GEOCRES No: 31G5-201

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DRAFT
REPORT ON

FOUNDATION INVESTIGATION AND DESIGN
CN RAIL OVERPASS WIDENING
STRUCTURE SITE 3-257 (EBL)
HIGHWAY 417
G.W.P. 458-98-00

DRAFT

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FOUNDATION INVESTIGATION REPORT
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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Marshall Macklin Monaghan (MMM) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations associated with the twinning of Highway 7 from two to four lanes in West Carleton and Goulbourn Townships in the City of Ottawa, and in Beckwith Township in Lanark County. The sections of Highway 7 included in this assignment extend from Highway 417 westerly 7 km to 3 km west of Jinkinson Road (W.P. 256-99-00), and from 3 km west of Jinkinson Road westerly to Carleton Place (W.P. 251-99-00 and 252-99-00). Foundation investigation services are also required as part of this assignment for the widening of Highway 417 from the Highway 417-7 interchange easterly (W.P. 458-98-00) to Moodie Drive.

Foundation investigation services are required for the following components:

- **W.P. 256-99-00:** New structures at the Highway 417E-7W ramp and Hazeldean Road, including a high fill embankment along the Highway 417E-7W ramp, and overhead signs.

- **W.P. 251-99-00 and 252-99-00:** Five new structures at Appleton Road, Ashton Station Road, Dwyer Hill Road, the Trans-Canada Trail, and Lavallee Creek.

- **W.P. 458-98-00:** Widening of two existing structures (the Carp River Bridge and CN Rail overpass) into the existing Highway 417 median area, a 900 m long section of high fill embankment within the Highway 417 median in the vicinity of the CN Rail overpass, high mast light poles, and overhead signs.

This report addresses the widening of the CN Rail overpass structure (eastbound lanes).

The terms of reference for the original scope of work and Addenda 1 through 7 issued during the proposal period are outlined in the MTO's Request for Proposal (RFP) and in Golder Associates' Proposal No. P21-1301, dated July 2002. Scope changes related to additional borehole investigation work at the abutments of several structures and the high fill embankment on the Highway 417E-7W ramp are outlined in Golder Associates' letters dated November 12, 2002 and November 18, 2002, respectively.

2.0 SITE DESCRIPTION

The existing CN Rail overpass structures are located on Highway 417 approximately 5 km west of Richmond Road (Highway 15) in Ottawa, Ontario. Through this section, Highway 417 consists of three eastbound lanes (EBL) and four westbound lanes (WBL) divided by a 30 m to 40 m wide median. The eastbound and westbound lanes are carried over the CN Rail line on separate structures. These two existing structures are designated as MTO's Structure Site 3-257.

The existing bridges for both the eastbound and westbound lanes consist of a concrete deck on precast concrete girders, supported on concrete abutments and piers. The bridges span approximately 45 m between the abutments. The foundation investigations for the design of these two bridges were carried out in 1966 and the results of those investigations are summarised in MTO's GEOCRES No. 31G5-64-01 and 31G5-64-02, *Foundation Investigation Report, W.P. 108-65, Proposed Overhead at the Ottawa Queensway and C.N.R. Crossing, District #9 (Ottawa) Hwy: Queensway W.J. 66-F-17.* The Department of Highways' Ontario Bridge Division Drawing No. D6138-P-2, "*C.N.R. Overhead,*" dated May 1967, indicates the EBL structure to be supported on steel H-piles and the elevation of the bridge deck to vary from about Elev. 93.5 m to 94.6 m. The WBL bridge is assumed to be of similar construction.

It is currently proposed to increase the capacity of Highway 417 in this area to five lanes, including a bus lane in both the eastbound and westbound directions. It is our understanding that the existing WBL structure is wide enough to accommodate the increased number of lanes without requiring any structural modification. However, the existing EBL structure is too narrow and is proposed to be widened by about 8.9 m to accommodate the additional traffic lanes. Widening of the existing approach embankments by about 8 m (EBL) and 4 m (WBL) into the median swale with new fill up to about 1.3 m thick will also be required.

The results of the 1966 foundation investigations indicate that the overburden at the CN Rail overpass structures consists of surficial sands, overlying a thick deposit of sensitive clayey silt to silty clay, overlying a thin layer of sandy till underlain by bedrock at a depth of about 18 m.

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out for the widening of the eastbound CN Rail overpass structure between November 25 and December 6, 2002. During this time, a total of three sampled boreholes and two cone penetration tests (CPT) were advanced within the area of the proposed structure and approach embankment widening.

Clarification a location of BH's

The boreholes were advanced by hollow stem augers using a truck-mounted drill rig, supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario. The three sampled boreholes were advanced to depths ranging from 19.7 m to 23.7 m below the existing ground surface. Samples of the overburden were obtained at 0.75 m to 1.5 m intervals of depth increasing to 3 m intervals below a depth of about 12 m, using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. In situ vane shear strength testing was carried out using an MTO 'N'-sized vane at regular intervals of depth, where appropriate, in clayey strata. Samples of bedrock were obtained using an 'NQ' size rock core barrel.

The water level in the open boreholes was observed throughout the drilling operations and two standpipe piezometers were installed in selected boreholes to monitor the groundwater level(s) at the site. The screened portion of each standpipe was installed within the overburden soils at a depth of about 20 m. Both of the standpipes consist of 1 inch diameter rigid PVC pipe with a 1.5 m long slotted screen section, installed within a silica sand filter pack, and sealed below minimum 1 m long sections of bentonite pellet backfill.

Two additional boreholes were advanced through the fill embankment in the median swale in order to facilitate the start of the CPTs. The CPT is a in situ technique for site characterisation studies. The CPT consists of a special cone tip equipped with electronic sensing elements to continuously measure tip resistance, local side friction on a sleeve 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 track-mounted drill rig. The two CPTs were advanced to refusal, which was encountered at depths ranging from about 18.6 m to 20.8 m below ground surface. Record of Cone Penetration Test sheets are included with the Record of Borehole sheets following the text of this report. Profiles of tip resistance, porewater pressure during pushing and sleeve-friction are presented together with an interpreted profile of undrained shear strength (s_u) and classification index (I_c) that is used to infer soil type (stratigraphy).

The field work was supervised on a full-time basis by members 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 laboratory in Ottawa for further examination, and subsequently to Golder's laboratory in Mississauga for testing. Laboratory testing, including water content determinations, Atterberg Limits testing and grain size distribution analyses, was carried out on selected soil samples.

The borehole/CPT locations and ground surface elevations were determined by Golder relative to points staked in the field by MMM. The borehole locations, including MTM NAD83 northing and easting coordinates and ground surface elevations referenced to geodetic datum, are summarized in the following table and are shown on Drawing 1.

Borehole / CPT Number	*Borehole/CPT Location*	*MTM NAD83 Northing (m)*	*MTM NAD83 Easting (m)*	*Ground Surface Elevation (m)*
CPT02-401	STA 12+636, 3.2 m north of centreline stake	5021560.1	355216.3	93.0
02-402	STA 12+650, 5 m north of centerline stake	5021574.1	355243.2	94.0
02-403	STA 12+694 at centreline stake	5021592.4	355264.0	87.0
02-404	STA 12+721, 5.3 m north of centerline stake	5021601.8	355290.1	92.5
CPT02-405	STA 12+738, 1.9 m north of centerline stake	5021613.4	355303.0	90.7

Borehole Abandonment

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

The study area for this assignment lies within two minor physiographic regions that lie within the major physiographic region of the Ottawa-St. Lawrence Lowland (Chapman and Putnam, 1984). The Highway 7 area between the Highway 417-7 interchange and Carleton Place is part of the Smiths Falls Limestone Plain, while the area along Highway 417 east of the Highway 417-7 interchange is part of the Ottawa Valley Clay Plain. Most of both physiographic regions are underlain by a series of sedimentary rocks, consisting of sandstones, dolostones, limestones and shales that are, in turn, underlain by igneous and metamorphic bedrock of the Precambrian Shield. The Shield rock generally outcrops to the north of the Ottawa River, and it is also present immediately below the overburden in a localized area between the Hazeldean Fault (approximately the location of the Carp River) and the Ottawa River.

The Smiths Falls Limestone Plain is characterized by shallow overburden deposits overlying limestone bedrock of the Ottawa Formation; this formation consists of grey limestone with some shaly partings and seams (Belanger, 1998). The shallow overburden soils are typically between 1 m and 3 m in thickness and are commonly comprised of sandy to gravelly till derived from the Precambrian Shield to the north, overlain by glaciofluvial sediments that consist of layered sands and gravels. Large areas of the plain are covered with peat and muck, due to poor drainage as a consequence of the relatively flat topography and shallow depth to bedrock (Chapman and Putnam, 1984).

The Ottawa Valley Clay Plain region, present along Highway 417 from the Highway 417-7 interchange site eastward, is characterized by relatively thick deposits of sensitive marine clay, silt and silty clay that were deposited within the Champlain Sea basin. These deposits, known as the Champlain Sea clay or Leda clay, overlie relatively thin, commonly reworked glacial till and glaciofluvial deposits, that in turn overlie bedrock (Chapman and Putnam, 1984). West of the Carp River valley along Highway 417, the upper bedrock consists of limestone of the Ottawa Formation, as described above. Along the east flank of the Carp River valley, the upper bedrock consists of sandstones and dolostones that have been cut by igneous and metamorphic rocks, controlled by faulting in the vicinity of the Hazeldean Fault and associated higher ground of the Carp Ridge (Belanger, 1998). That fault is considered to be seismically inactive, however the bedrock in the vicinity of that fault is known to be rather fractured and to have a highly irregular surface topography.

4.2 Site Stratigraphy

As part of the subsurface investigation at this site, three boreholes and two CPTs were advanced within the limits of the foundation elements and immediate approach embankments for the proposed structure and embankment widening. The borehole locations and ground surface elevations are shown on Drawing 1.

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, Drillhole and Cone Penetration Test Sheets following the text of this report. The results of the laboratory testing are provided in Appendix A. A summary of the in situ field results along with the laboratory test results is presented on Figure 1. 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 boreholes therefore, represents transition between soil types rather than exact planes of geological change. Furthermore, subsurface conditions will vary between and beyond the borehole locations.

In summary, the subsurface conditions in the area of the CN Rail structure consist of topsoil underlain by about 1.5 m to 10.1 m of fill, varying in composition from earth fill to rock fill. Beneath the fill material is a deposit of silty clay to clay with a total thickness ranging from about 10.0 m to 17.2 m. The upper approximately 4 m of the silty clay stratum has a weathered, stiff to very stiff crust, while the underlying portion has a stiff to firm consistency. The silty clay is underlain by a 0.2 m to 0.7 m layer of silty sand and gravel till overlying the bedrock surface. The surface of the bedrock appears to slightly slope across the site from west to east with depths at the borehole locations ranging from about 16.8 m to 23.2 m (Elevation 70.2 m to 70.8 m).

A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections. Stratigraphic profiles and sections of this site are shown on Drawing 1 ***(Note: Awaiting electronic General Arrangement drawing files to produce Soil Strata drawing).***

4.2.1 Topsoil

Topsoil was encountered at the ground surface in the boreholes advanced near the abutment areas of the proposed EBL structure widening. About 200 mm and 300 mm of topsoil was encountered in Boreholes 02-402 and 02-404, respectively.

4.2.2 Fill

Fill, associated with the construction of the existing approach embankments and associated with the construction of the existing CN Rail access road located adjacent to the rail line, was encountered in all of the boreholes and CPTs advanced as part of this subsurface investigation.

In Borehole 02-403 drilled on the CN Rail access road located at the toe of the eastern approach embankments, approximately 1.5 m of sand and gravel fill was encountered immediately below the ground surface. A single Standard Penetration Test (SPT) 'N' value equal to 19 blows per 0.3 m of penetration was measured in this layer, indicating that the fill in this area has a compact relative density.

