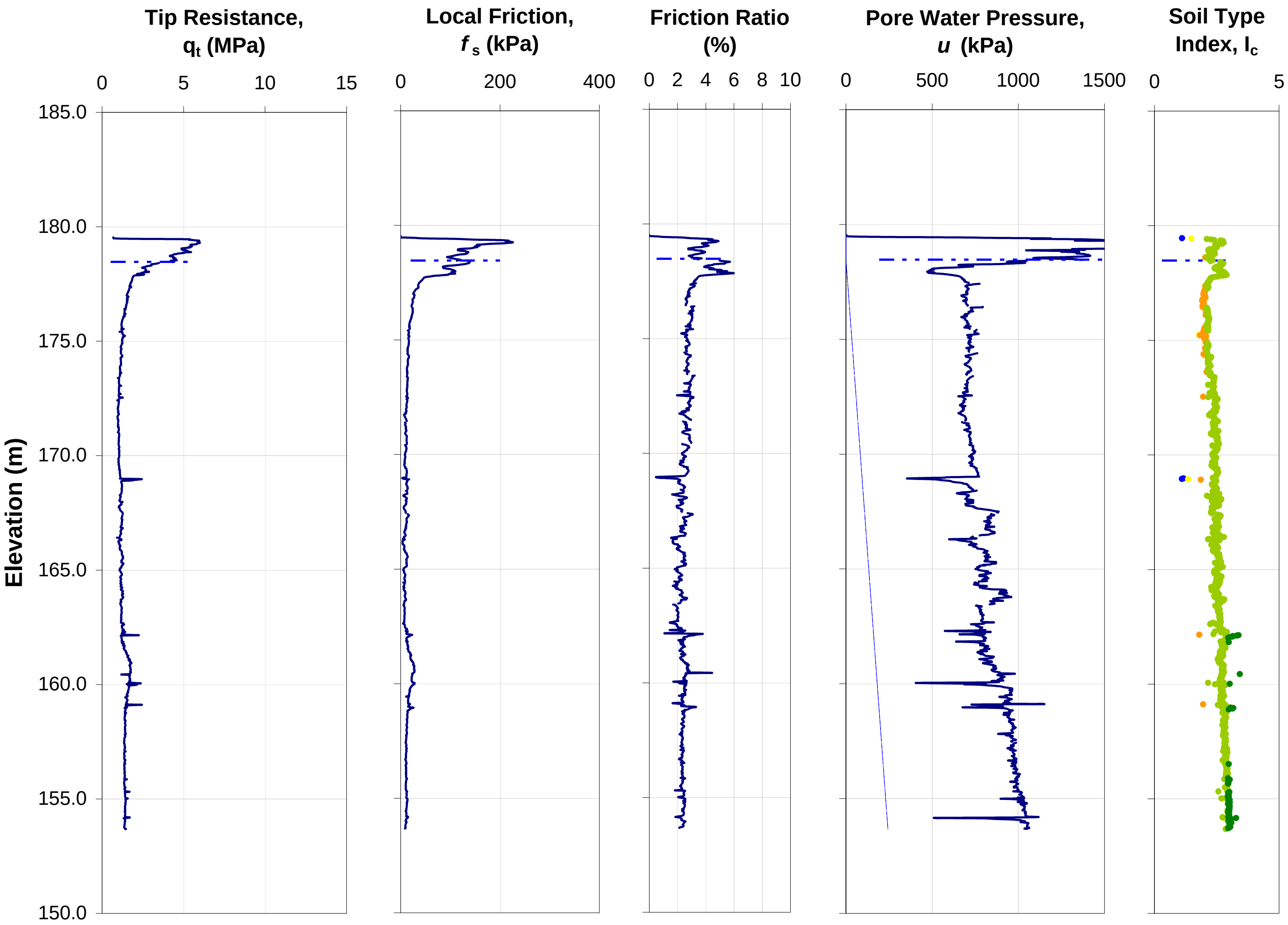
Golder Associates

Project No.:	04-1111-060
Date:	12/11/2006
Check:	SJB
Review:	FJH

Detroit River International Crossing

Cone Penetration Test

7



Notes:

1. Soil Index Type, Ic, is an inferred soil type and actual conditions may differ.
2. Soil Index Type based on "Guidelines for Geotechnical Design Using CPT and CPTU", PK Robertson and RG Campanella 1989, Robertson and Wride, 1998, and/or site specific data.
3. See report text for limitations of testing and interpretation of data

Legend:

	Ic <	1.30	Gravelly Sand	🟦
1.30	< Ic <	1.80	Sand to Silty Sand	🟨
1.80	< Ic <	2.10	Silty Sand to Sandy Silt	🟧
2.10	< Ic <	2.95	Clayey Silt to Silty Clay	🟩
2.95	< Ic <	3.60	Clay	🟩
3.60	< Ic		Clay and Organic Soils	🟪

Static Groundwater Surface — · · — · · —

Static Groundwater Pressure ··························

Operator Initials: CC

Cone Identification Number: 3806

Correction Factor a 0.584

Correction Factor b 0.012

Easting - X Coordinate (m): 333,109.0

Northing - Y Coordinate (m): 4,678,967.0

Ground Surface Elevation (m): 182.5

Depth to Groundwater (m): 4.0

Predrilling Depth (m): 3.0

Golder Associates

Project No.: 04-1111-060

Date: 11/11/2006

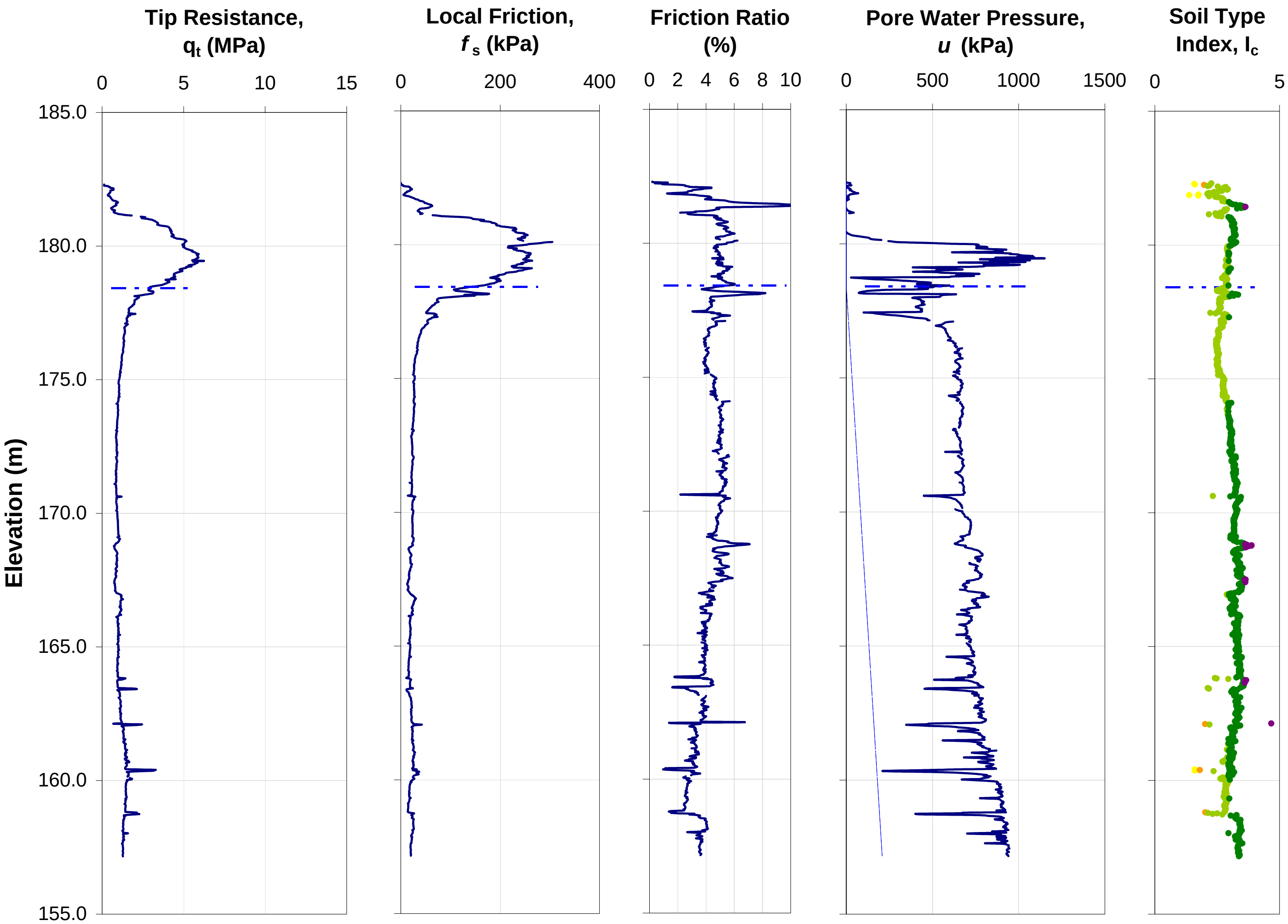
Check: SJB

Review: FJH

Detroit River International Crossing

Cone Penetration Test

8



Notes:

1. Soil Index Type, Ic, is an inferred soil type and actual conditions may differ.
2. Soil Index Type based on "Guidelines for Geotechnical Design Using CPT and CPTU", PK Robertson and RG Campanella 1989, Robertson and Wride, 1998, and/or site specific data.
3. See report text for limitations of testing and interpretation of data

Legend:

	Ic <	1.30	Gravelly Sand	🟦
1.30	< Ic <	1.80	Sand to Silty Sand	🟨
1.80	< Ic <	2.10	Silty Sand to Sandy Silt	🟧
2.10	< Ic <	2.95	Clayey Silt to Silty Clay	🟩
2.95	< Ic <	3.60	Clay	🟩
3.60	< Ic		Clay and Organic Soils	🟪

Static Groundwater Surface — · — · — · —

Static Groundwater Pressure ··························

Operator Initials:	CC	Easting - X Coordinate (m):	332,828.0
Cone Identification Number:	3742	Northing - Y Coordinate (m):	4,679,105.0
Correction Factor a	0.6	Ground Surface Elevation (m):	182.3
Correction Factor b	0.013	Depth to Groundwater (m):	3.9
		Predrilling Depth (m):	0.0