In Boreholes 02-402 and 02-404 drilled near ~~the~~ west and east abutment areas of the proposed EBL structure widening, a total of about 5.6 m and 9.4 m, respectively, of earth fill was encountered immediately below ~~the topsoil~~. ~~The~~ upper 0.6 m to 2.9 m of the fill in these areas is composed of a grey-brown silty clay containing trace sand and gravel, to sandy silt containing trace clay. The lower 5.0 m ~~to 6.4 m of~~ the fill is composed of a brown sand and gravel to sand, trace to some silt and gravel, containing occasional cobbles.

firm to describe cohesive

A single Standard Penetration Test (SPT) carried out in the upper portions ~~of the~~ earth fill measured an 'N' value of 2 blows per 0.3 m of penetration, suggesting a very loose state of packing. In the lower portion of the earth fill, the measured SPT 'N' values ~~range from~~ 6 to 17 blows per 0.3 m of penetration, indicating a loose to compact relative density. Natural water contents measured on selected samples of the earth fill ranged ~~between 4 and 22 per cent~~. Grain size distribution curves for selected samples of the fill are shown on Figures A2 and A3. give titles

In Boreholes CPT02-401 and CPT02-405 drilled in the area of the new approach embankments for the EBL and WBL structures, about 4.1 m to 10.1 m, respectively, of fill was encountered immediately below the ground surface. The fill in these areas is generally composed of sand and gravel, containing cobbles and boulders.

4.2.3 Clayey Silt to Sandy Silt

cohesive vs non cohesive.

Beneath the fill material in Borehole 02-403, a 2.9 m thick layer of clayey silt to sandy silt trace to some clay was encountered. Standard Penetration Test (SPT) measured 'N' values ranged between 4 and 19 blows per 0.3 m of penetration (decreasing with depth) indicating a very stiff to firm consistency. Natural water contents of 21 and 25 per cent were measured on selected samples from this deposit.

4.2.4 Silty Clay to Clay

A deep deposit of silty clay to clay was encountered below the fill and/or the clayey silt to sandy silt deposit in all of the boreholes and CPTs in this area. The thickness of the silty clay to clay ranges from about 10.0 to 17.2 m at the locations investigated. The upper approximately 4.0 m of the deposit is composed of a weathered brown to grey brown silty clay crust containing occasional silty sand and clayey silt seams. The lower approximately 8 m to 13 m of the deposit is composed of a brown grey to grey silty clay to clay. Standard Penetration Testing (SPT) carried out in the upper weathered silty clay measured 'N' values varying from 2 to 14 blows per 0.3 m of penetration indicating a soft to stiff consistency while SPTs carried out in the lower silty clay measured 'N' values of 0 (weight of rod or static weight of hammer) to 2 blows per 0.3 m of penetration indicating a very soft to soft consistency.

provide Elevation

The results of in situ vane tests carried out in Boreholes 02-402 to 02-404 are shown on the Record of Borehole sheets and summarised on Figure 1. The interpreted profiles of undrained shear strength from the CPTs carried out in Boreholes CPT02-401 and CPT02-405 are shown on the Cone Penetration Test sheets and included in the summary on Figure 1. Based on the in situ vane tests, the undrained shear strength of the upper weathered silty clay varies from about 50 kPa to 85 kPa and the undrained shear strength of the lower silty clay varies from about 40 kPa to 60 kPa. These results are generally consistent with the results from the CPTs that estimate the undrained shear strength to vary from 50 kPa to greater than 100 kPa in the upper silty clay and vary from 45 kPa to 70 kPa in the lower silty clay.

The sensitivity of the silty clay to clay stratum, as estimated from the in situ vane tests, generally ranges from about 3.5 to 15 (increasing with depth) with a few values of 25 or greater, as shown on Figure 1. This range implies that the silty clay to clay is typically of medium to extra-sensitivity (CFEM, 1992).

Natural water contents measured on selected samples of the upper silty clay to clay ranged from about 22 to 51 per cent with an average of about 35 per cent. Atterberg Limits testing conducted on selected samples of the upper silty clay to clay gave liquid limits ranging from about 38 to 71 per cent and plasticity indices ranging from about 20 to 48 per cent, indicating clay of intermediate to high plasticity. Natural water contents measured on samples of lower silty clay ranged from about 35 to 65 per cent with an average of about 47 per cent. Atterberg Limits testing conducted on selected samples of the lower silty clay to clay gave liquid limits ranging from about 45 to 66 per cent and plasticity indices ranging from about 19 to 42 per cent, indicating clay of intermediate ~~to high plasticity~~. The results of the Atterberg Limits tests are plotted on the plasticity chart on Figure A5.

title

A summary of the natural water content, Atterberg Limits, and undrained shear strengths for this deposit is shown on Figure 1.

The result of a grain size distribution carried out on a selected sample of the clay is shown on Figure A4.

Laboratory oedometer (consolidation) tests were not performed on samples obtained from the boreholes carried out for the investigation and design of the overpass widening. However, oedometer tests were performed on selected samples of the silty clay obtained from the investigation for the Tall Rock Fill embankment widening (Golder report: 021-1155-12) located immediately adjacent to the overpass widening area. The results of these oedometer tests were used to calibrate the estimations of the preconsolidation pressure profiles from the CPTs carried out in Boreholes CPT02-401 and CPT02-405.

Discussion should be include of oedometer

The results of the 1966 foundation investigation carried out at this site indicate the presence of an approximately 16 m thick silty clay deposit beneath the existing EBL structure. The top of this deposit was encountered at about at Elevation 87.9 m corresponding to a depth of about 0.8 m below the ground surface at that time. The results of the in situ vane tests measured undrained shear strength values ranging from about 45 to 95 kPa, which are consistent with our 2002 investigation.

4.2.5 Silty Sand Till

A 0.2 m to 0.7 m thick layer of silty sand till containing some gravel was encountered below the silty clay to clay stratum in all of the sampled boreholes. Measured SPT 'N' values of 10 blows for 0.03 m and 0.18 m of penetration (before reaching refusal on bedrock) suggest a compact to dense state of packing. The 1966 foundation investigation also encountered this thin layer of sandy till material, about 1.0 m thick, below the silty clay stratum.

Any boulders & cobbles? see pg. 13

4.2.6 Sandstone Bedrock

Sandstone bedrock underlies the silty sand and gravel till deposit at this site. In the three sampled boreholes (Boreholes 02-402, 02-403 and 02-404), the surface of the bedrock was encountered between Elevations 70.2 m and 70.8 m. The following table summarises the bedrock surface depth and elevation as encountered at the borehole locations.

Borehole Number	*Ground Surface Elevation (m)*	*Depth to Bedrock (m)*	*Bedrock Surface Elevation (m)*
02-402	94.0	23.2	70.8
02-403	87.0	16.8	70.2
02-404	92.5	22.3	70.2

The sandstone bedrock at the site is a member of the Nepean Formation; it is weak to medium strong, and very thinly- to medium-bedded. Rock Quality Designation (RQD) values measured on recovered bedrock core samples ranged from 0 to 90 per cent (but typically 36 to 90 per cent) in the upper 0.5 m of the bedrock, and from 80 to 100 per cent in the lower 2.5 m of the recovered bedrock core. The typical RQD values indicate that the bedrock is generally of good to excellent quality. The discontinuities observed in the rock core are typically horizontal to sub-horizontal, associated with the bedding planes.

weathering

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

4.3 Groundwater Conditions

Two standpipe piezometers were installed within the overburden soil deposits at this site. The water levels were measured in the piezometers on January 8, 2003, and the observations are summarised in the following table:

Borehole No.	**Location**	**Water level on January 8, 2003**	
		Elevation (m)	**Depth (m)**
02-402	STA 12+650, 5 m north of centerline stake	84.3	9.7
02-404	STA 12+721, 5.3 m north of centerline stake	82.0	10.5

At the time of the 1966 foundation investigation, the groundwater levels were observed to be about 1 m to 3 m below the ground surface (or about Elevation 85.6 m to 82.9 m). It should be noted that groundwater levels are expected to fluctuate seasonally and are expected to rise during wet periods of the year.

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PART B

FOUNDATION DESIGN REPORT
CN RAIL OVERPASS WIDENING
STRUCTURE SITE 3-257 (EBL)
HIGHWAY 417
G.W.P. 458-98-00

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides foundation design recommendations for the proposed widening of the EBL CN Rail Bridge structure. The recommendations are based on interpretation of the factual geotechnical data obtained from the boreholes and CPTs advanced during the subsurface investigation at this site.

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

5.2 Bridge and Retaining Wall Foundation Options

— RSS vs Integral vs Semi Integral

no assumptions

It is understood that the proposed widening for the EBL structure will maintain the existing three-span, pre-cast girder construction supported on concrete abutments and piers (which are in turn supported on piled foundations). It is expected that the existing conventional earth-sloped embankments and abutment / wing walls configuration will be maintained and that other forms of earth retention (i.e. RSS walls) will not be required.

The firm to stiff silty clay found in this area is relatively compressible and makes shallow foundations impractical for this site. The proposed bridge widenings should therefore be supported on deep foundations, consistent with the existing bridge structures, which transfer the foundation loads to more competent bearing at depth.

Two options that could be considered for these foundations are:

- Steel H-piles driven to end-bearing on bedrock
- Cast-in-place concrete caissons

Recommendations for steel H-piles and cast-in-place concrete caissons for the bridge abutments are presented in the following sections. A summary comparison of the advantages, disadvantages, relative costs, and risks associated with both of these foundation options is presented in Table 1 following the text of this report.

5.3 Steel H-Pile Foundations

Steel H-piles driven to found on the sandstone bedrock may be used for support of the abutment widening. The following table assumes that the abutment widening will be constructed with the same underside of pile cap elevation as the existing abutment. All elevations have been inferred from the Preliminary General Arrangement drawing (Figure 6.70) by McCormick Rankin Corporation (MRC) dated August 2002.

Tip Elevat

Location	*Pile Cap Elevation (m)**	*Approximate Length of Pile (m)**
East Abutment	*89.5*	*19*
West Abutment	*90*	*19*
East Pier	*84.5*	*13.5*
West Pier	*84.5*	*13.5*

*Pile cap elevations and pile lengths will be updated after final General Arrangement drawing is provided.

5.3.1 Axial Geotechnical Resistance

For HP 310 x 110 piles driven to found on the sandstone bedrock, a factored axial resistance at Ultimate Limit States (ULS) of 2000 kN may be assumed for design. This value takes into account the structural capacity limitation of the pile, and potential difficulties that the pile may have seating into the bedrock surface that may be variable and inclined. A Serviceability Limit States (SLS) value is not provided because the sandstone 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.

Consideration must be given to the presence of cobbles and boulders within the relatively thin layer of glacial till which underlies the silty clay deposit at this site. The pile tips should be suitably reinforced with flange reinforcement for vertical piles. The thick sensitive clay deposits and very thin basal till layer may offer only limited resistance to sliding of battered piles along the bedrock surface; in this regard, battered piles should be fitted with Titus Ejector rock points or equivalent to ensure seating in the bedrock.

Pile installation should be in accordance with SP903S01. For this site, the piles will essentially be driven to practical refusal on the bedrock. The drawings should incorporate the appropriate note stating that the piles should be equipped with flange reinforcement and/or rock points (as discussed above) and be driven to bedrock.

The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile. All of these factors must be taken into consideration in establishing the driving criteria to ensure that the piles are not overdriven and to avoid possible damage to the piles. In this regard, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and then to gradually increase the energy over a series of blows to seat the pile.

5.3.2 Downdrag Load (Negative Skin Friction)

It should be noted that the widening of the existing approach embankments will necessitate some filling in the median area and result in an increase in the effective stress level in the thick silty clay deposit which underlies this site. The precise details of that filling (i.e. width and grade) are not known, but it is anticipated that the grade in the centre of the median (the lowest point) could be raised by about 1.1 m at the EBL east embankment and about 1.3 m at the EBL west embankment. The stress increase caused by the filling will lead to some compression of the deposit. However the bridge abutments will be located at the edge of the filling and there will therefore be a lesser stress increase beneath those locations. The consolidation settlement of the underlying clay deposits will be discussed in a later section.

The consolidation settlement is time-dependent and will not completely occur during the construction period. That is, post-construction settlement of the clay deposit will take place. Since the piles will be end-bearing on bedrock, a small amount of settlement of the clay relative to the pile will result in the development of negative skin friction on the piles. Therefore, negative skin friction or downdrag loads will need to be taken into account during design of the piles supporting the new abutments. In addition, settlement of the clay under the new embankment loads will impose new downdrag loads on the existing piles, through to a lesser degree depending upon proximity to the new fill areas.

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

For cohesionless soils

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

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

For cohesive soils

$q_n = \alpha \tau_u$ where
q_n is the unit negative skin friction (kN)
α is the reduction coefficient ranging from 0.5 to 1.0
τ_u is the undrained shear strength (kPa)

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

$\sigma_v' = \gamma' z$ where
γ' is the buoyant unit weight of soil (assume 11 kN/m^3)
Z is the depth below final pile cap elevation (kPa)

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

East Abutment

Soil Unit	τ_u	$\alpha\tau_u$
Firm to stiff weathered silty clay crust from Elevation 82.9 m to 78.9 m	80 kPa	35 kPa
Firm unweathered silty clay from Elevation 78.9 m to 70.3 m	50 kPa	35 kPa

West Abutment

Soil Unit	τ_u	$\alpha\tau_u$
Firm to stiff weathered silty clay crust from Elevation 88.2 m to 78.2 m	80 kPa	35 kPa
Firm unweathered silty clay from Elevation 78.2 m to 70.8 m	50 kPa	30 kPa

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

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

Location	Nominal (unfactored) Downdrag Load
East Abutment	600 kN
West Abutment	400 kN

No downdrag loads are anticipated for the piles at the piers given that no additional loading or settlement is anticipated at these locations.

It should be noted that the structural engineer needs to review the piles within the existing abutment foundation structures to determine whether there is sufficient capacity to carry the downdrag loads.

discussion on extent of influence

Alternatively, consideration could be given to installing EPS in the new fill area to offset any increase in the embankment loading and eliminate the need to consider downdrag forces on the new and existing piles.

detailed recs for options to deal with downdrag forces

5.3.3 Resistance to Lateral Loads

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

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

For cohesionless soils

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

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

For cohesive soils

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

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

The following ranges for the value of n_h and s_u may be assumed in the structural analysis:

East Abutment

Soil Deposit	n_h	s_u
Compact sand and gravel from Elevation 89.5 to 82.9 m	6.6 MPa/m	--
Firm to stiff weathered silty clay crust from Elevation 82.9 to 78.9 m	--	80 kPa
Firm unweathered silty clay from Elevation 78.9 to 70.3 m	--	50 kPa

West Abutment

Soil Deposit	n_h	s_u
Compact sand and gravel from Elevation 90 to 88.2 m	6.6 MPa/m	--
Firm to stiff weathered silty clay crust from Elevation 88.2 to 78.2 m	--	80 kPa
Firm unweathered silty clay from Elevation 78.2 to 70.8 m	--	50 kPa

The above tables have been prepared based on the assumption that the pile cap elevation for the abutment widenings will match that of the existing abutments, at an estimated elevation of 89 m and 90 m for the EBL east abutment and EBL west abutment respectively.

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than six to eight pile diameters. Group action can be evaluated by reducing the coefficient of lateral subgrade reaction in the direction of loading by a reduction factor as follows:

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

5.3.4 Frost Protection

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

5.4 Cast-in-Place Concrete Caissons

Cast-in-place concrete caissons could be used for support of the abutment widenings. The following table assumes that the abutment widening will be constructed with the same underside of pile cap elevation as the existing abutments. All elevations have been inferred from the

Caisson

Preliminary General Arrangement drawing (Figure 6.70) prepared by McCormick Rankin Corporation, dated August 2002.

Top Elevation

Location	*Pile Cap Elevation (m)**	*Approximate Length of Shaft (m)**
East Abutment	89.5	19
West Abutment	90.0	19
East Pier	83.5	13.5
West Pier	83.5	13.5

* Note: Pile cap elevations ~~and shaft~~ lengths will be updated after Final General Arrangement drawing is provided.

why are these elevations different from on page 13?

It is noted that the native marine (Champlain Sea) clay at this site is a rather sensitive soil. The disturbed clay could "flow" into the auger hole during drilled shaft installation if left unsupported. The use of a permanent casing will be required in order to advance the drilled shafts with minimal loss of ground. For caissons of this length, it should not be planned to remove the casing during concreting.

The sandstone bedrock at the site is considered to be generally weak to medium strong. If socketting of the drilled shafts into the bedrock is required, the sockets will have to be advanced by rock coring or churn drilling. Further, the sandstone bedrock is likely rather abrasive (due to its silica content) and could result in relatively high equipment wear.

Is socketing recommended

5.4.1 Axial Geotechnical Resistance

caissons and downdrag

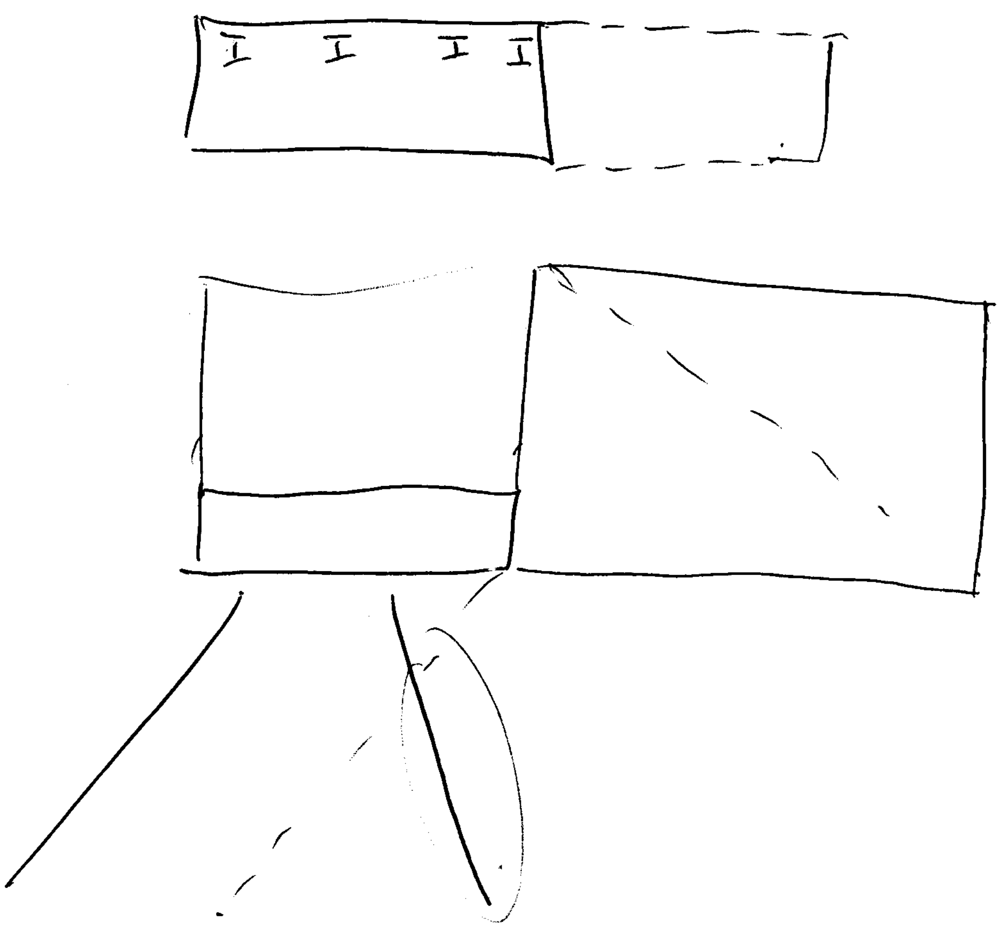
Caissons socketted nominally (0.3 m) into the bedrock should ~~be~~ designed based on end-bearing resistance, and a factored geotechnical resistance at ULS of 7 MPa should be used. Serviceability Limit States resistances do not apply, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

It should be noted that the caisson excavation must be cleaned using methods such as airlifting prior to concreting, and that tremie concreting techniques may be required for placing concrete.

will tremie methods be required?

5.4.2 Downdrag Load (Negative Skin Friction)

It should be noted that the widening of the existing approach embankments will result in some filling of the median area and an increase in the effective stress level in the thick silty clay deposit which underlies this site. Using the method described in Section 5.3.2, the estimated downdrag loads acting on a single caisson are summarised in the following table. The loads given are the estimated nominal (unfactored) downdrag loads acting on 1.5 m diameter caisson for the



structure.

Location	Nominal (unfactored) Downdrag Load
East Abutment	1,900 kN
West Abutment	1,600 kN

No downdrag loads are anticipated for caissons at the piers, given that no additional loading or settlement is anticipated at these locations. The downdrag loads provided in the above table are relatively large and may render the use of caisson foundations impractical. If caisson foundations are considered advantageous for other reasons, 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.

Detailed MTO

It should be noted that the structural engineer needs to review the piles within the existing abutment foundations to determine whether there is sufficient capacity to carry the downdrag loads as described in Section 5.3.2.

As discussed in Section 5.7.2, EPS could be employed in the embankment construction to eliminate downdrag loads.

5.4.3 Resistance to Lateral Loads

The resistance to lateral loading developed by the soils in front of the caissons, and the reductions due to group effects, may be determined as per Section 5.3.3. limitations on batter

5.4.4 Frost Protection

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

5.5 Site Coefficient

For seismic design purposes, the Site Coefficient, S, for this site in accordance with Section 4.4.6 of the CHBDC may be taken as 2.0, consistent with Soil Profile Type IV.

Recs on articulation of new vs existing.

5.6 Lateral Earth Pressures for Design

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

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

- Select free-draining granular fill meeting the specifications of Ontario Provincial Standard Specifications (OPSS) Granular 'A' or Granular 'B' but with less than 5 per cent passing the 200 sieve should be used as backfill behind the walls. This fill should be compacted in loose lifts not greater than 200 mm in thickness to 95 per cent of the material's Standard Proctor maximum dry density in accordance with OPSS 501. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Other aspects of the granular backfill requirements with respect to sub-drains and frost tapers should be in accordance with OPSD 3501.00 and 3504.00.

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with *CHBDC* ~~Section 6.9.3~~ and Figure 6.9.3. Compaction equipment should be used in accordance with OPSS 501.06. Other surcharge loadings should be accounted for in the design, as required.

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

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

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

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

	Granular 'A'	Granular 'B' Type II
Soil unit weight:	22 kN/m³	21 kN/m³
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.31
At rest, K_o	0.43	0.47

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

- Seismic loading will result in increased lateral earth pressures acting on the abutment stem and retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure. According to the National Building Code of Canada, this site is located in Seismic Zone 4. The site-specific zonal acceleration ratio for Ottawa is 0.2. Based on experience, for the subsurface conditions at this site, a 30 per cent amplification of the ground motion will occur, resulting in an increase in the ground surface acceleration from 0.2g to 0.26g. The seismic lateral earth pressure coefficients given below have been derived based on a design zonal acceleration ratio of A = 0.26.

- In accordance with Sections 4.6.4 and C.4.6.4 of the CHBDC and its Commentary, for structures which do not allow lateral yielding, the horizontal seismic coefficient, k_h, used in the calculation of the seismic active pressure coefficient, is taken as 1.5 times the zonal acceleration ratio (i.e. k_h = 0. 39). For structures which allow lateral yielding, k_h is taken as 0.5 times the zonal acceleration ratio (i.e. k_h = 0. 13). The seismic active earth pressure coefficient is also dependent on the vertical component of the earthquake acceleration, k_v. Three discrete values of vertical acceleration are typically selected for analysis, corresponding to $k_v = +2/3\ k_h$, $k_v = 0$, and $k_v = -2/3\ k_h$.

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

SEISMIC ACTIVE PRESSURE COEFFICIENTS, K_{AE}

	Case I	Case II	
		Granular A	Granular B Type II
Yielding wall	0.42	0.33	0.37
Non-yielding wall	1.17	0.84	0.96

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

- The above K_{AE} values for yielding walls are applicable provided that the wall can move up to 250A (mm), where A is the design zonal acceleration ratio of 0.26. This corresponds to displacements of up to 65 mm at this site.

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

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

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

5.7 Approach Embankment Design and Construction

The widening of the existing approach embankments into the median to accommodate the new lanes will require filling of the existing swale by about 1.1 to 1.3 m, assuming that the existing median swale side-slopes and shoulder widths will be approximately maintained.

Based on the borehole results, the embankment subgrade soils will consist of the existing embankment fill materials overlying a thick deposit of sensitive marine clay.