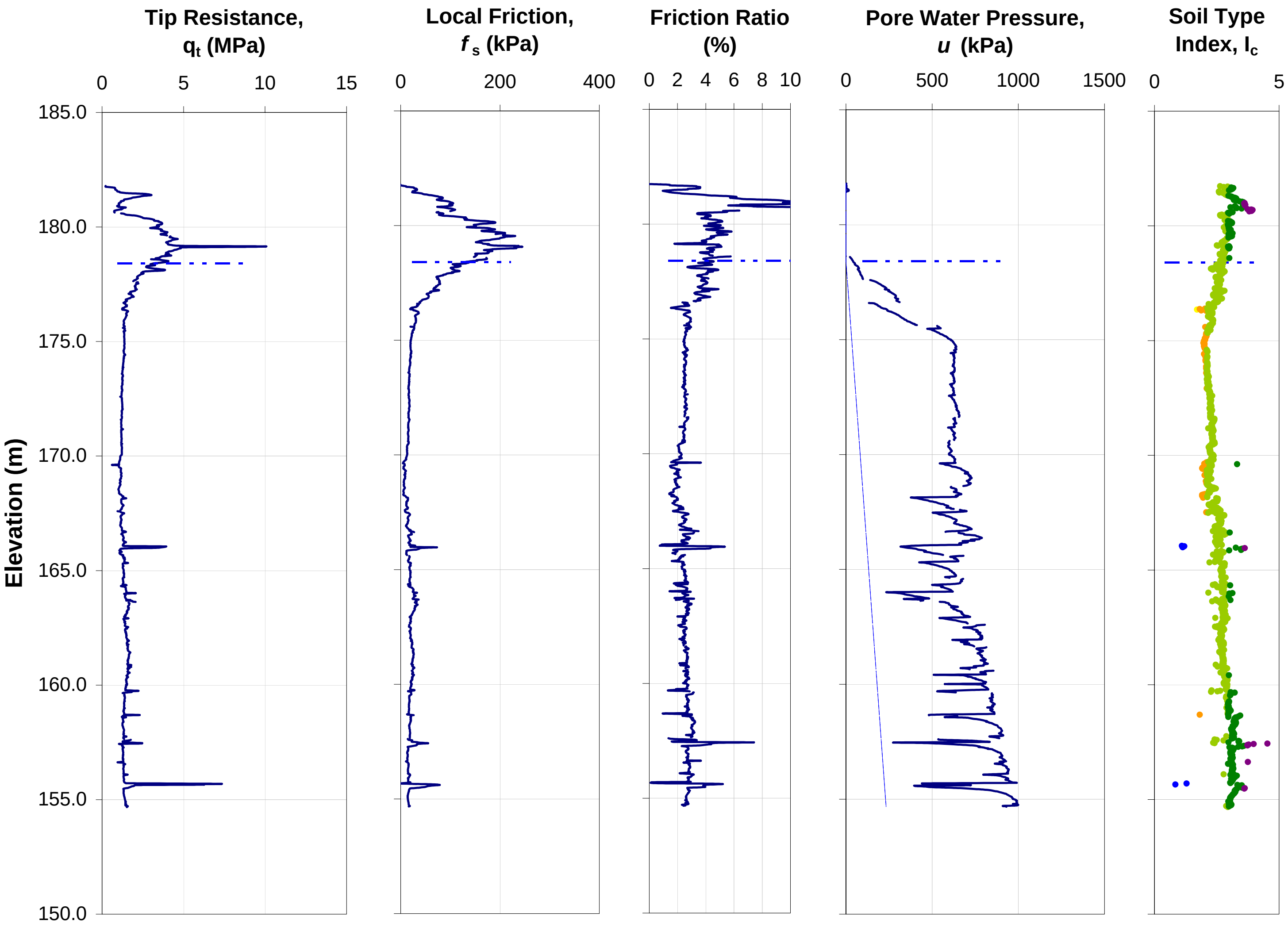
Golder Associates

Project No.:	04-1111-060
Date:	11/11/2006
Check:	SJB
Review:	FJH

Detroit River International Crossing

Cone Penetration Test

9



Notes:

1. Soil Index Type, Ic, is an inferred soil type and actual conditions may differ.
2. Soil Index Type based on "Guidelines for Geotechnical Design Using CPT and CPTU", PK Robertson and RG Campanella 1989, Robertson and Wride, 1998, and/or site specific data.
3. See report text for limitations of testing and interpretation of data

Legend:

Ic <	1.30	Gravelly Sand	🟦
1.30 < Ic <	1.80	Sand to Silty Sand	🟨
1.80 < Ic <	2.10	Silty Sand to Sandy Silt	🟧
2.10 < Ic <	2.95	Clayey Silt to Silty Clay	🟩
2.95 < Ic <	3.60	Clay	🟩
3.60 < Ic		Clay and Organic Soils	🟪

Static Groundwater Surface — · — · —

Static Groundwater Pressure ··········

Operator Initials:	CC	Easting - X Coordinate (m):	332,533.0
Cone Identification Number:	3806	Northing - Y Coordinate (m):	4,679,264.0
Correction Factor a	0.584	Ground Surface Elevation (m):	181.8
Correction Factor b	0.012	Depth to Groundwater (m):	3.4
		Predrilling Depth (m):	0.0

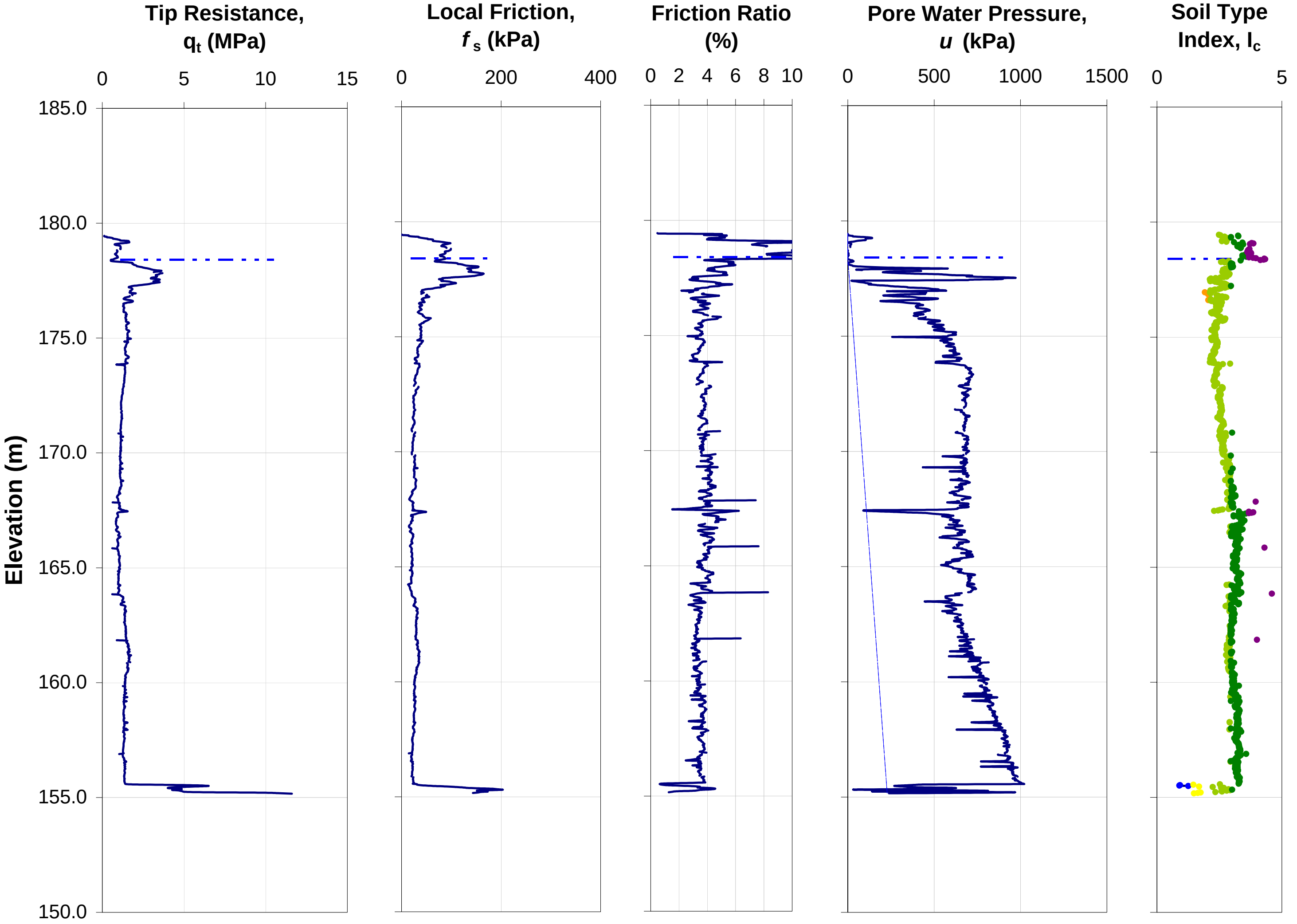
Golder Associates

Project No.:	04-1111-060
Date:	10/11/2006
Check:	SJB
Review:	FJH

Detroit River International Crossing

Cone Penetration Test

10



Notes:

1. Soil Index Type, Ic, is an inferred soil type and actual conditions may differ.
2. Soil Index Type based on "Guidelines for Geotechnical Design Using CPT and CPTU", PK Robertson and RG Campanella 1989, Robertson and Wride, 1998, and/or site specific data.
3. See report text for limitations of testing and interpretation of data

Legend:

Ic <	1.30	Gravelly Sand	🟦
1.30 < Ic <	1.80	Sand to Silty Sand	🟨
1.80 < Ic <	2.10	Silty Sand to Sandy Silt	🟧
2.10 < Ic <	2.95	Clayey Silt to Silty Clay	🟩
2.95 < Ic <	3.60	Clay	🟩
3.60 < Ic		Clay and Organic Soils	🟪