5.7.1 Approach Embankment Stability

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

Limit equilibrium slope stability analyses were performed using the commercially available program SLOPE/W, produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the factor of safety of numerous potential failure surfaces was computed in order to establish the minimum factor of safety. The factor of safety is defined as the ratio of the forces tending to resist the failure to the driving forces tending to cause failure. Target factors of safety of 1.3 and 1.1 are normally used for the design of embankment slopes under static conditions and seismic events, respectively. This factor of safety is considered adequate for the embankments at this site considering the design requirements and the field data available. The stability analyses were performed to check that the target minimum factor of safety is achieved for the proposed embankment widening.

For cohesive soils, total stress parameters were employed in the analyses. The total stress parameters (i.e. average mobilised undrained shear strength – s_u) for the cohesive soils were assessed based on the results of field vane tests and inferred from the oedometer (consolidation) test results.

A site-specific correlation was developed relating natural moisture content to undrained shear strength as measured with the field vane. This correlation was utilised to estimate shear strengths in the silty clay deposits. For oedometer tests, the following correlation proposed by Mesri (1975) was employed to estimate undrained shear strength:

$$s_u = 0.27\sigma_p'$$

where s_u is the average mobilised undrained shear strength (kPa)
σ_p' is the preconsolidation pressure (kPa)

The following equation was used to calculate the undrained shear strength profile of the silty clay from the results of the CPT (Lunne, 1997). It should be noted that the factor, N_k, is calculated based on the measured undrained shear strength from field vanes. The undrained shear strength profile is summarized on Figure 2.

$$s_u = \frac{q_t - \sigma_{vo}}{N_k}$$

where s_u is the average mobilised undrained shear strength (kPa)
q_t is the net tip resistance (kPa)
σ_{vo} is the total vertical stress (kPa)
N_k is the correlated factor calculated as 14 for this site

The CPT results indicate that the weathered silty clay crust has an undrained shear strength of about 200 kPa decreasing linearly to about 50 ~~kPa,~~ and that the unweathered silty clay has an undrained shear strength between 35 kPa and 70 kPa .

Soil Deposit	*Unit Weight (kN/m^3)*	*Effective Friction Angle*	*Undrained Shear Strength (kPa)*
Earth Fill	20	30°	--
Weathered Silty Clay Crust	17	--	80
Unweathered Grey Silty Clay	17	--	50

The stability for the proposed filling of the median was assessed based on precedent experience in similar soil conditions. The factor of safety against instability under static conditions would be in excess of 1.3, assuming 2H:1V side slopes and that all of the surface and near-surface organic layers have been removed prior to construction. Stability analyses undertaken using the above parameters and the assumed filling geometry indicate a factor of safety against instability under design seismic conditions in excess of 1.1. Based on the elevation of the groundwater table, and the assumed geometry of the slope and the type of fill being used, the liquefaction potential of the soils under seismic loading is considered low but it is possible that surficial sloughing would occur in localised areas.

Stability analyses were undertaken for the overall embankment using the above parameters extrapolated to and beyond the toe of the overall embankment. Based on this analysis, it has been determined that a factor of safety against instability for the overall embankments under static conditions is also in excess of 1.3.

It should be noted that the factor of safety against instability under seismic conditions for the overall embankment has not been analysed and/or calculated given that it is outside the terms of reference for this report.

Stability analyses were also undertaken for excavation of the west pier where the approximate loading from the rail traffic is included, based on the Cooper E90 locomotive axle loading. The loading considered is based on a line load of 40,820 kg (90,000 lbs) per axle, for 4 axles spaced 1.5 m apart. For analysis purposes, it is assumed that the axle loads are spread uniformly across the 2.5 m wide rail ties and are treated as a uniform surcharge of 65 kPa. Assuming 1H:1V side slopes for the excavation, the factor of safety against instability under static conditions is in excess of 1.3. Is this acceptable to two CN

Due to the space restrictions for the east pier, stability analyses were not carried out since this area will likely require temporary shoring as described in Section 5.9.2.

5.7.2 Approach Embankment Settlement

Settlement analyses were performed on the critical sections of the proposed embankment widening. For this report, critical sections are assumed to correspond to the areas where filling is greatest. The settlement analyses were carried out using the commercially available program UNISETTLE, produced by Unisoft Limited.

As discussed previously, the expected filling of the median swale will be relatively modest, assuming that the existing median side slopes as well as the shoulder and lane widths are approximately maintained. The expected settlement of the embankment in the area of filling has been assessed based on a maximum filling of about 1.1 m at the EBL east embankment, and about 1.3 m at the EBL west embankment.

Despite the relatively small amount of additional load that the proposed filling will apply to the underlying subgrade, the silty clay deposit which underlies this site is relatively compressible and it is normally consolidated at depth (i.e. it has not experienced loads in the past any greater than the current loads). Settlement analyses were carried out using the results of the borehole information, in situ field test data (field vane, SPT and CPT), and/or laboratory tests to estimate the deformation parameters of the subsoils.

Oedometer (consolidation) tests were not performed for this investigation. However, oedometer tests were performed on selected samples from the investigation for the "Tall Rock Fill" embankment widening adjacent to the bridge widening area. For the details of the lab test results, reference should be made to Golder Report No. 021-1155-12. The results of those oedometer tests were used to calibrate the calculations of the preconsolidation pressure profiles from the CPT results.

The over-consolidation ratio (OCR) profile required in the analyses was established using correlations with the results of the in situ vane tests, CPT tests, and oedometer test results from the Tall Rock Fill Embankment Report. The following correlation proposed by Mesri (1975) was employed to estimate preconsolidation pressure using average mobilised undrained shear strength:

$$s_u = 0.27\sigma_p'$$

where s_u is the average mobilised undrained shear strength (kPa)
σ_p' is the preconsolidation pressure (kPa)

The following correlation proposed by Demer and Leroueil (2002) was employed to estimate preconsolidation pressure using net tip resistance, $q_t - \sigma_{vo}$, from the CPT:

$$\sigma_p' = \frac{q_t - \sigma_{vo}}{3.4}$$ where σ_p' is the preconsolidation pressure (kPa)
q_t is the corrected tip resistance (kPa)
σ_{vo}' is the in-situ vertical effective stress (kPa)

The calculated profile of preconsolidation pressure below natural ground surface is shown on Figure 2. The CPT results indicate a preconsolidation pressure of about 800 kPa decreasing linearly to about 200 to 300 kPa for the weathered silty clay deposit; below this, the preconsolidation pressure is constant with depth between 200 to 300 kPa for the unweathered silty clay. $p_c - p_o$

The piezometric conditions used in the analyses were based on the groundwater levels noted during drilling and measured in standpipe installations. In general, the groundwater level is located at about the elevation of the natural ground surface.

It should be noted that all settlement analyses assume that all of the surface and near-surface organic layers have been removed prior to construction.

The following table summarises the simplified stratigraphy and the unit weight and deformation parameters employed for the different layers for each of the abutment areas. The maximum estimated settlement of the foundation soils in these areas is presented, and a discussion on the rate of settlement is provided.

East Abutment

Soil Deposit	Thickness (m)	Unit Weight (kN/m^3)	OCR*	e_o*	C_r*	C_c*
Weathered Silty Clay Crust	1.0	19	4.2 / 4.1	0.6 / 0.8	0.010 / 0.014	0.40 / 0.49
Weathered Silty Clay Crust	3.0	19	1.0 / 1.0	0.8 / 1.4	0.014 / 0.025	0.49 / 0.75
Unweathered Silty Clay	8.6	19	1.0 / 1.0	1.4 / 1.2	0.025 / 0.025	0.75 / 0.75

*The numbers represent the values used at the top and bottom of the deposit respectively, with values between them interpolated linearly.

West Abutment

Soil Deposit	Thickness (m)	Unit Weight (kN/m^3)	OCR*	e_o*	C_r*	C_c*
Weathered Silty Clay Crust	1.0	19	7.0 / 6.5	0.6 / 0.8	0.010 / 0.014	0.40 / 0.49
Weathered Silty Clay Crust	3.0	19	6.5 / 1.4	0.8 / 1.4	0.014 / 0.025	0.49 / 0.75
Unweathered Silty Clay	7.8	19	1.4 / 1.0	1.4 / 1.3	0.025 / 0.025	0.75 / 0.75
Unweathered Silty Clay	5.6	19	1.0 / 1.0	1.3 / 1.2	0.025 / 0.025	0.75 / 0.75

*The numbers represent the values used at the top and bottom of the deposit respectively, with values between them interpolated linearly.

A coefficient of consolidation (c_v) of 1.4 x 10-3 (cm^2/s) has been used to represent the consolidation behaviour of the ground mass beneath the site, based on the empirical correlation found in US Navy (1971) for normally consolidated clays based on the average value of liquid limits. Values from the consolidation tests are considered to be rather low based on engineering judgement, considering experience in similar soil conditions, and represent $c_{v\,r}$ values of only the particular samples tested.

Based on the results of the settlement analyses, the maximum primary settlement of the foundation soils and the estimated time for 90 per cent of the settlement to occur, assuming two-way drainage, is summarised below. The settlement of the cohesionless soils (i.e. earth fill) will occur immediately after filling has completed and it is not included in the primary settlement results.

Location	Thickness of Silty Clay Deposit	Primary Settlement*	Estimated Time to 90 per cent Settlement
East Abutment	12.6 m	25 mm	8 years
West Abutment	17.4 m	35 mm	15 years

*Primary settlement estimates correspond to the areas beneath the centre median swale, where the fill thickness will be greatest.

The time for 90 per cent of the settlement to occur is relatively long for the modest amount of fill, and is due to the thick silty clay deposit and low coefficient of consolidation. It should be noted that details are not available on the actual geometry of the filling and therefore the above estimates are approximate. The estimates could be refined once details of the proposed filling are available but, given the settlement magnitudes, such refinement may not be waranted. No primary settlement is anticipated at the pier given that no increase in net loading is anticipated at these locations.

Secondary (creep) settlement has also been analysed, using the correlation proposed by Mesri (1973) based on natural water content. The results of the secondary settlement analyses are summarised below. Secondary settlement will occur once 90 per cent of the primary settlement has been completed.

Location	Thickness of Silty Clay Deposit	Secondary Settlement per log cycle*	Time for Secondary Compression to Occur
East Abutment	12.6 m	45 mm	8 to 80 years
West Abutment	17.4 m	65 mm	15 to 150 years

*Secondary settlement estimates correspond to the areas beneath the centre median swale, where the fill thickness will be greatest.

Options – preload / surcharge, EPS, LWF

If EPS of sufficient thickness is employed in embankment construction, the primary settlement, and to some extent the secondary consolidation, would be eliminated.

5.8 Subgrade Preparation and Approach Embankment Construction

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

Embankment fill should be placed in regular lifts with loose thickness not exceeding 300 mm, and be compacted to at least 95 per cent of the material's Standard Proctor maximum dry density. The final lift prior to placement of the granular subbase and base courses should be compacted to 100 per cent of the Standard Proctor maximum dry density. Inspection and field density testing should be carried out by qualified personnel during placement operations to ensure that appropriate materials are used and that adequate levels of compaction have been achieved.

In order to minimise differential settlement between the widened portion of Highway 417 and the existing embankments, the use of granular fill is recommended for the widening. The majority of settlement of granular fills will occur during construction whereas the majority of settlement of cohesive fills, if used, would occur post-construction. In addition, keying of the new embankment fill into the existing side slopes along the eastbound and westbound lanes of

Highway 403 would help to reduce the affects of differential settlement. The new embankment fills should be benched into the existing embankment in accordance with OPSD 208.010.

5.9 Design and Construction Considerations

5.9.1 Excavation

It is assumed that the abutment widenings will be constructed with the same underside of pile cap level as the existing abutments as describe in Section 5.3. The excavations will extend through the existing embankment fills for the abutment widening and through the weathered silty clay for the pier widening.

Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities. These soils are classified as Type 3 soils according to the OHSA and therefore excavations for the east abutment, west abutment and the west pier should be made with side slope no steeper than 1 horizontal to 1 vertical. However, due to the proximity of the railway tracks and retaining wall at the east pier, temporary shoring will be required as discussed in Section 5.9.2.

5.9.2 Temporary Shoring

Due to the proximity of the rail tracks and the retaining wall at the east pier, there is insufficient distance to satisfy the 1H:1V side slopes requirement for temporary excavations, and therefore a support system will be required to minimise the movement of the railroad track and to maintain the stability of the existing highway embankment. The support system would be located at the edge of the railroad track ballast and at the toe of the existing highway embankment. In addition, it is expected that temporary shoring will be necessary for the construction of the abutments.

The temporary excavation support should be in accordance with MTO Special Provision 539S01 (Roadway Protection). The temporary shoring system should be designed to Performance Level 1 as defined in SP 539S01.

Discussion of some alternatives

5.9.3 Groundwater and Surface Water Control

Given that the abutment pile cap elevation is above the measured groundwater level, it is anticipated that the excavations at the east and west abutments will not experience significant groundwater inflows. However, a small amount of groundwater flow is expected for the east and west pier excavations, and it is anticipated that adequate groundwater control can be achieved by pumping from properly filtered sumps in the excavation.

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

5.9.4 Obstructions

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

A thin layer of basal silty sand till was encountered beneath the silty clay deposit at this site and this material is expected to contain cobbles and boulders that may affect the seating of piles and caissons. Provision will have to be made in the Contract Documents to ensure that the Contractor is equipped to handle such obstructions.

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PROVINCE OF ONTARIO

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CN/JPD/SJB/FJH/sm
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REFERENCES

Belanger, J.R. 1998. Urban Geology of Canada's National Capital Area Urban Geology of Canadian Cities, Geological Association of Canada Special Paper 42, Ed. P.F. Karrow and O.L. White.

Briaud, Jean-Louis and Tucker, Larry M. 1994. Design and Construction Manual for Downdrag on Uncoated and Bitumen-Coated Piles, National Cooperative Highway Research Program.

Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA-S6-00. 2001. CSA Special Publication, S6.1-00. Canadian Standards Association.

Canadian Foundation Engineering Manual (CFEM). 1992. 3rd Edition. Canadian Geotechnical Society.

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

Demer, D and Leroueil, S. 2002. Evaluation of Preconsolidation Pressure and OCR from Piezocone Test of Clay Deposits in Quebec. Canadian Geotechnical Journal. Vol 39. pp. 174-192.

Lunne, T. et al. 1997. Cone Penetration Testing in Geotechnical Practice. Ch. 5, pp. 65.

Mesri, G. 1973. "Coefficient of Secondary Compression," Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 99, No. SM1, pp. 123-137.

Mesri, G. 1975. Discussion on "New Design Procedure for Stability of Soft Clays," Journal of the Geotechnical Engineering Division, ASCE, Vol. 101, No. GT4, pp. 409-412.

U.S. Navy, 1971. Soil Mechanics, Foundations and Earth Structures. NAVFAC Design Manual DM-7, Washington, D.C.

TABLE 1
COMPARISON OF FOUNDATION ALTERNATIVES
CN RAIL OVERPASS STRUCTURE WIDENING

Foundation Option	*Feasibility*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks/Consequences*
Spread footings supported on native silty clay soils	• Not feasible	*N/A*	*N/A*	*N/A*	*N/A*
Steel H-pile foundations founded on or socketted into bedrock	• Feasible for support of all foundation elements	• High bearing resistance • Negligible settlement • Site conditions appropriate for use of integral abutments	• Possibility of encountering cobbles or boulders during pile driving • Negative skin friction ("down drag") loads must be considered in design.	• May be less expensive than rock-socketted caissons option	• If required for pile toe fixity, socketting into the medium strong bedrock could be difficult and time-consuming
Cast-in-place concrete caissons founded or socketted into rock	• Feasible for support of all foundation elements	• High resistance • Negligible settlement	• Permanent casings required to construct caissons. • Possibility of encountering cobbles or boulders during drilled shaft installation • If rock socket required, coring or churn drilling will be required to form socket in medium strong bedrock • Negative skin friction ("down drag") loads must be considered in design and these may be relatively high compared to those for steel H-piles.	• May be more expensive than steel H-pile option if rock sockets are necessary	• If required for pile toe fixity, socketting into the medium strong bedrock could be difficult and time-consuming

Golder Associates

RECORD OF BOREHOLE No CPT02-401

METRIC

PROJECT 021-1155-5100

W.P. 458-98-00

DIST 9 HWY 417

DATUM Geodetic

LOCATION N 5021560.1 ;E 355216.3

BOREHOLE TYPE CME 55 Bombardier

DATE December 02 and 06, 2002

ORIGINATED BY PAH

COMPILED BY CN

CHECKED BY JPD

SOIL PROFILE: ELEV / DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
93.0	Ground Surface										
0.0	Sand and gravel, cobbles and boulders (ROCK FILL) Brown Moist						92 91 90 89				
88.9 4.1	Silty Clay Stiff Grey brown Moist		1	SS	12						
88.2 4.9	End of Borehole Cone Penetration Test from 4.88m depth For details refer to Cone Penetration Test CPT02-401										

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

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

MISS_MTO 0211155-5100-MTO.GPJ ON_MOT.GDT 24/8/04

Golder Associates

RECORD OF BOREHOLE No 02-402

METRIC

PROJECT 021-1155-5100			
W.P. 458-98-00	LOCATION N 5021574.1 ;E 355243.2		ORIGINATED BY PAH
DIST 9	HWY 417	BOREHOLE TYPE CME 55 Bombardier	COMPILED BY CN
DATUM Geodetic	DATE November 27 to 29, 2002		CHECKED BY JPD

SOIL PROFILE / SAMPLES / GROUND WATER CONDITIONS / ELEVATION SCALE / DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED (20 40 60 80 100) / PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) (25 50 75) / UNIT WEIGHT γ kN/m³ / REMARKS & GRAIN SIZE DISTRIBUTION (%)

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
94.0	Ground Surface							
0.0	Topsoil							
0.2	Silty clay, trace sand and gravel and organics (FILL) Grey brown Moist	1	AS	-				
93.3 / 0.8	Sand and gravel, containing cobbles (FILL) Compact Brown Moist	2	AS	-	49	34	10	7
		3	SS	17				
		4	SS	12				
88.2 / 5.8	Silty Clay, containing silty sand and clayey silt seams Firm to stiff Grey brown Moist to wet	5	SS	14				
		6	SS	10				
		7	SS	4				
		8	SS	4				
84.3 / 9.7								

Elevation scale: 93, 92, 91, 90, 89, 88, 87, 86, 85

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No 02-402

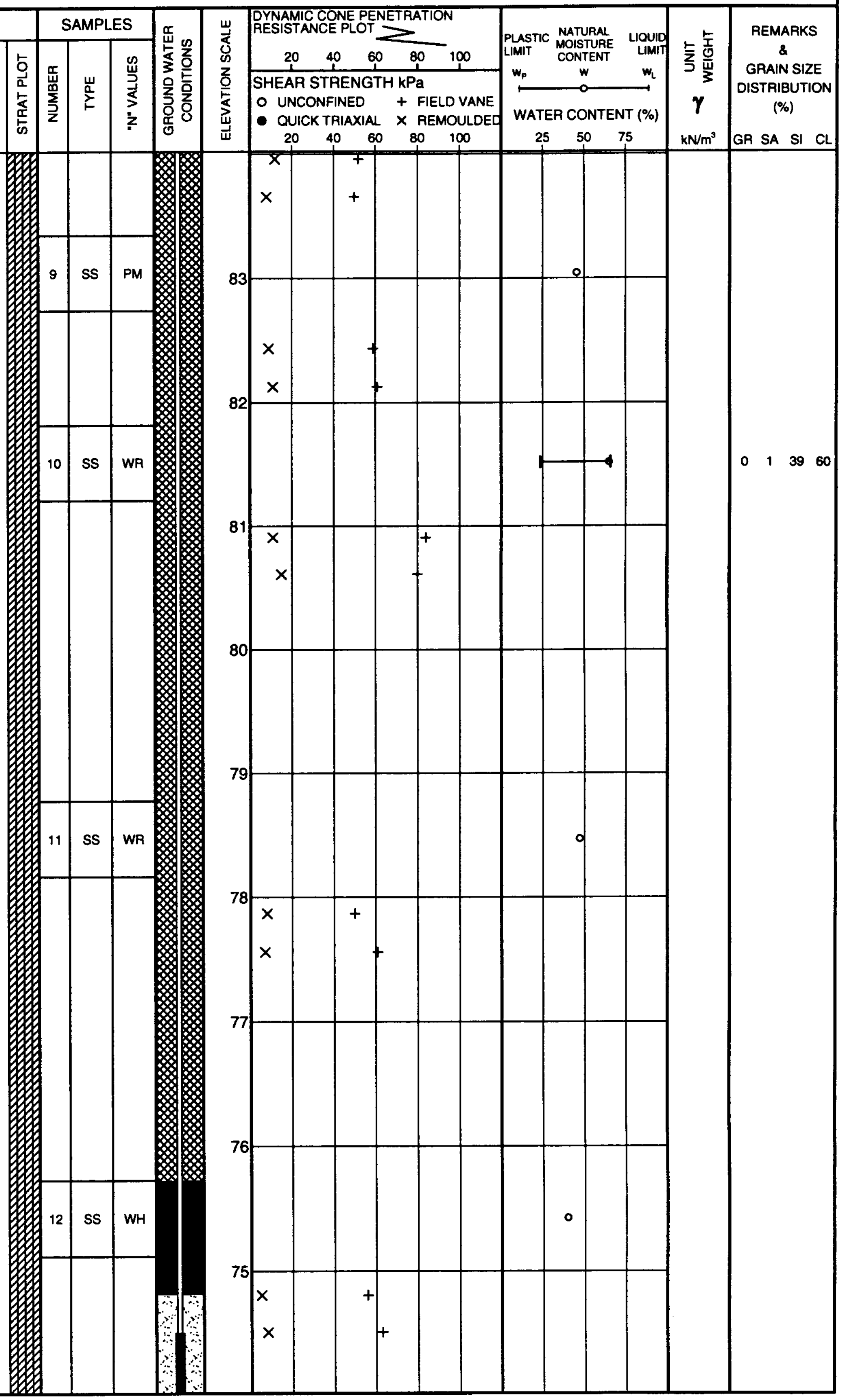
METRIC

PROJECT 021-1155-5100
W.P. 458-98-00
DIST 9 HWY 417
DATUM Geodetic
LOCATION N 5021574.1 ;E 355243.2
BOREHOLE TYPE CME 55 Bombardier
DATE November 27 to 29, 2002
ORIGINATED BY PAH
COMPILED BY CN
CHECKED BY JPD

--- CONTINUED FROM PREVIOUS PAGE ---

SOIL PROFILE: Silty Clay to Clay, occasional silty sand and clayey silt seams
Firm to stiff
Grey brown
Moist to wet

SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
9	SS	PM				
10	SS	WR	0	1	39	60
11	SS	WR				
12	SS	WH				



Continued Next Page

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

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

MISS_MTO 0211155-5100-MTO.GPJ ON_MOT.GDT 24/8/04

Golder Associates

RECORD OF BOREHOLE No 02-402

METRIC

PROJECT 021-1155-5100			
W.P. 458-98-00	LOCATION N 5021574.1 ;E 355243.2	ORIGINATED BY PAH	
DIST 9	HWY 417	BOREHOLE TYPE CME 55 Bombardier	COMPILED BY CN
DATUM Geodetic	DATE November 27 to 29, 2002	CHECKED BY JPD	

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

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES
	— *CONTINUED FROM PREVIOUS PAGE* —				
	Silty Clay to Clay, occasional silty sand and clayey silt seams Firm to stiff Grey brown Moist to wet		13	SS	WR
71.0	Inferred Silty Sand and Gravel (TILL)				
23.2	Sandstone (BEDROCK) Fresh Thinly to medium bedded Fine to medium grained Light grey Bedrock cored between 23.2 m and 26.1 m depth For bedrock coring details refer to Record of Drillhole 02-402				
67.9 26.1	End of Borehole Note: Water level in piezometer at 9.7 m depth on January 8, 2003				

Elevation scale: 73, 72, 71, 70, 69, 68

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

MISS_MTO 0211155-5100-MTO.GPJ ON_MOT.GDT 24/8/04

RECORD OF DRILLHOLE: 02-402

PROJECT: 021-1155-5100

SHEET 1 OF 1

LOCATION: N 5021574.1; E 355243.2

DRILLING DATE: November 27 to 29, 2002

DATUM: Geodetic

DRILL RIG: CME 55

INCLINATION: -90° AZIMUTH: ---

DRILLING CONTRACTOR: Marathon Drilling

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; MB - Mechanical Break; BR - Broken Rock

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. DEPTH (m)	RUN No.	FLUSH % RETURN	NOTES WATER LEVELS INSTRUMENTATION
		Continue from previous page	70.85			
	Rotary Drill NQ Core	Sandstone (BEDROCK) Fresh Thinly to medium bedded Fine to medium grained Light grey	23.17	1	100	
24				2	0	
25				3	0	
26				4	0	
			67.88			
		End of Drillhole	26.14			
27						
28						
29						
30						
31						
32						
33						

Other columns: SYMBOLIC LOG; PENETRATION RATE (m/min); COLOUR; RECOVERY (TOTAL CORE %, SOLID CORE %); R.Q.D. %; FRACT. INDEX PER 1m; DISCONTINUITY DATA (B Angle, DIP w.r.t. CORE AXIS, TYPE AND SURFACE DESCRIPTION); HYDRAULIC CONDUCTIVITY K, cm/sec (10^{-6} 10^{-5} 10^{-4} 10^{-3}); Diametral Point Load Index (MPa); RMC -Q' AVG.