Static Groundwater Surface —·—·—·—

Static Groundwater Pressure ··············

Operator Initials: CC

Cone Identification Number: 3742

Correction Factor a 0.6

Correction Factor b 0.013

Easting - X Coordinate (m): 332,110.0

Northing - Y Coordinate (m): 4,679,634.0

Ground Surface Elevation (m): 180.9

Depth to Groundwater (m): 2.5

Predrilling Depth (m): 1.5

Golder Associates

Project No.: 04-1111-060

Date: 10/11/2006

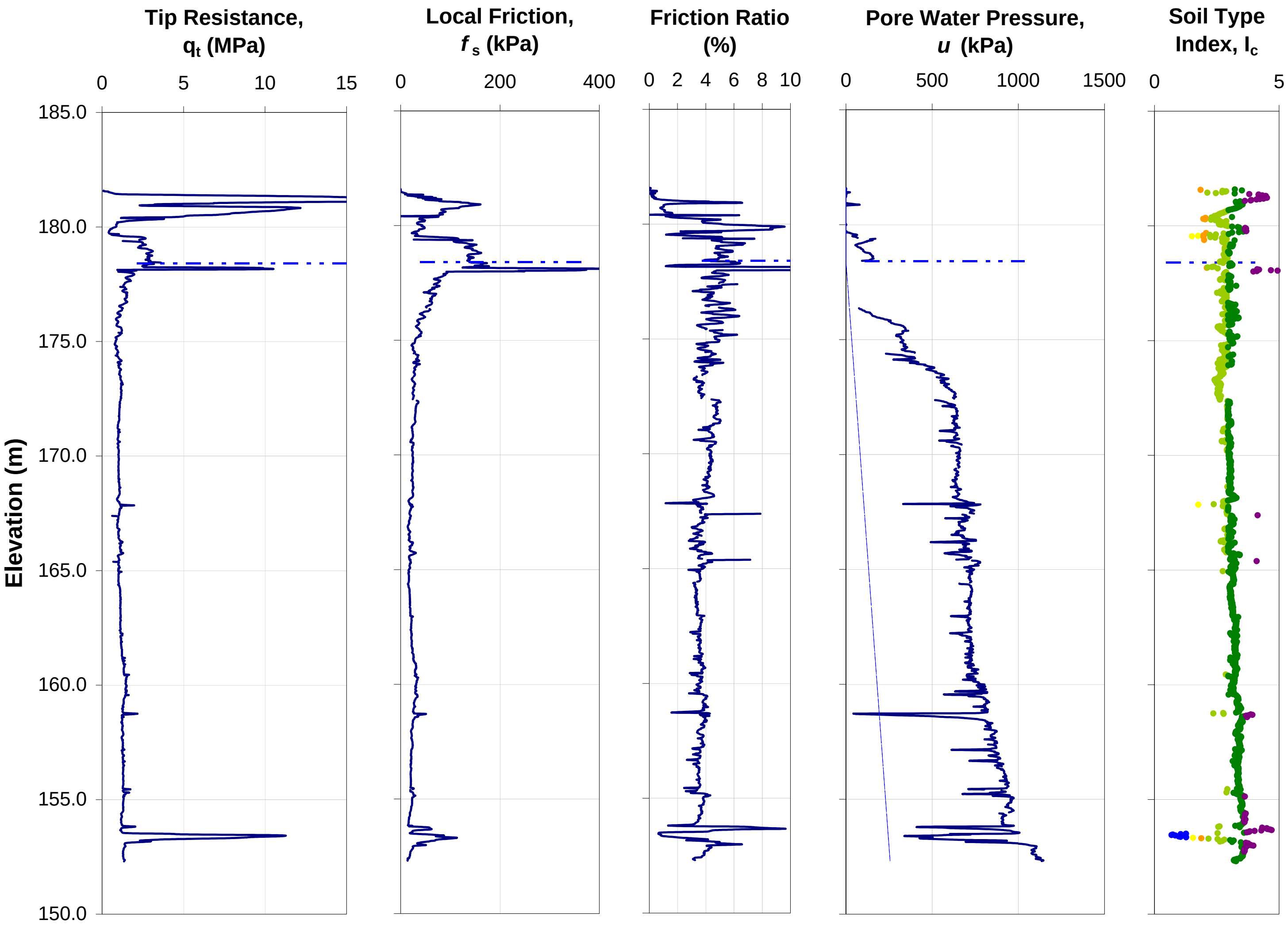
Check: SJB

Review: FJH

Detroit River International Crossing

Cone Penetration Test

11



Notes:

1. Soil Index Type, Ic, is an inferred soil type and actual conditions may differ.
2. Soil Index Type based on "Guidelines for Geotechnical Design Using CPT and CPTU", PK Robertson and RG Campanella 1989, Robertson and Wride, 1998, and/or site specific data.
3. See report text for limitations of testing and interpretation of data

Legend:

Ic <	1.30 Gravelly Sand	🟦
1.30 < Ic <	1.80 Sand to Silty Sand	🟨
1.80 < Ic <	2.10 Silty Sand to Sandy Silt	🟧
2.10 < Ic <	2.95 Clayey Silt to Silty Clay	🟩
2.95 < Ic <	3.60 Clay	🟩
3.60 < Ic	Clay and Organic Soils	🟪

Static Groundwater Surface —·—··—·—

Static Groundwater Pressure ··········

Operator Initials:	CC	Easting - X Coordinate (m):	331,924.0
Cone Identification Number:	3742	Northing - Y Coordinate (m):	4,680,072.0
Correction Factor a	0.6	Ground Surface Elevation (m):	181.6
Correction Factor b	0.013	Depth to Groundwater (m):	3.2
		Predrilling Depth (m):	0.0

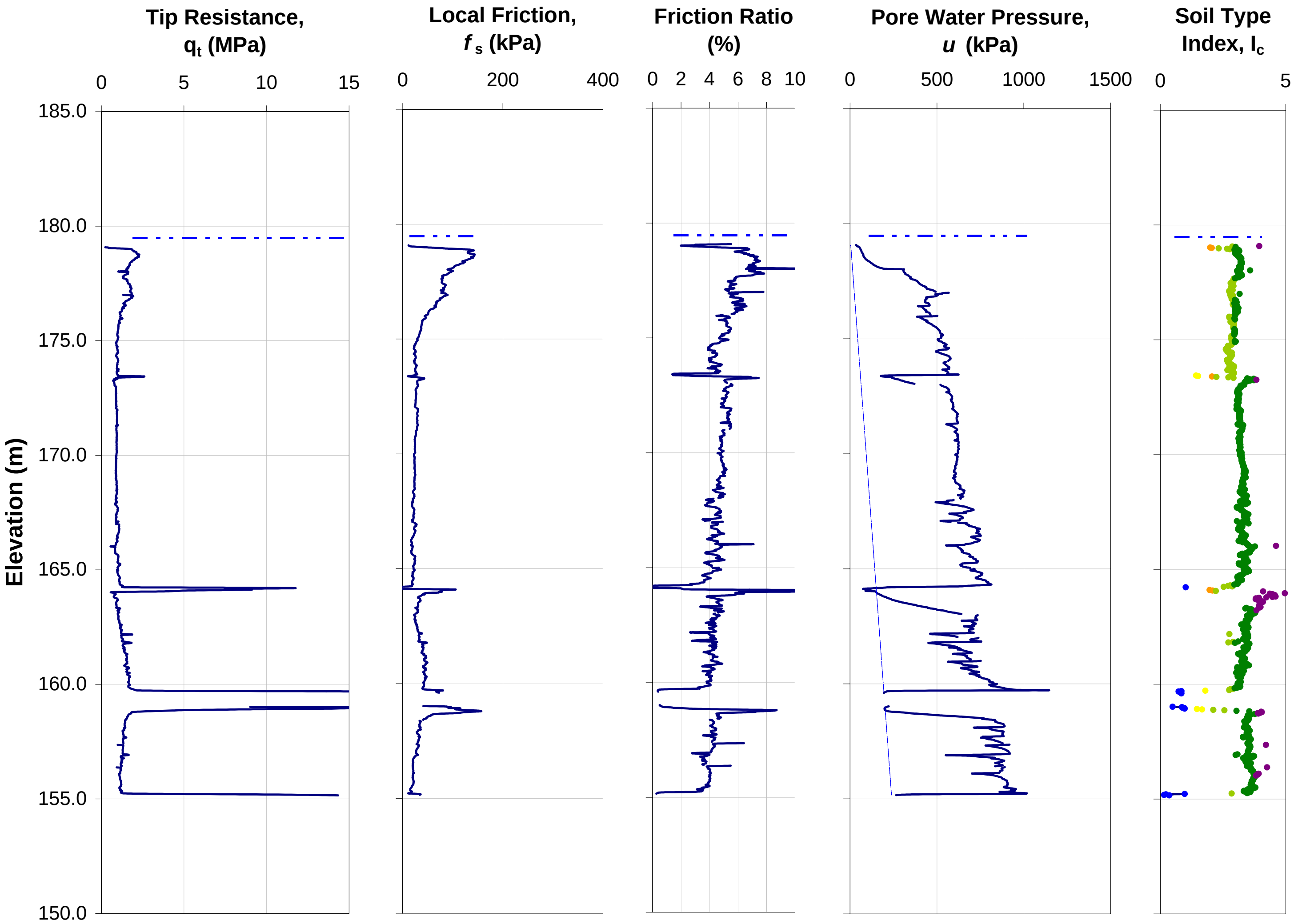
Golder Associates

Project No.:	04-1111-060
Date:	10/11/2006
Check:	SJB
Review:	FJH

Detroit River International Crossing

Cone Penetration Test

12



Notes:

1. Soil Index Type, Ic, is an inferred soil type and actual conditions may differ.
2. Soil Index Type based on "Guidelines for Geotechnical Design Using CPT and CPTU", PK Robertson and RG Campanella 1989, Robertson and Wride, 1998, and/or site specific data.
3. See report text for limitations of testing and interpretation of data

Legend:

	Ic <	1.30 Gravelly Sand	🟦
1.30	< Ic <	1.80 Sand to Silty Sand	🟨
1.80	< Ic <	2.10 Silty Sand to Sandy Silt	🟧
2.10	< Ic <	2.95 Clayey Silt to Silty Clay	🟩
2.95	< Ic <	3.80 Clay	🟩
3.80	< Ic	Clay and Organics	🟪

Static Groundwater Surface — · · — · · —

Static Groundwater Pressure ··················

Operator Initials:	CC	Easting - X Coordinate (m):	331,749.0
Cone Identification Number:	3806	Northing - Y Coordinate (m):	4,680,350.0
Correction Factor a	0.584	Ground Surface Elevation (m):	182.1
Correction Factor b	0.012	Depth to Groundwater (m):	2.6
		Predrilling Depth (m):	3.0

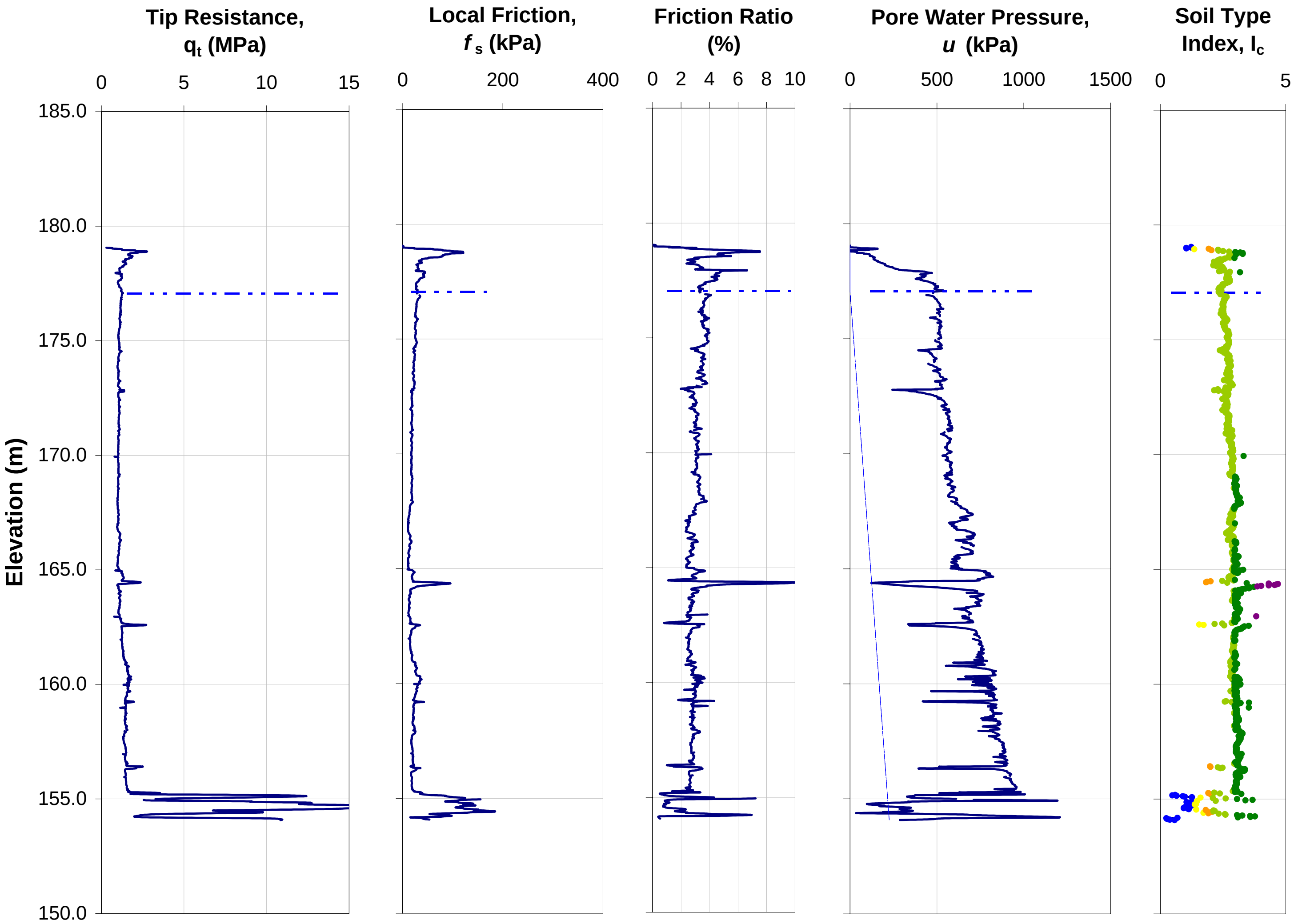
Golder Associates

Project No.:	04-1111-060
Date:	08/11/2006
Check:	SJB
Review:	FJH

Detroit River International Crossing

Cone Penetration Test

13



Notes:

1. Soil Index Type, Ic, is an inferred soil type and actual conditions may differ.
2. Soil Index Type based on "Guidelines for Geotechnical Design Using CPT and CPTU", PK Robertson and RG Campanella 1989, Robertson and Wride, 1998, and/or site specific data.
3. See report text for limitations of testing and interpretation of data

Legend:

	Ic <	1.30	Gravelly Sand	🟦
1.30	< Ic <	1.80	Sand to Silty Sand	🟨
1.80	< Ic <	2.10	Silty Sand to Sandy Silt	🟧
2.10	< Ic <	2.95	Clayey Silt to Silty Clay	🟩
2.95	< Ic <	3.80	Clay	🟩
3.80	< Ic		Clay and Organics	🟪

Static Groundwater Surface — · · — · · —

Static Groundwater Pressure ··········

Operator Initials:	CC	Easting - X Coordinate (m):	335,500.0
Cone Identification Number:	3806	Northing - Y Coordinate (m):	4,677,738.0
Correction Factor a	0.584	Ground Surface Elevation (m):	182.1
Correction Factor b	0.012	Depth to Groundwater (m):	5.0
		Predrilling Depth (m):	3.0

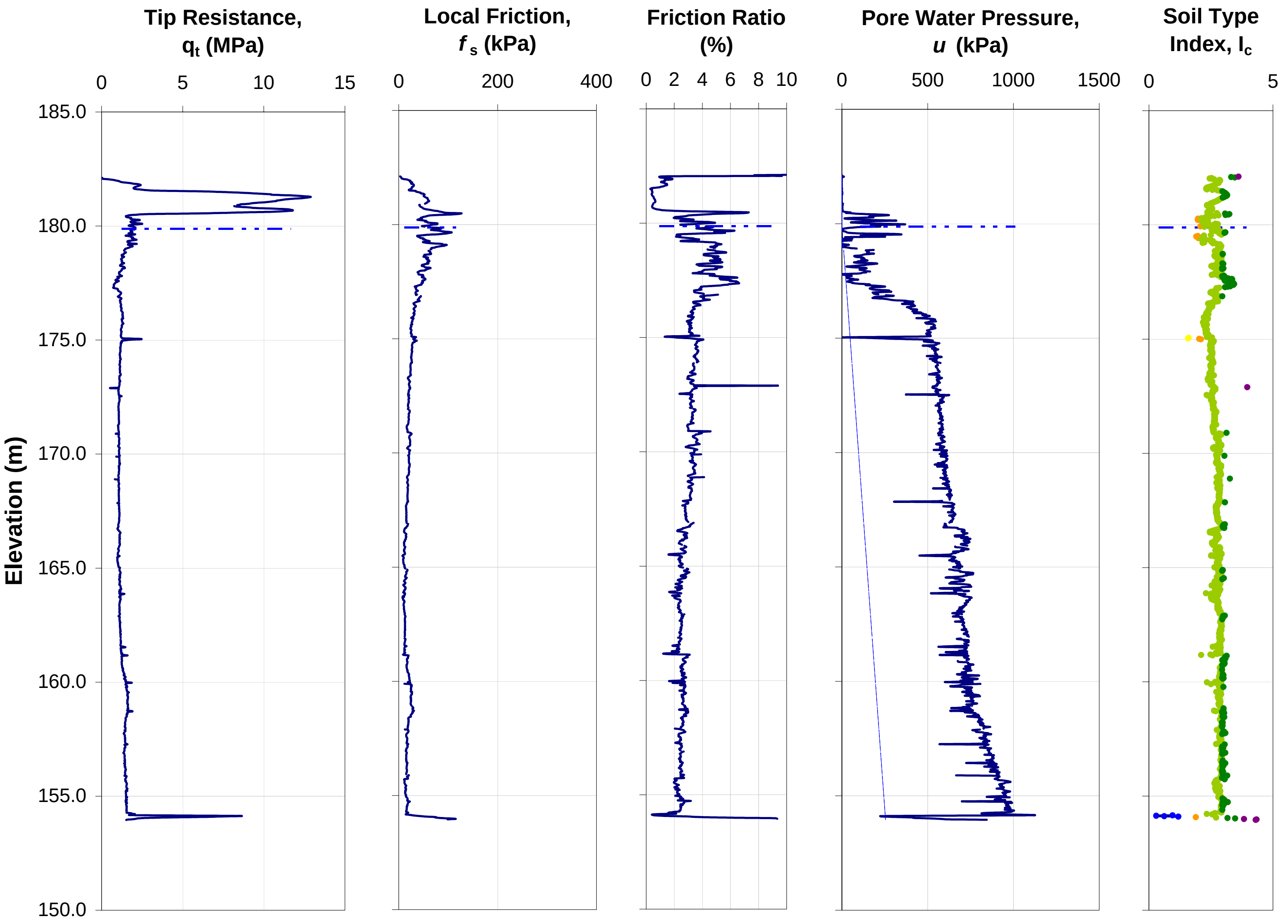
Golder Associates

Project No.:	04-1111-060
Date:	07/11/2006
Check:	SJB
Review:	FJH

Detroit River International Crossing

Cone Penetration Test

14



Notes:

1. Soil Index Type, Ic, is an inferred soil type and actual conditions may differ.
2. Soil Index Type based on "Guidelines for Geotechnical Design Using CPT and CPTU", PK Robertson and RG Campanella 1989, Robertson and Wride, 1998, and/or site specific data.
3. See report text for limitations of testing and interpretation of data

Legend:

		Ic <	1.30	Gravelly Sand		🟦
1.30	<	Ic <	1.80	Sand to Silty Sand		🟨
1.80	<	Ic <	2.10	Silty Sand to Sandy Silt		🟧
2.10	<	Ic <	2.95	Clayey Silt to Silty Clay		🟩
2.95	<	Ic <	3.60	Clay		🟩
3.60	<	Ic		Clay and Organic Soils		🟪

Static Groundwater Surface —·—··—·—

Static Groundwater Pressure ··········

Operator Initials:	CC	Easting - X Coordinate (m):	331,480.0
Cone Identification Number:	3806	Northing - Y Coordinate (m):	4,681,049.0
Correction Factor a	0.584	Ground Surface Elevation (m):	182.1
Correction Factor b	0.012	Depth to Groundwater (m):	2.3
		Predrilling Depth (m):	0.0

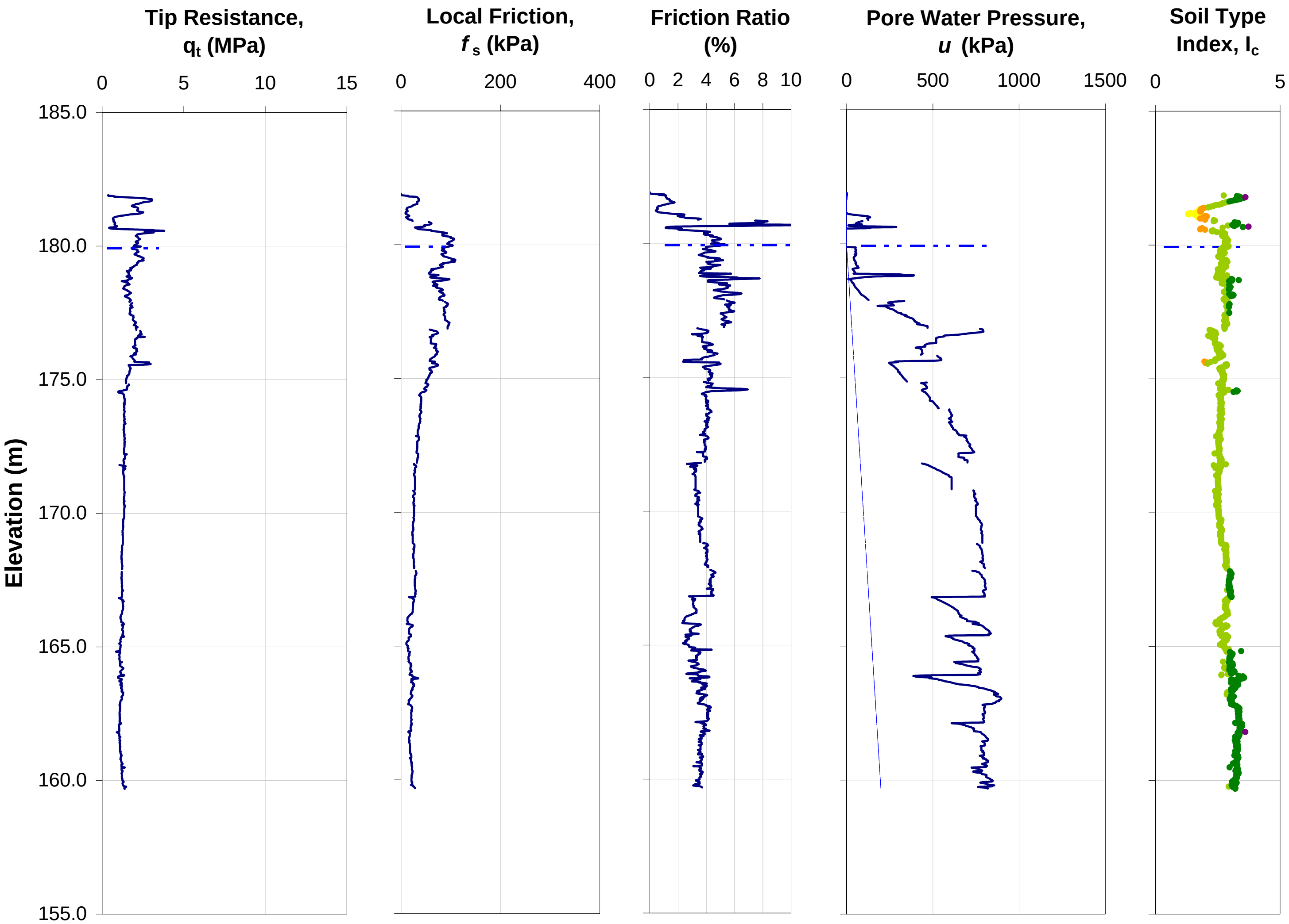
Golder Associates

Project No.:	04-1111-060
Date:	09/11/2006
Check:	SJB
Review:	FJH

Detroit River International Crossing

Cone Penetration Test

15



Notes:

1. Soil Index Type, Ic, is an inferred soil type and actual conditions may differ.
2. Soil Index Type based on "Guidelines for Geotechnical Design Using CPT and CPTU", PK Robertson and RG Campanella 1989, Robertson and Wride, 1998, and/or site specific data.
3. See report text for limitations of testing and interpretation of data

Legend:

	Ic <	1.30	Gravelly Sand	🟦
1.30	< Ic <	1.80	Sand to Silty Sand	🟨
1.80	< Ic <	2.10	Silty Sand to Sandy Silt	🟧
2.10	< Ic <	2.95	Clayey Silt to Silty Clay	🟩
2.95	< Ic <	3.60	Clay	🟩
3.60	< Ic		Clay and Organic Soils	🟪

Static Groundwater Surface —·—··—·—

Static Groundwater Pressure ·····················

Operator Initials:	CC	Easting - X Coordinate (m):	331,376.0
Cone Identification Number:	3742	Northing - Y Coordinate (m):	4,681,417.0
Correction Factor a	0.6	Ground Surface Elevation (m):	181.9
Correction Factor b	0.013	Depth to Groundwater (m):	2.0
		Predrilling Depth (m):	0.0

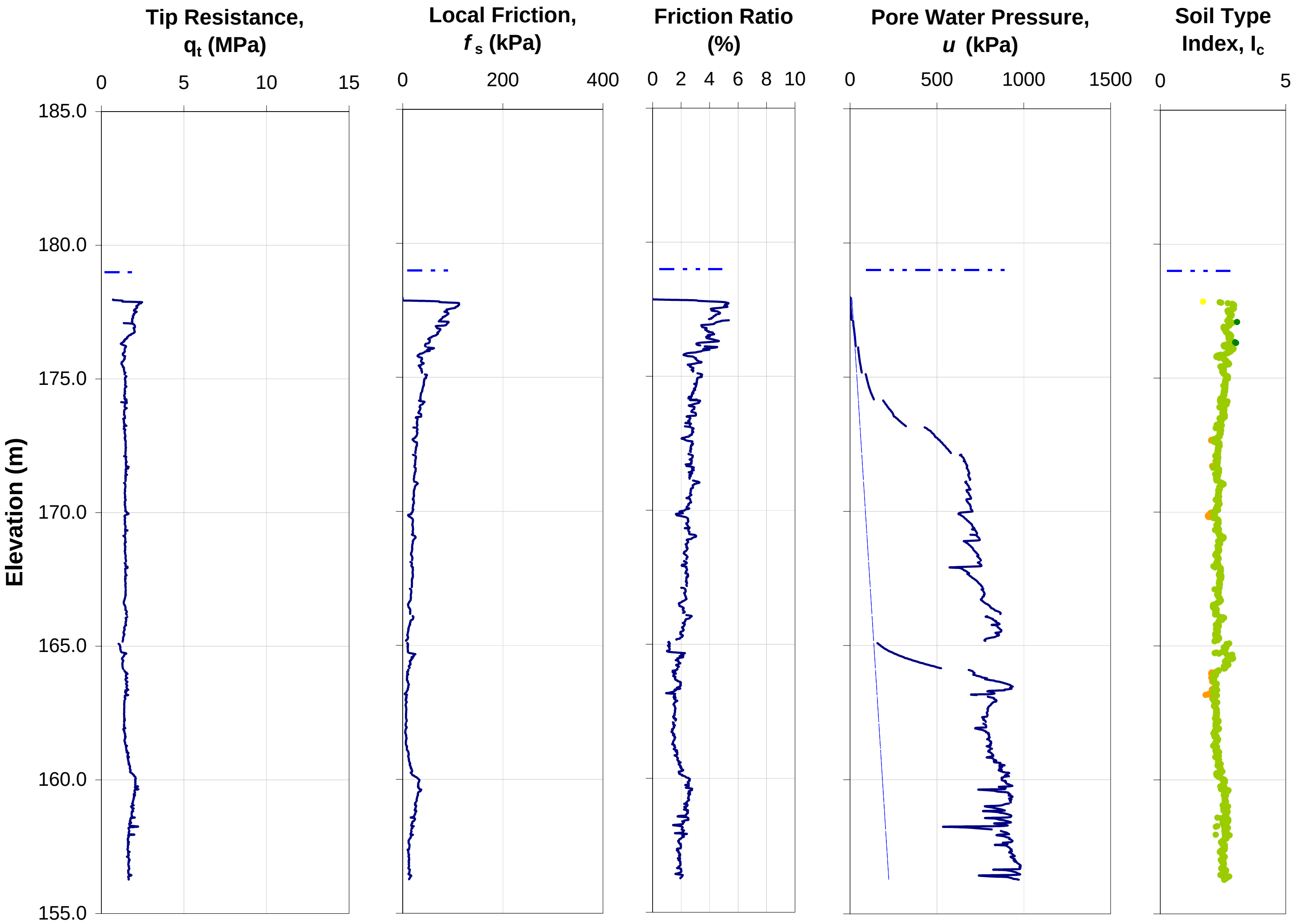
Golder Associates

Project No.: 04-1111-060
Date: 03/11/2006
Check: SJB
Review: FJH

Detroit River International Crossing

Cone Penetration Test

16



Notes:

1. Soil Index Type, Ic, is an inferred soil type and actual conditions may differ.
2. Soil Index Type based on "Guidelines for Geotechnical Design Using CPT and CPTU", PK Robertson and RG Campanella 1989, Robertson and Wride, 1998, and/or site specific data.
3. See report text for limitations of testing and interpretation of data

Legend:

Ic <	1.30	Gravelly Sand	🟦
1.30 < Ic <	1.80	Sand to Silty Sand	🟨
1.80 < Ic <	2.10	Silty Sand to Sandy Silt	🟧
2.10 < Ic <	2.95	Clayey Silt to Silty Clay	🟩
2.95 < Ic <	3.80	Clay	🟩
3.80 < Ic		Clay and Organics	🟪

Static Groundwater Surface —·—··—··—

Static Groundwater Pressure ·························

Operator Initials:	CC	Easting - X Coordinate (m):	331,208.0
Cone Identification Number:	3806	Northing - Y Coordinate (m):	4,681,625.0
Correction Factor a	0.584	Ground Surface Elevation (m):	182.1
Correction Factor b	0.012	Depth to Groundwater (m):	3.1
		Predrilling Depth (m):	4.1

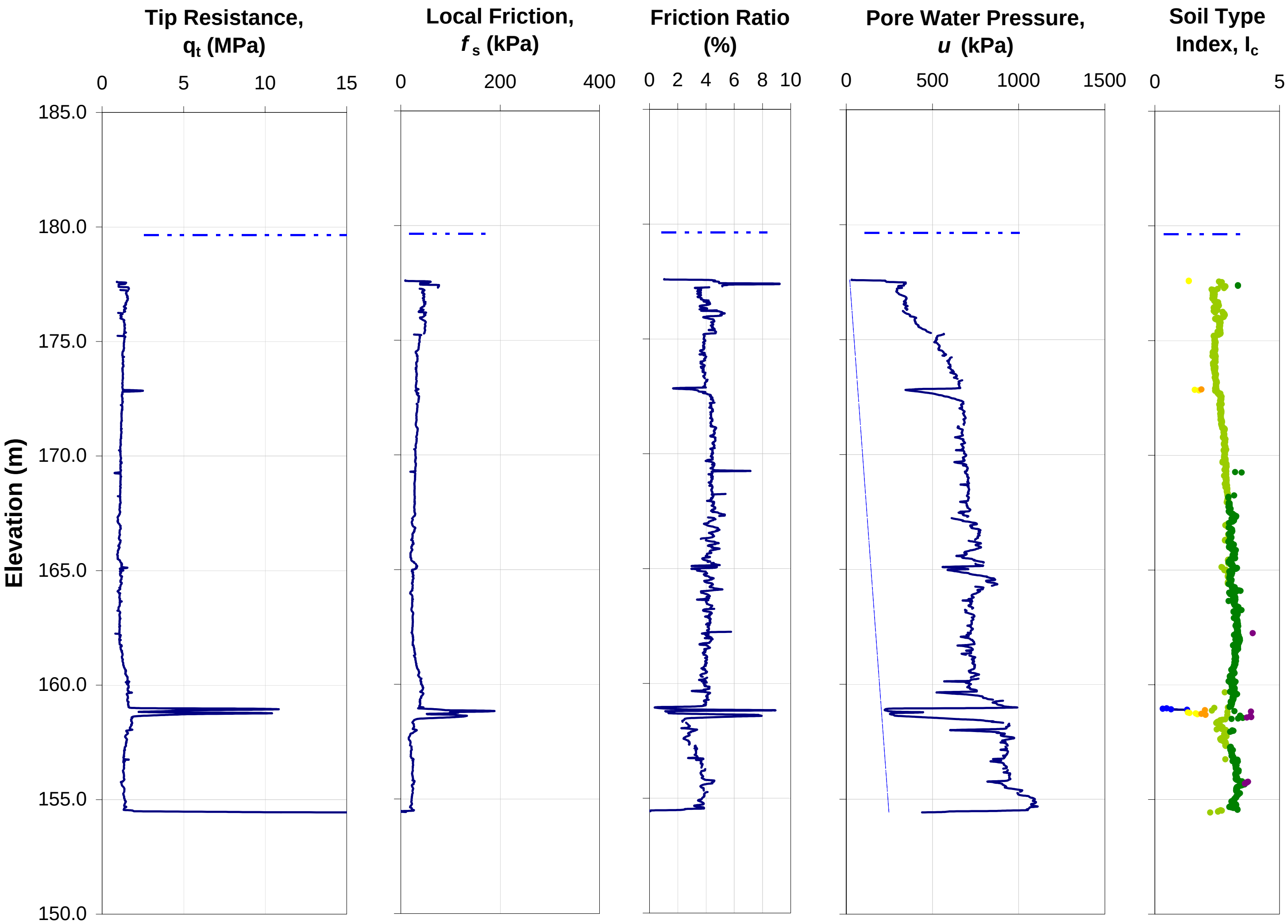
Golder Associates

Project No.: 04-1111-060
Date: 02/11/2006
Check: SJB
Review: FJH

Detroit River International Crossing

Cone Penetration Test

17



Notes:

1. Soil Index Type, Ic, is an inferred soil type and actual conditions may differ.
2. Soil Index Type based on "Guidelines for Geotechnical Design Using CPT and CPTU", PK Robertson and RG Campanella 1989, Robertson and Wride, 1998, and/or site specific data.
3. See report text for limitations of testing and interpretation of data

Legend:

		Ic <	1.30	Gravelly Sand	🟦
1.30	<	Ic <	1.80	Sand to Silty Sand	🟨
1.80	<	Ic <	2.10	Silty Sand to Sandy Silt	🟧
2.10	<	Ic <	2.95	Clayey Silt to Silty Clay	🟩
2.95	<	Ic <	3.60	Clay	🟩
3.60	<	Ic		Clay and Organic Soils	🟪

Static Groundwater Surface —·—··—·—

Static Groundwater Pressure ·············

Operator Initials:	CC	Easting - X Coordinate (m):	330,938.0
Cone Identification Number:	3806	Northing - Y Coordinate (m):	4,681,547.0
Correction Factor a	0.584	Ground Surface Elevation (m):	180.6
Correction Factor b	0.012	Depth to Groundwater (m):	1.0
		Predrilling Depth (m):	3.1