Foundation Design

Golder Associates

RECORD OF BOREHOLE No 02-403

METRIC

PROJECT 021-1155-5100

W.P. 458-98-00

DIST 9 HWY 417

DATUM Geodetic

LOCATION N 5021592.4 ;E 355264.0

BOREHOLE TYPE CME 55 Bombardier

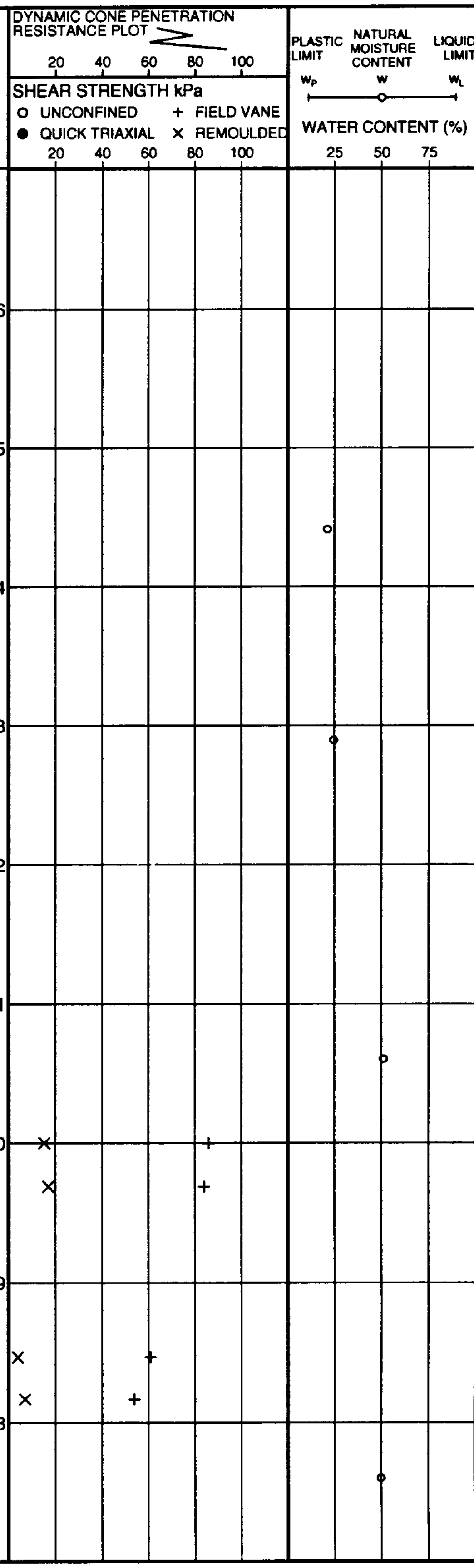
DATE November 25, 2002

ORIGINATED BY DB / PAH

COMPILED BY CN

CHECKED BY JPD

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES
87.0	Ground Surface			
0.0	Sand and gravel (FILL) Compact Brown Moist			
		1	SS	19
85.5 / 1.5	Clayey Silt and Sandy Silt, trace to some clay Firm to very stiff Grey brown Moist to wet	2	SS	19
		3	SS	10
		4	SS	4
		5	SS	4
82.6 / 4.4	Silty Clay Firm to stiff Grey brown Wet	6	SS	5
		7	SS	4
		8	SS	2
79.4 / 7.6	Silty Clay, containing silt seams Firm to stiff Grey Wet	9	SS	WH
		10	SS	WH



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

Continued Next Page

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

MISS_MTO 0211155-5100-MTO.GPJ ON_MOT.GDT 24/8/04

Golder Associates

RECORD OF BOREHOLE No 02-403

 METRIC

PROJECT 021-1155-5100

W.P. 458-98-00 LOCATION N 5021592.4 ;E 355264.0 ORIGINATED BY DB / PAH

DIST 9 HWY 417 BOREHOLE TYPE CME 55 Bombardier COMPILED BY CN

DATUM Geodetic DATE November 25, 2002 CHECKED BY JPD

ELEV DEPTH	DESCRIPTION (— CONTINUED FROM PREVIOUS PAGE —)	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
	Silty Clay, containing silt seams Firm to stiff Grey Wet	11	SS	WH	76
					75
					74
		12	SS	WH	73
					72
					71
70.9 16.2	Silty Sand and Gravel (TILL) Dense Grey Wet	13	SS	10/0.03	
70.2 16.8	Sandstone (BEDROCK) Fresh to slightly weathered Very thinly to medium bedded Fine to medium grained Light grey Bedrock cored between 16.8 m and 19.7 m depth For bedrock coring details refer to Record of Drillhole 02-403				70 69 68
67.3 19.7	End of Borehole				

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

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

MISS_MTO 0211155-5100-MTO.GPJ ON_MOT.GDT 24/8/04

RECORD OF DRILLHOLE: 02-403

PROJECT: 021-1155-5100
LOCATION: N 5021592.4; E 355264.0
INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 25, 2002
DRILL RIG: CME 55
DRILLING CONTRACTOR: Marathon Drilling

SHEET 1 OF 1
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, MB - Mechanical Break, BR - Broken Rock

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

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	ELEV. / DEPTH (m)	RUN No.	FLUSH % RETURN	RECOVERY	R.Q.D. %	FRACT. INDEX PER 1m	DISCONTINUITY DATA	HYDRAULIC CONDUCTIVITY K, cm/sec	Diametral Point Load Index (MPa)	RMC -Q' AVG.	NOTES WATER LEVELS INSTRUMENTATION
		Continue from previous page	70.24										
17–19	Rotary Drill, NQ Core	Sandstone (BEDROCK) Fresh to slightly weathered Very thinly to medium bedded Fine to medium grained Light grey	16.77	1	100								
				2	100								
			67.29										
		End of Borehole	19.72										

Depth scale: 17, 18, 19, 20, 21, 22, 23, 24, 25, 26

DEPTH SCALE
1 : 50

Golder Associates

LOGGED: P.A.H.
CHECKED: ---------------

MISS-ROCK-2 021-1155-5100-ROCK.GPJ GAL-CANADA.GDT 24/8/04

Golder Associates

RECORD OF BOREHOLE No 02-404

METRIC

PROJECT 021-1155-5100

W.P. 458-98-00

DIST 9 HWY 417

DATUM Geodetic

LOCATION N 5021601.8 ;E 355290.1

BOREHOLE TYPE CME 55 Bombardier

DATE November 26, 2002

ORIGINATED BY DB

COMPILED BY CN

CHECKED BY JPD

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	NATURAL MOISTURE CONTENT	GR	SA	SI	CL
92.5	Ground Surface									
0.0	Topsoil									
92.3 / 0.3	Sandy silt, trace clay (FILL) Very loose Grey brown Moist				92					
		1	SS	2	91	o				
89.3 / 3.2	Sand and gravel to sand, trace to some silt and gravel, containing cobbles (FILL) Loose to compact Brown Moist	2	SS	6	90, 89					
		3	SS	16	88, 87					
		4	SS	7	86	o	6	80	12	2
		5	SS	16	85, 84					
		6	SS	10	83					
82.9 / 9.6										

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 25 50 75

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

GROUND WATER CONDITIONS

Continued Next Page

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

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

Golder Associates

RECORD OF BOREHOLE No 02-404

METRIC

PROJECT 021-1155-5100
W.P. 458-98-00
DIST 9 HWY 417
DATUM Geodetic

LOCATION N 5021601.8 ;E 355290.1
BOREHOLE TYPE CME 55 Bombardier
DATE November 26, 2002

ORIGINATED BY DB
COMPILED BY CN
CHECKED BY JPD

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES
	--- CONTINUED FROM PREVIOUS PAGE ---			
	Clayey Silt to Clay, Firm to stiff, Grey brown, Moist	7	SS	13
		8	SS	6
		9	SS	8
		10	SS	6
79.6 / 13.0	Silty Clay, containing silt seams, Firm to stiff, Grey-brown to Grey, Wet	11	SS	2
		12	SS	2
		13	SS	WH
		14	SS	PM
		15	SS	WR

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (82, 81, 80, 79, 78, 77, 76, 75, 74, 73); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (25, 50, 75); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No 02-404

METRIC

PROJECT 021-1155-5100			
W.P. 458-98-00	LOCATION N 5021601.8 ;E 355290.1		ORIGINATED BY DB
DIST 9	HWY 417	BOREHOLE TYPE CME 55 Bombardier	COMPILED BY CN
DATUM Geodetic	DATE November 26, 2002		CHECKED BY JPD

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

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES
	Silty Clay, containing silt seams Firm to stiff Grey-brown to Grey Wet	16	SS	WR
70.9 / 21.6	Silty Sand and Gravel (TILL) Compact Grey Wet	17	SS	10/.18
70.3 / 22.3	Sandstone (BEDROCK) Fresh to slightly weathered Very thinly to medium bedded Fine to medium grained Light grey Bedrock cored between 22.3 m and 26.0 m depth For bedrock coring details refer to Record of Drillhole 02-404			
66.6 / 26.0	End of Borehole Note: Water level in piezometer at 10.5 m depth on January 8, 2003			

Elevation scale: 72, 71, 70, 69, 68, 67

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

MISS_MTO 0211155-5100-MTO.GPJ ON_MOT.GDT 24/8/04

RECORD OF DRILLHOLE: 02-404

PROJECT: 021-1155-5100

LOCATION: N 5021601.8; E 355290.1

INCLINATION: -90° AZIMUTH: ---

DRILLING DATE: November 27, 2002

DRILL RIG: CME 55

DRILLING CONTRACTOR: Marathon Drilling

SHEET 1 OF 1

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, MB - Mechanical Break, 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.	FLUSH COLOUR % RETURN	NOTES WATER LEVELS INSTRUMENTATION
		Continue from previous page		70.24			
	Rotary Drill, NQ Core	Sandstone (BEDROCK) Fresh to slightly weathered Very thinly to medium bedded Fine to medium grained Light grey		22.28	1	100	
23					2	100	
24							
25					3	100	
26		End of Drillhole		66.55 25.97			
27							
28							
29							
30							
31							
32							

DEPTH SCALE

1 : 50

Golder Associates

LOGGED: D.B.

CHECKED: ---------------

MISS-ROCK-2 021-1155-5100-ROCK.GPJ GAL-CANADA.GDT 24/8/04

Golder Associates

RECORD OF BOREHOLE No CPT02-405

METRIC

PROJECT 021-1155-5100

W.P. 458-98-00 LOCATION N 5021613.4 ;E 355303.0 ORIGINATED BY DB

DIST 9 HWY 417 BOREHOLE TYPE CME 55 Bombardier COMPILED BY CN

DATUM Geodetic DATE November 29, 2002 CHECKED BY JPD

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) (25 50 75)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
90.7	Ground Surface										
0.0	Sand and gravel, containing cobbles and boulders (ROCK FILL) Moist Brown						90				
							89				
							88				
							87				
							86				
							85				
							84				
							83				
							82				
							81				

Continued Next Page

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

Golder Associates

PROJECT 021-1155-5100	RECORD OF BOREHOLE No CPT02-405	2 OF 2	METRIC
W.P. 458-98-00	LOCATION N 5021613.4 ;E 355303.0		ORIGINATED BY DB
DIST 9 HWY 417	BOREHOLE TYPE CME 55 Bombardier		COMPILED BY CN
DATUM Geodetic	DATE November 29, 2002		CHECKED BY JPD

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L / WATER CONTENT (%) (25 50 75)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	— CONTINUED FROM PREVIOUS PAGE —										
80.9 10.1	Silty Clay Firm to stiff Brown										
							80				
			1	SS	PH						
							79				
78.8 11.9	End of Borehole Cone Penetration Test from 11.89m depth For details refer to Cone Penetration Test CPT02-405										

MISS_MTO 0211155-5100-MTO.GPJ ON_MOT.GDT 24/8/04

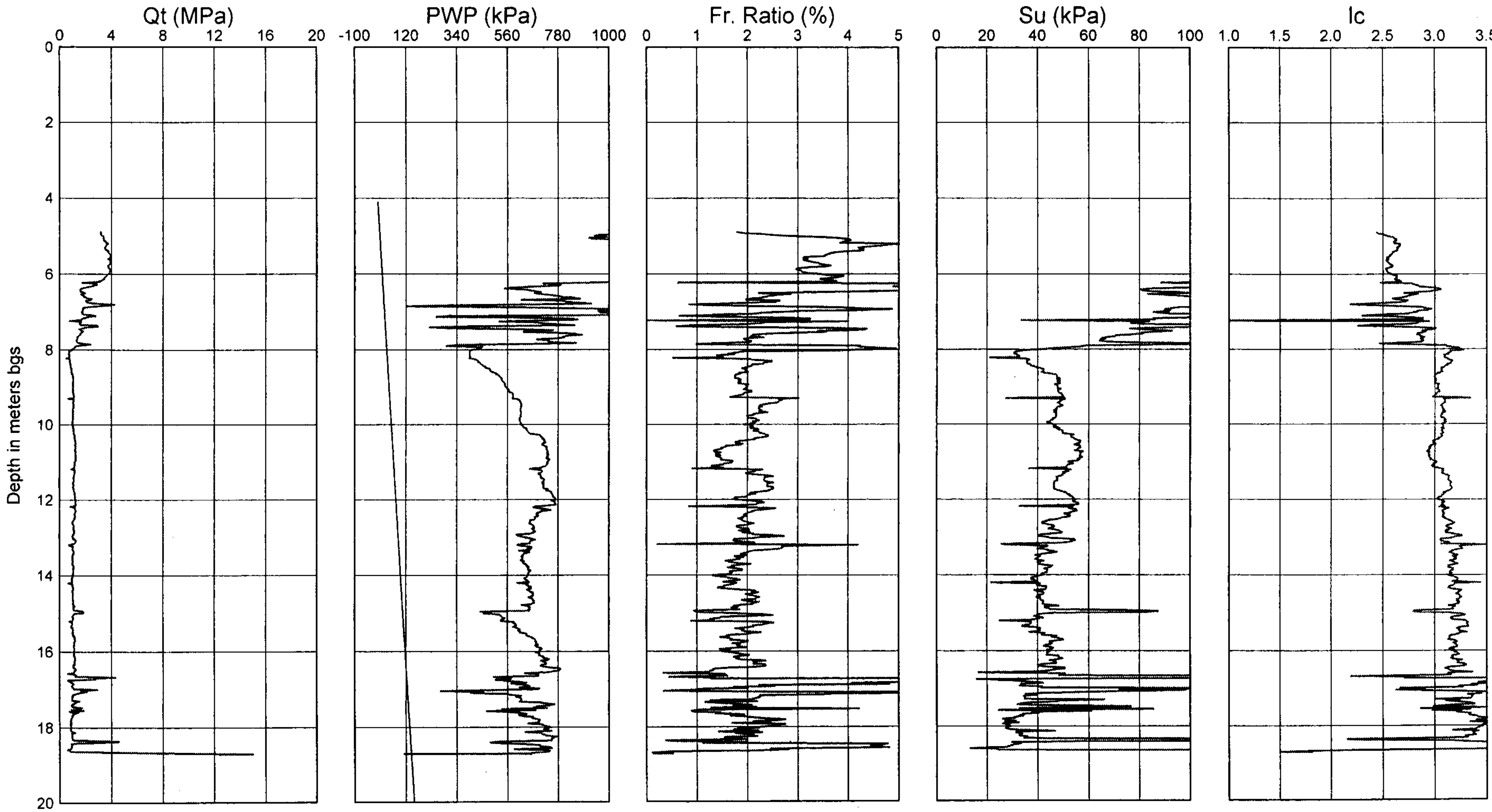
$+^3, \times^3$: Numbers refer to Sensitivity $\circ^{3\%}$ STRAIN AT FAILURE

Cone Penetration Test - CPT02-401

Test Date : Dec 06, 2002
Location : West Side of CN Rail Overpass

Operator : Golder Associates

Ground Surf. Elev. : 93.00
Water Table Depth : 4.10



Qt normalized for
unequal end area effects

Fr Ratio = 100*F/(Qt-Sigmav)
Gamma = 19 kN/m³

Su = (Qt - SigmaV) / Nk
Nk = 18
Gamma = 19 kN/m³

After Robertson and (Fear) Wride (1998)
Ic < 1.31 - Gravelly sands
1.31 <Ic < 2.05 - Clean to silty sand
2.05 <Ic < 2.60 - Silty sand to sandy silt
2.60 <Ic < 2.95 - Clayey silt to silty clay
2.95 <Ic < 3.60 - Clays

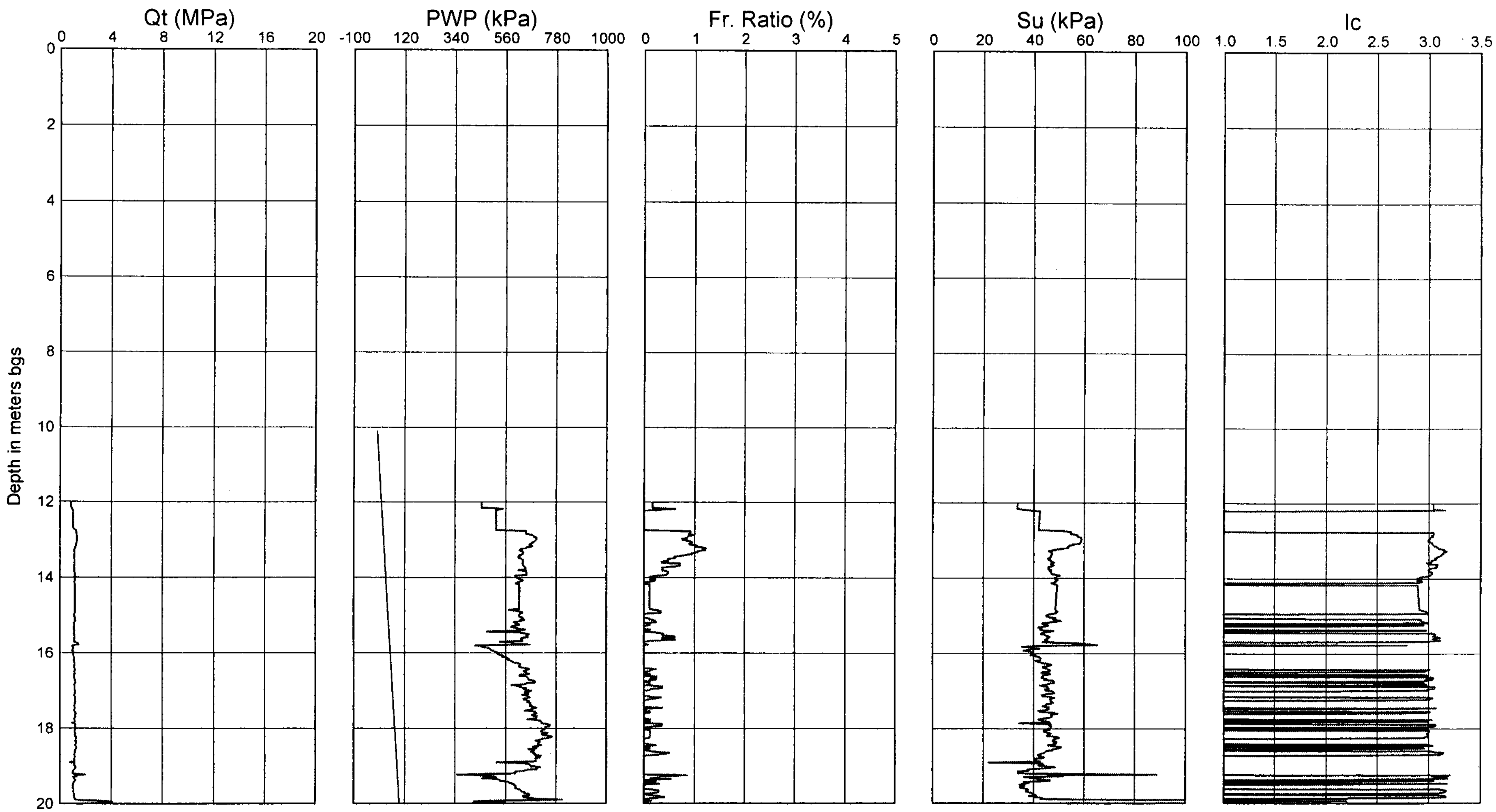
PROJECT NO. 21-1155-5100 DATE: Mar 10, 2004 DRAWN BY: CN

Cone Penetration Test - CPT02-405

Test Date : Nov 29, 2002
Location : East Side of CN Rail Overpass

Operator : Golder Associates

Ground Surf. Elev. : 90.70
Water Table Depth : 10.10



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PROJECT NO. 021-1155-5100 DATE: Mar 10, 2004 DRAWN BY: CN

Cone Penetration Test - CPT02-405

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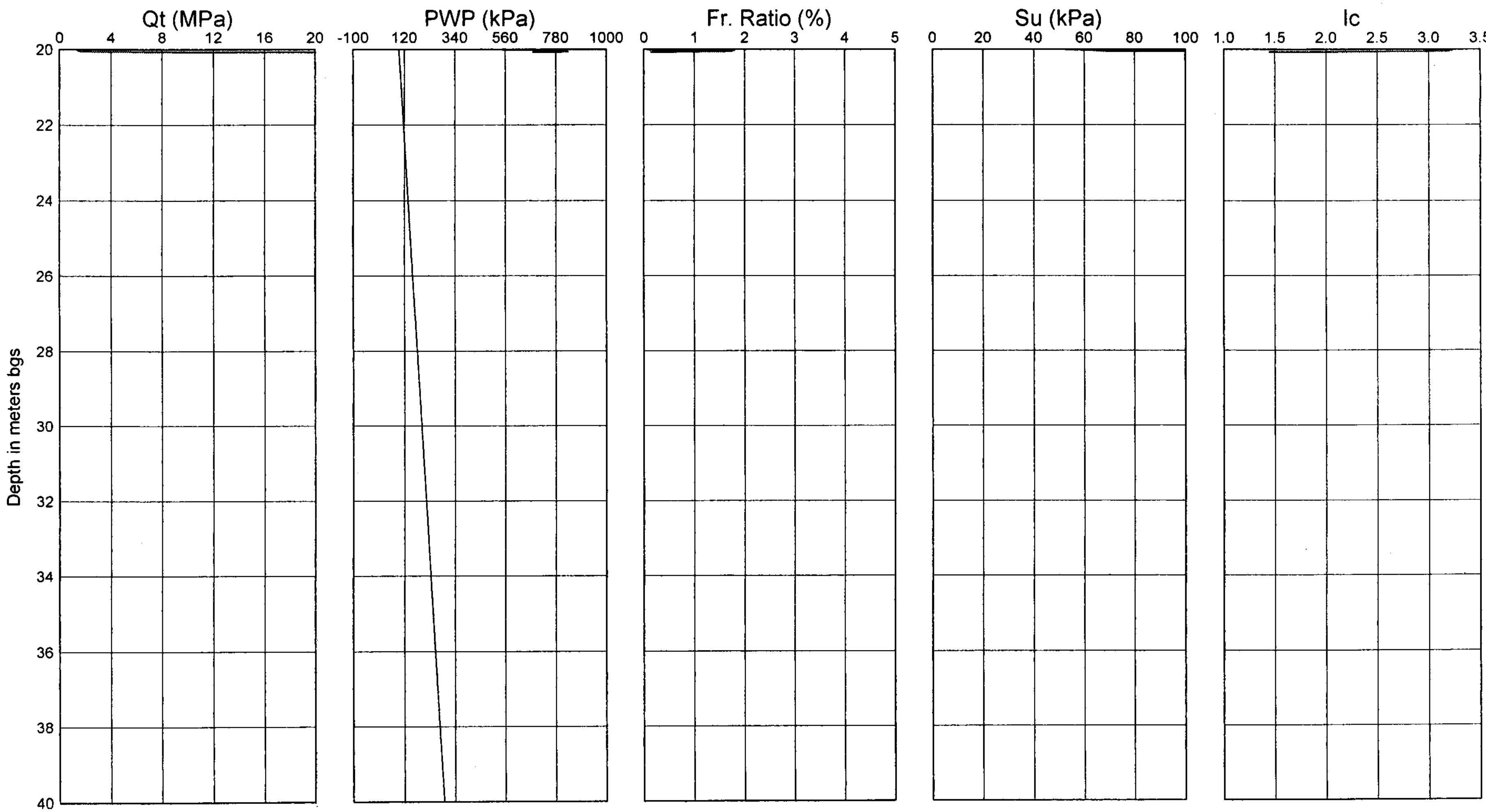