Golder Associates

Project No.:	04-1111-060
Date:	09/11/2006
Check:	SJB
Review:	FJH

Detroit River International Crossing

Cone Penetration Test

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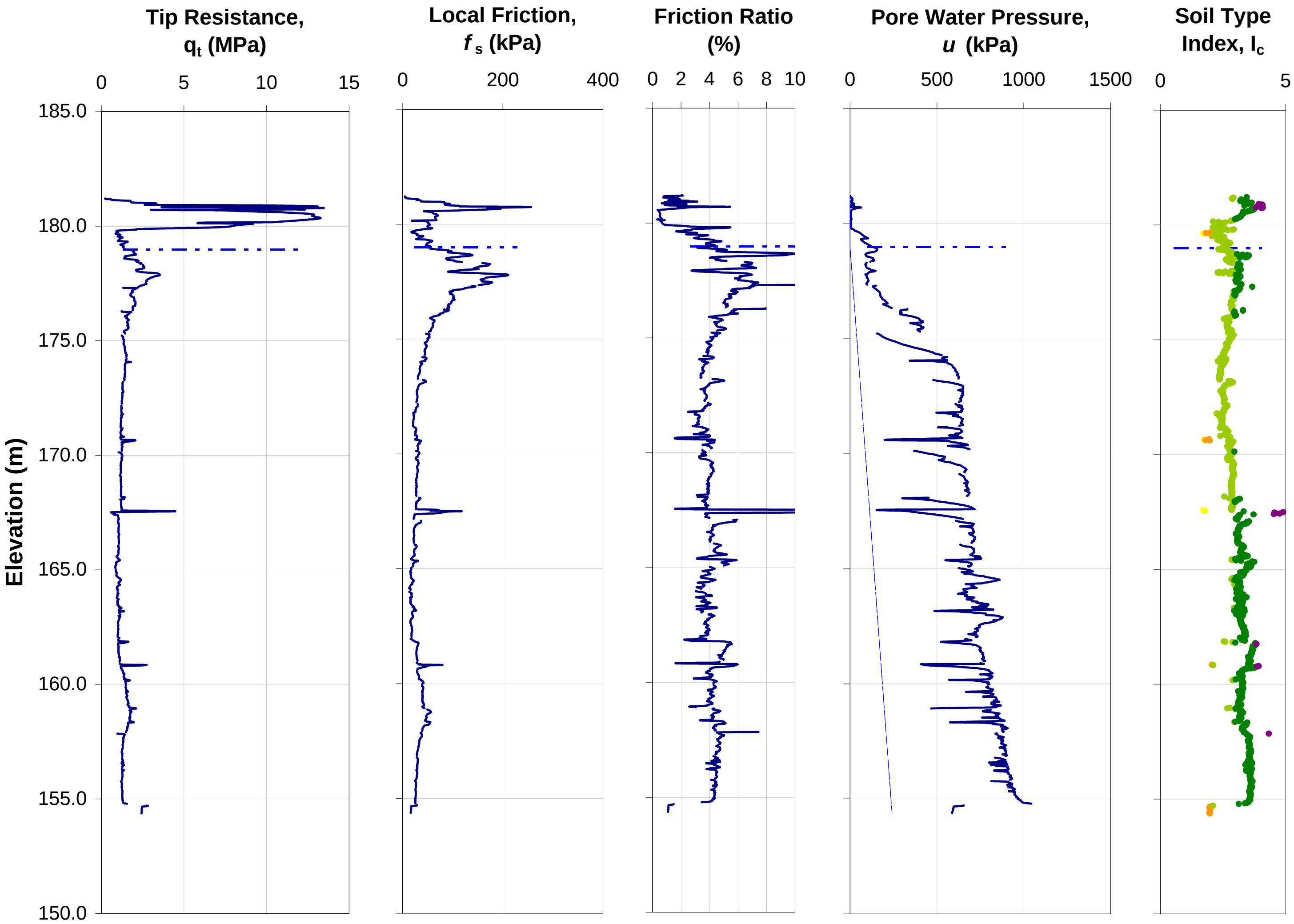

Notes:

1. Soil Index Type, Ic, is an inferred soil type and actual conditions may differ.
2. Soil Index Type based on "Guidelines for Geotechnical Design Using CPT and CPTU", PK Robertson and RG Campanella 1989, Robertson and Wride, 1998, and/or site specific data.
3. See report text for limitations of testing and interpretation of data

Legend:

Ic <	1.30 Gravelly Sand	🟦
1.30 < Ic <	1.80 Sand to Silty Sand	🟨
1.80 < Ic <	2.10 Silty Sand to Sandy Silt	🟧
2.10 < Ic <	2.95 Clayey Silt to Silty Clay	🟩
2.95 < Ic <	3.80 Clay	🟩
3.80 < Ic	Clay and Organics	🟪

Static Groundwater Surface — · · — · · —

Static Groundwater Pressure ··················

Operator Initials:	CC	Easting - X Coordinate (m):	330,413.0
Cone Identification Number:	3742	Northing - Y Coordinate (m):	4,681,906.0
Correction Factor a	0.6	Ground Surface Elevation (m):	181.2
Correction Factor b	0.013	Depth to Groundwater (m):	2.2
		Predrilling Depth (m):	0.0

Golder Associates

Project No.:	04-1111-060
Date:	02/11/2006
Check:	SJB
Review:	FJH

Detroit River International Crossing

Cone Penetration Test

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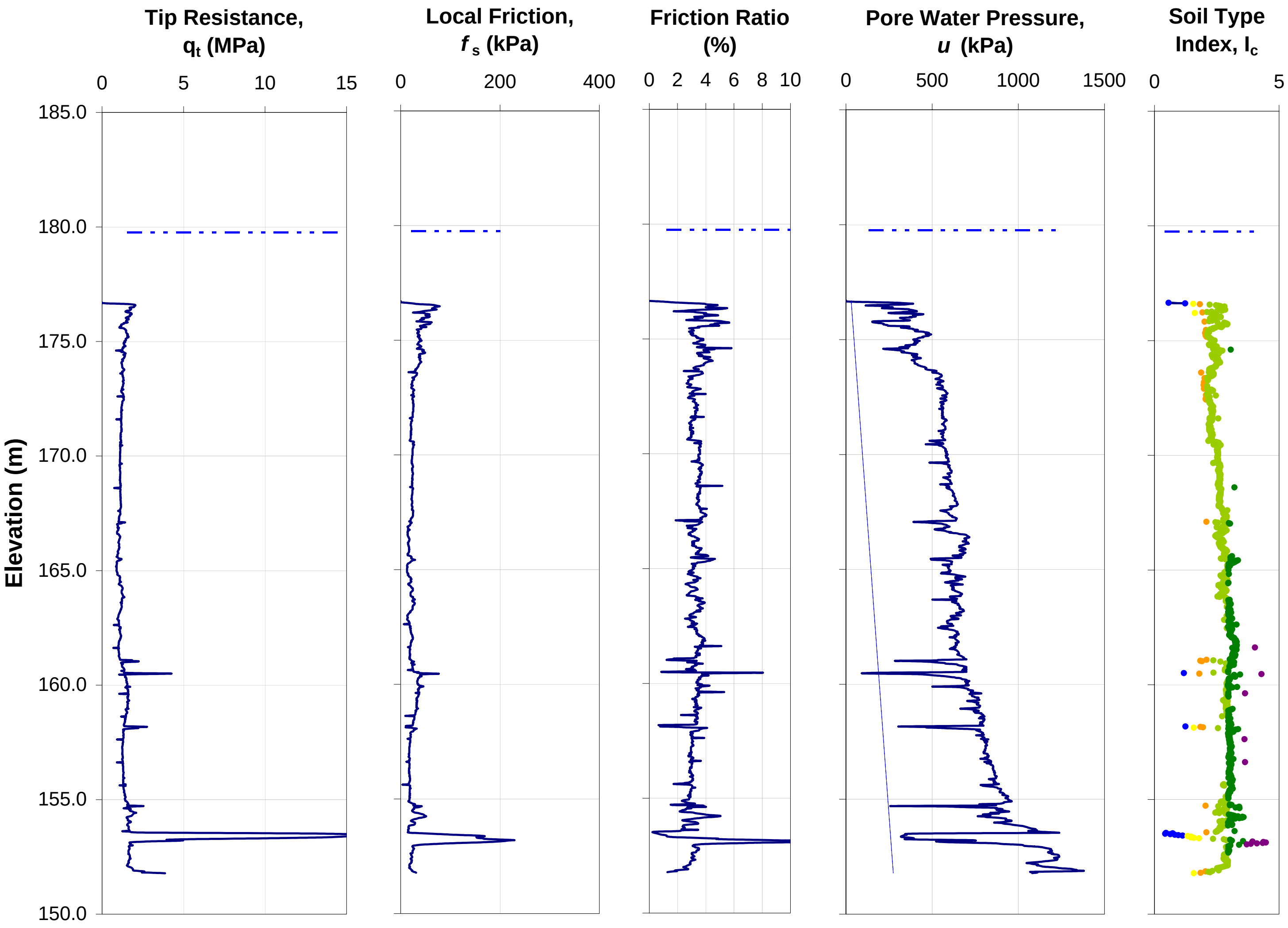

Notes:

1. Soil Index Type, Ic, is an inferred soil type and actual conditions may differ.
2. Soil Index Type based on "Guidelines for Geotechnical Design Using CPT and CPTU", PK Robertson and RG Campanella 1989, Robertson and Wride, 1998, and/or site specific data.
3. See report text for limitations of testing and interpretation of data

Legend:

		Ic <	1.30	Gravelly Sand	🟦
1.30	<	Ic <	1.80	Sand to Silty Sand	🟨
1.80	<	Ic <	2.10	Silty Sand to Sandy Silt	🟧
2.10	<	Ic <	2.95	Clayey Silt to Silty Clay	🟩
2.95	<	Ic <	3.60	Clay	🟩
3.60	<	Ic		Clay and Organic Soils	🟪

Static Groundwater Surface — · · — · · —

Static Groundwater Pressure ···············

Operator Initials:	CC	Easting - X Coordinate (m):	329,868.0
Cone Identification Number:	3806	Northing - Y Coordinate (m):	4,681,775.0
Correction Factor a	0.584	Ground Surface Elevation (m):	179.8
Correction Factor b	0.012	Depth to Groundwater (m):	0.0
		Predrilling Depth (m):	3.1

Golder Associates

Project No.: 04-1111-060
Date: 09/11/2006
Check: SJB
Review: FJH

Detroit River International Crossing

Cone Penetration Test

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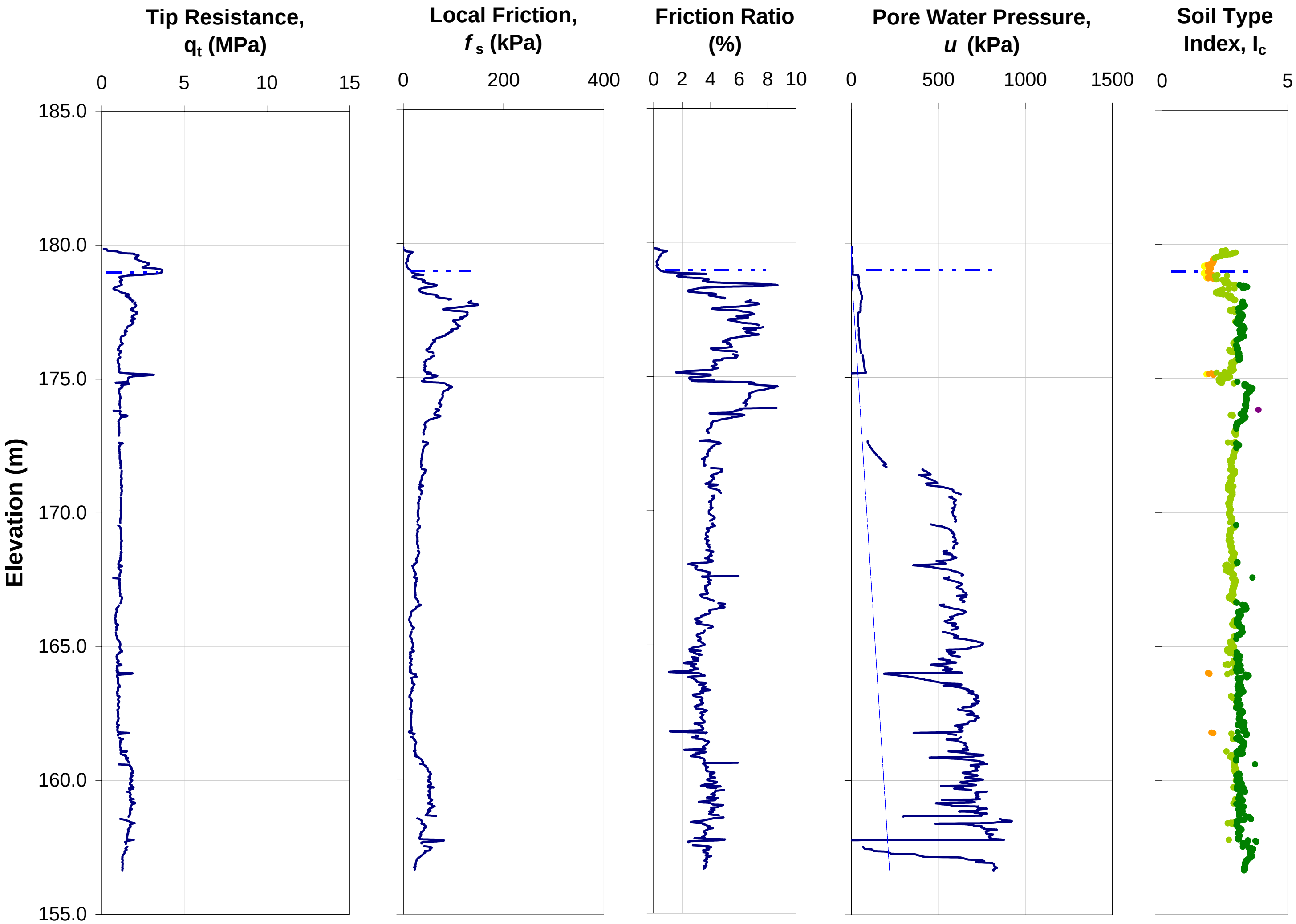

Notes:

1. Soil Index Type, Ic, is an inferred soil type and actual conditions may differ.
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3. See report text for limitations of testing and interpretation of data

Legend:

Ic <	1.30 Gravelly Sand	🟦
1.30 < Ic <	1.80 Sand to Silty Sand	🟨
1.80 < Ic <	2.10 Silty Sand to Sandy Silt	🟧
2.10 < Ic <	2.95 Clayey Silt to Silty Clay	🟩
2.95 < Ic <	3.80 Clay	🟩
3.80 < Ic	Clay and Organics	🟪
Static Groundwater Surface		— · · — · · —
Static Groundwater Pressure		··············

Operator Initials:	CC	Easting - X Coordinate (m):	329,759.0
Cone Identification Number:	3806	Northing - Y Coordinate (m):	4,682,147.0
Correction Factor a	0.584	Ground Surface Elevation (m):	179.9
Correction Factor b	0.012	Depth to Groundwater (m):	0.9
		Predrilling Depth (m):	0.0

Golder Associates

Project No.:	04-1111-060
Date:	01/11/2006
Check:	SJB
Review:	FJH

Detroit River International Crossing

Cone Penetration Test

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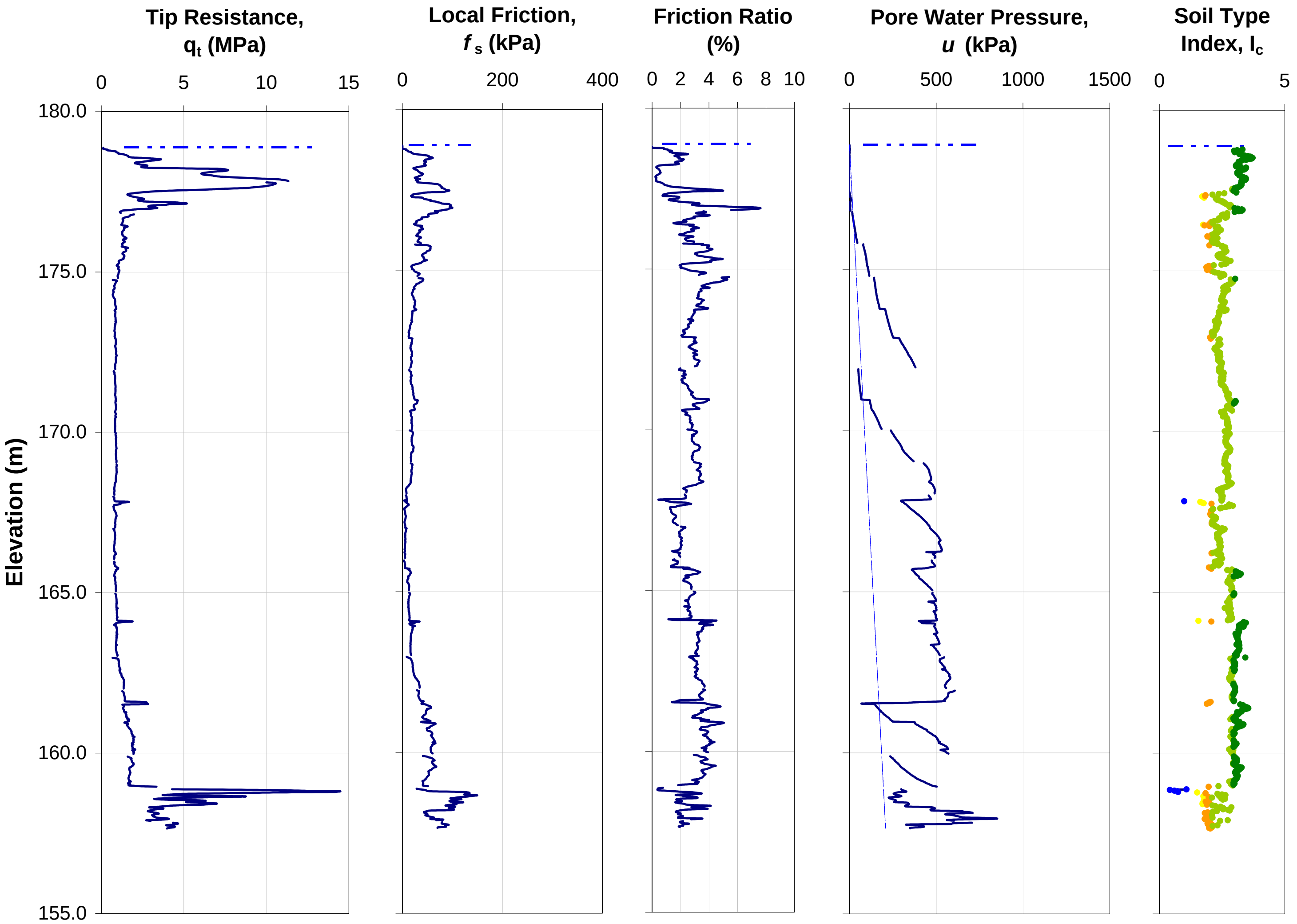


Notes:

1. Soil Index Type, Ic, is an inferred soil type and actual conditions may differ.
2. Soil Index Type based on "Guidelines for Geotechnical Design Using CPT and CPTU", PK Robertson and RG Campanella 1989, Robertson and Wride, 1998, and/or site specific data.
3. See report text for limitations of testing and interpretation of data

Legend:

Ic <	1.30 Gravelly Sand	🟦
1.30 < Ic <	1.80 Sand to Silty Sand	🟨
1.80 < Ic <	2.10 Silty Sand to Sandy Silt	🟧
2.10 < Ic <	2.95 Clayey Silt to Silty Clay	🟩
2.95 < Ic <	3.80 Clay	🟩
3.80 < Ic	Clay and Organics	🟪

Static Groundwater Surface — · · — · · —

Static Groundwater Pressure ··········

Operator Initials:	CC	Easting - X Coordinate (m):	328,986.0
Cone Identification Number:	3806	Northing - Y Coordinate (m):	4,682,412.0
Correction Factor a	0.584	Ground Surface Elevation (m):	178.9
Correction Factor b	0.012	Depth to Groundwater (m):	0.0
		Predrilling Depth (m):	0.0

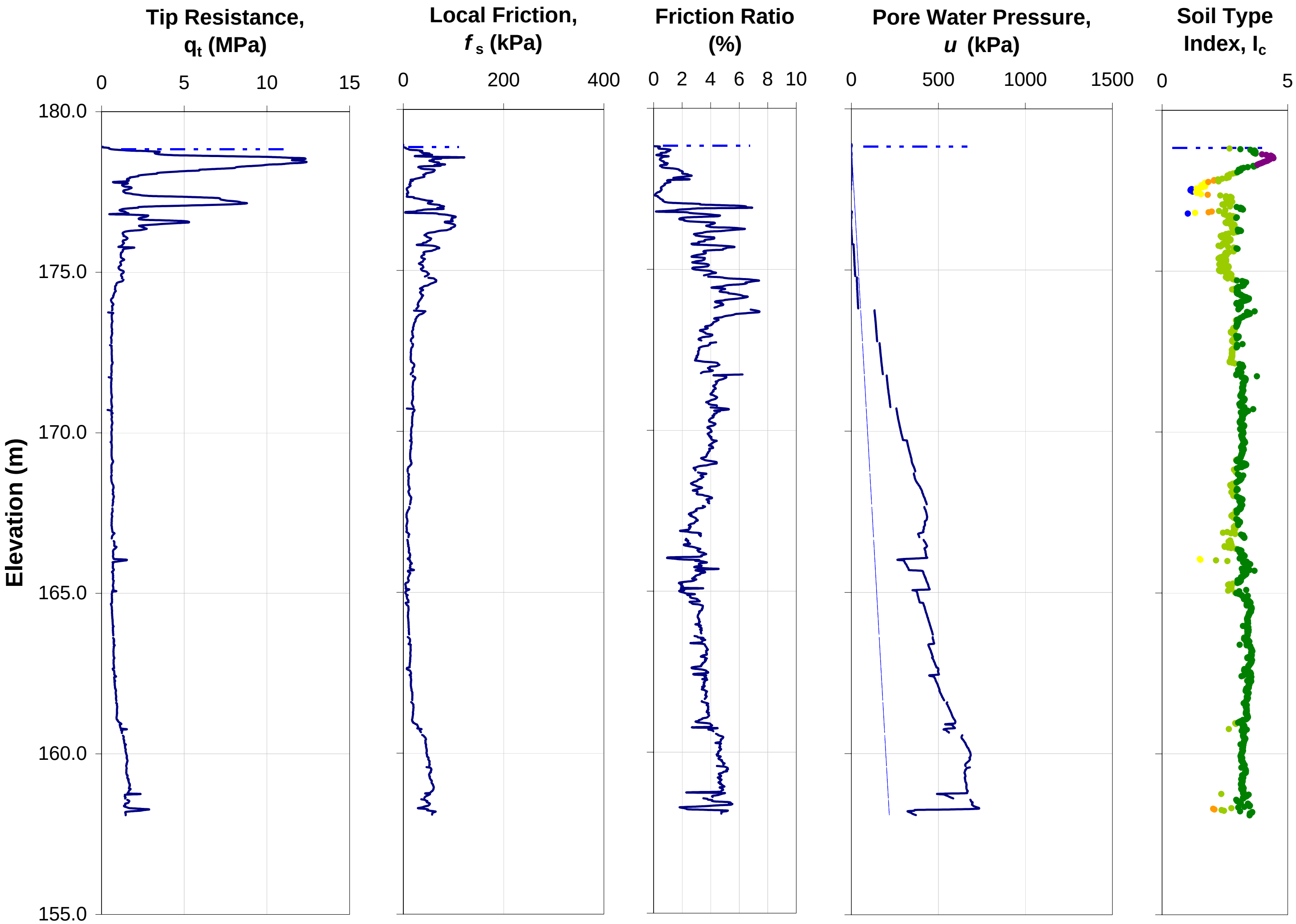
Golder Associates

Project No.: 04-1111-060
Date: 01/11/2006
Check: SJB
Review: FJH

Detroit River International Crossing

Cone Penetration Test

22



Notes:

1. Soil Index Type, Ic, is an inferred soil type and actual conditions may differ.
2. Soil Index Type based on "Guidelines for Geotechnical Design Using CPT and CPTU", PK Robertson and RG Campanella 1989, Robertson and Wride, 1998, and/or site specific data.
3. See report text for limitations of testing and interpretation of data

Legend:

		Ic <	1.30	Gravelly Sand	🟦
1.30	<	Ic <	1.80	Sand to Silty Sand	🟨
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2.95	<	Ic <	3.80	Clay	🟩
3.80	<	Ic		Clay and Organics	🟪

Static Groundwater Surface — · · — · · —

Static Groundwater Pressure ··························

Operator Initials:	CC	Easting - X Coordinate (m):	328,523.0
Cone Identification Number:	3806	Northing - Y Coordinate (m):	4,682,325.0
Correction Factor a	0.584	Ground Surface Elevation (m):	178.9
Correction Factor b	0.012	Depth to Groundwater (m):	0.1
		Predrilling Depth (m):	0.0

Golder Associates

Project No.: 04-1111-060
Date: 31/10/2006
Check: SJB
Review: FJH

Detroit River International Crossing

Cone Penetration Test

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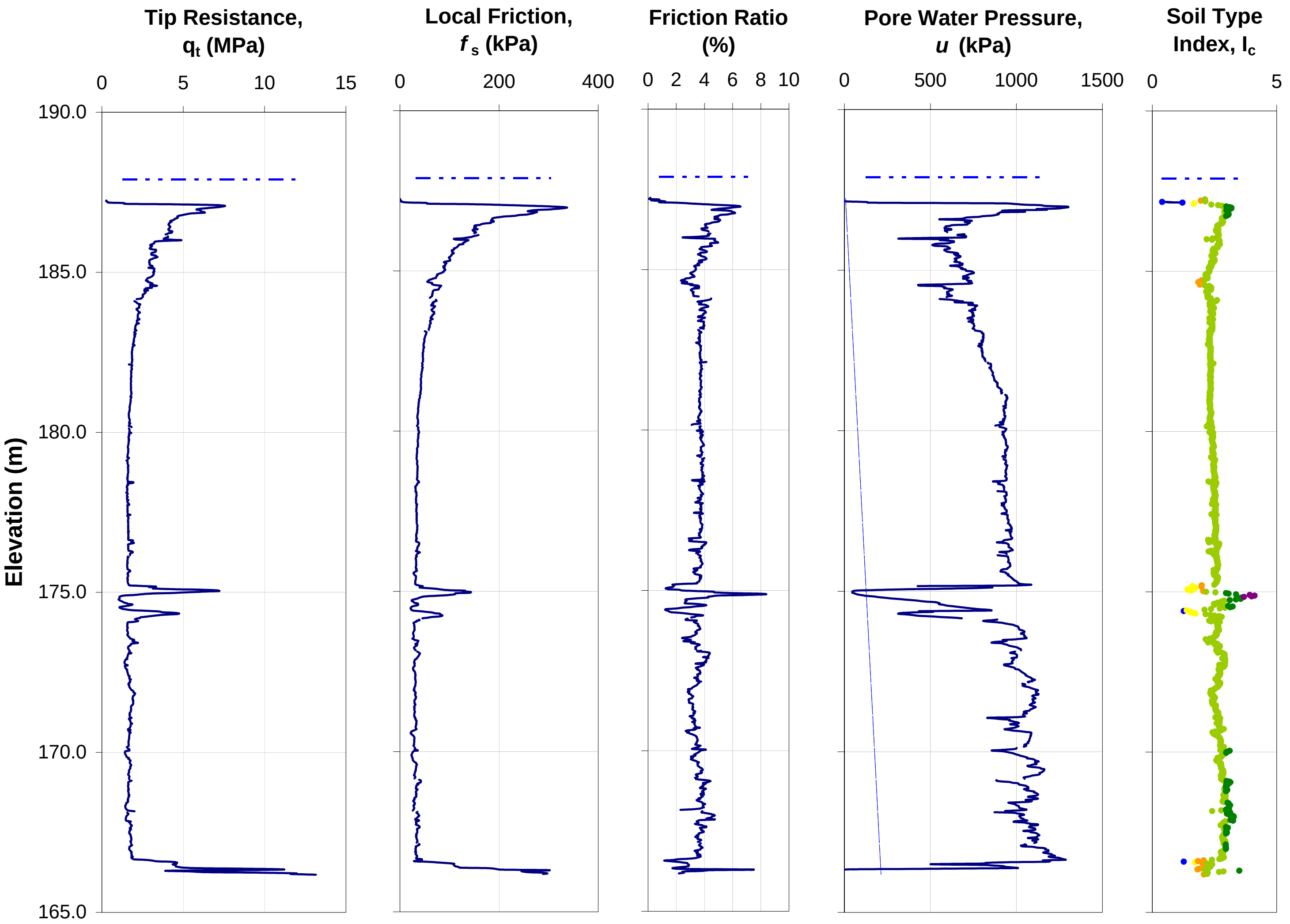

Notes:

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3. See report text for limitations of testing and interpretation of data

Legend:

		Ic <	1.30	Gravelly Sand	🟦
1.30	<	Ic <	1.80	Sand to Silty Sand	🟨
1.80	<	Ic <	2.10	Silty Sand to Sandy Silt	🟧
2.10	<	Ic <	2.95	Clayey Silt to Silty Clay	🟩
2.95	<	Ic <	3.60	Clay	🟩
3.60	<	Ic		Clay and Organic Soils	🟪

Static Groundwater Surface —·—··—·—

Static Groundwater Pressure ··············

Operator Initials:	CC	Easting - X Coordinate (m):	338,376.0
Cone Identification Number:	3806	Northing - Y Coordinate (m):	4,679,216.0
Correction Factor a	0.584	Ground Surface Elevation (m):	190.2
Correction Factor b	0.012	Depth to Groundwater (m):	2.3
		Predrilling Depth (m):	3.0

Golder Associates

Project No.:	04-1111-060
Date:	14/11/2006
Check:	SJB
Review:	FJH

Detroit River International Crossing

Cone Penetration Test

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APPENDIX C

FIELD VANE SHEAR TEST DATA

NILCON FIELD VANE SHEAR TEST RESULTS

Detroit River International Crossing

Depth	Elevation	Undrained Shear Strength (kPa)			Sensitivity
(m)	(m)	Natural	Post-Peak	Remoulded	
Field Vane Location 1 (Borehole BH-1)					
5.1	181.6	151	109	93	1.6
6.1	180.6	115	100	73	1.6
7.1	179.6	88	72	66	1.3
8.1	178.6	110	93	64	1.7
9.1	177.6	94	80	62	1.5
10.1	176.6	79	64	59	1.3
11.1	175.6	89	70	62	1.4
12.1	174.6	87	64	47	1.8
13.1	173.6	65	53	55	1.2
14.1	172.6	67	50	47	1.4
15.1	171.6	56	49	49	1.1
16.1	170.6	70	51	47	1.5
17.1	169.6	66	45	51	1.3
18.1	168.6	59	42	45	1.3
19.1	167.6	67	42	53	1.3
20.1	166.6	79	41	45	1.7
21.1	165.6	66	28	42	1.6
22.1	164.6	68	27	49	1.4
23.1	163.6	42	33	68	0.6
Field Vane Location 7 (Borehole BH-7)					
6.1	177.1	115	100	76	1.5
7.1	176.1	89	67	47	1.9
8.1	175.1	90	61	36	2.5
9.1	174.1	81	52	28	2.8
10.1	173.1	73	60	17	4.3
11.1	172.1	78	61	30	2.6
12.1	171.1	80	59	32	2.5
13.1	170.1	69	44	23	3.0
14.1	169.1	67	48	19	3.6
15.1	168.1	77	62	30	2.6
16.1	167.1	66	36	15	4.4
17.1	166.1	61	38	21	2.9
18.1	165.1	44	35	15	2.9
19.1	164.1	84	60	40	2.1
Field Vane Location 14 (Borehole BH-14)					
6.0	176.0	77	46	35	2.2
7.0	175.0	58	31	15	3.8
8.0	174.0	66	40	29	2.2
9.0	173.0	56	33	24	2.3
10.0	172.0	56	33	24	2.4
11.0	171.0	56	32	24	2.4
12.0	170.0	49	30	23	2.1
13.0	169.0	45	27	21	2.2
14.0	168.0	53	39	22	2.4
15.0	167.0	41	16	16	2.5

NILCON FIELD VANE SHEAR TEST RESULTS

Detroit River International Crossing

Depth	Elevation	Undrained Shear Strength (kPa)			Sensitivity
(m)	(m)	Natural	Post-Peak	Remoulded	
16.0	166.0	47	17	14	3.3
17.0	165.0	39	16	9	4.1
18.0	164.0	47	15	16	2.9
19.0	163.0	38	18	22	1.7
20.0	162.0	37	32	28	1.3
Field Vane Location 23 (Borehole BH-23)					
5.0	173.9	51	23	15	3.3
6.0	172.9	43	19	9	5.0
7.0	171.9	36	20	10	3.5
8.0	170.9	37	20	10	3.5
9.0	169.9	33	19	9	3.5
10.0	168.9	29	14	8	3.9
11.0	167.9	30	17	6	5.3
12.0	166.9	39	11	5	8.2
13.0	165.9	23	17	7	3.5
14.0	164.9	21	14	4	5.5
15.0	163.9	21	8	14	1.5
16.0	162.9	30	16	17	1.8
17.0	161.9	45	21	26	1.8