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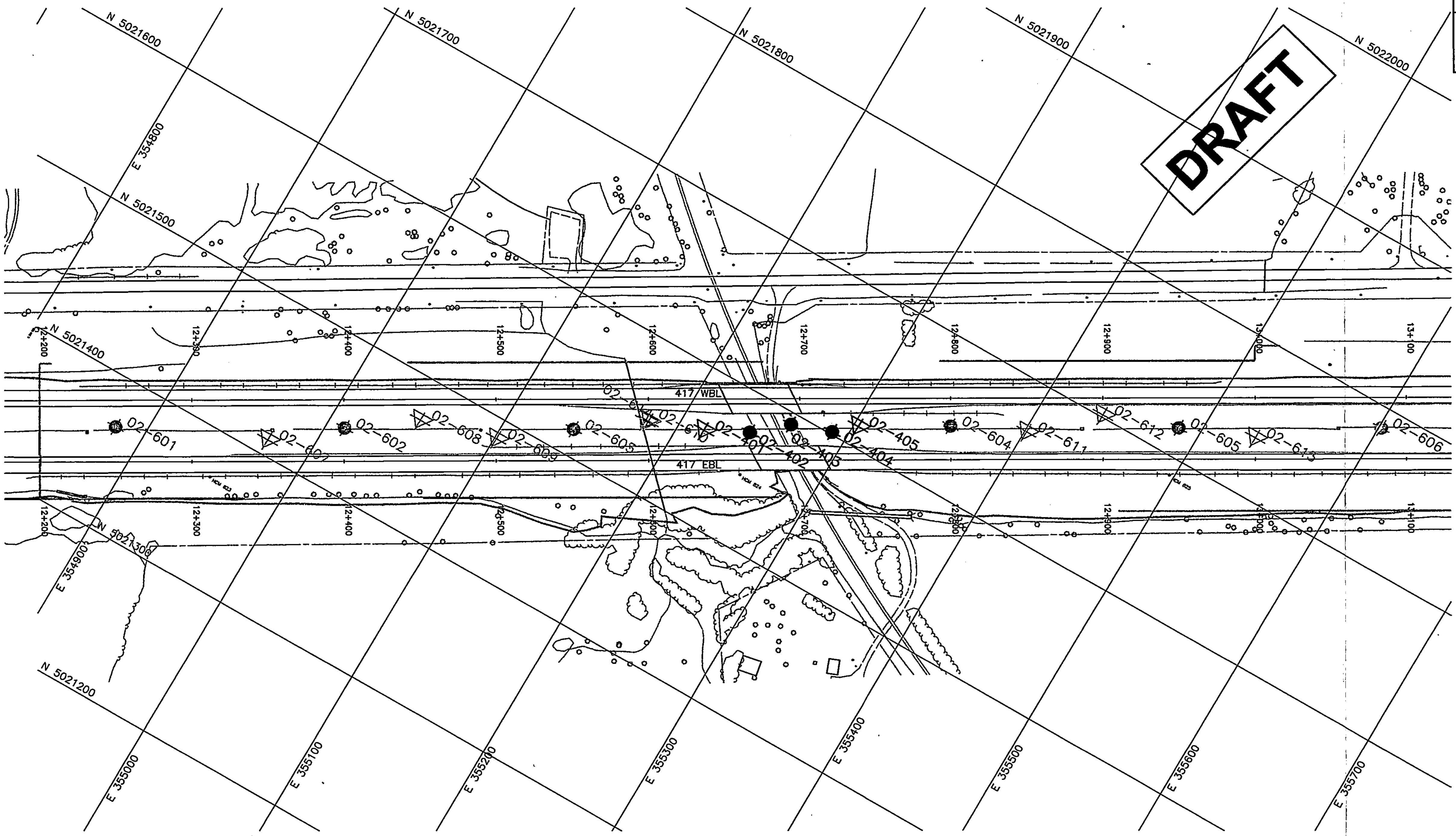
PROJECT NO. 021-1155-5100 DATE: Mar 10, 2004 DRAWN BY: CN

METRIC

DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN



25 0 25 50
SCALE METRES

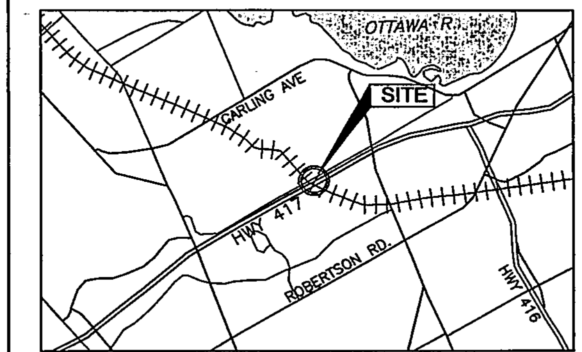
DIST. 9 HWY. 417
CONT No.
WP No.458-98-00

SHEET

CN RAIL OVERPASS WIDENING
BOREHOLE LOCATION PLAN

Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



KEY PLAN

SCALE
1.5 0 1.5 km

LEGEND

Borehole – Current Investigation

Penetration Test

No.	ELEVATION	CO-ORDINATES	
		NORTHING	EASTING
02–401	93.0	5021560.1	355216.3
02–402	94.0	5021574.1	355243.2
02–403	87.0	5021592.4	355264.0
02–404	92.5	5021601.8	355290.1
02–405	90.7	5021613.4	355303.0
02–601	98.2	5021367.4	354881.0
02–602	95.4	5021442.5	355011.5
02–603	94.1	5021517.5	355142.0
02–604	88.2	5021644.6	355356.1
02–605	84.1	5021719.2	355486.3
02–606	80.2	5021786.3	355601.6
02–607	96.8	5021410.3	354971.9
02–608	95.7	5021471.9	355051.6
02–609	95.5	5021486.3	355101.2
02–610	94.0	5021546.9	355181.6
02–611	88.0	5021666.5	355401.2
02–612	86.4	5021701.5	355438.7
02–613	83.1	5021738.6	355532.8
02–614	94.0	5021546.0	355179.7

NOTES

1. For details of 600 series boreholes. refer to Golder report 021–1155 (12)

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

For subsurface information only.

REFERENCE

Base plans provided in digital format by Marshall Macklin Monaghan, on December 19, 2003.

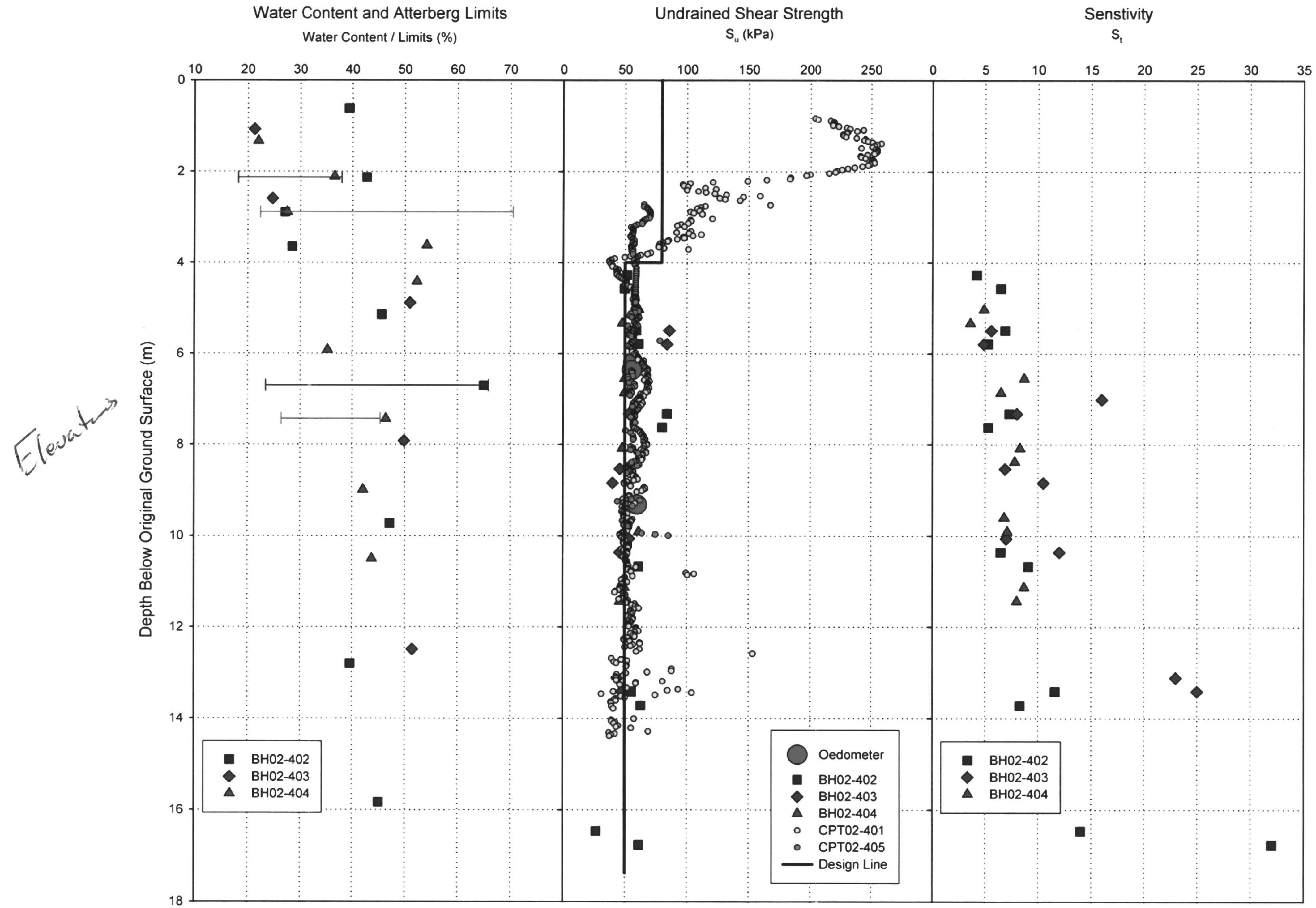
NO.	DATE	BY	REVISION

Geocres No.

HWY. 417		PROJECT NO. 021–1155 (4)	DIST. 9
SUBM'D. CN	CHKD. CN	DATE: FEB. 2004	SITE: 3–257
DRAWN: JDR	CHKD. JPD	APPD.	DWG. 1

SUMMARY OF WATER CONTENTS, ATTERBERG LIMITS AND UNDRAINED SHEAR STRENGTHS

FIGURE 1



Date: MARCH 2004

Project: 021-1155-4

Golder Associates

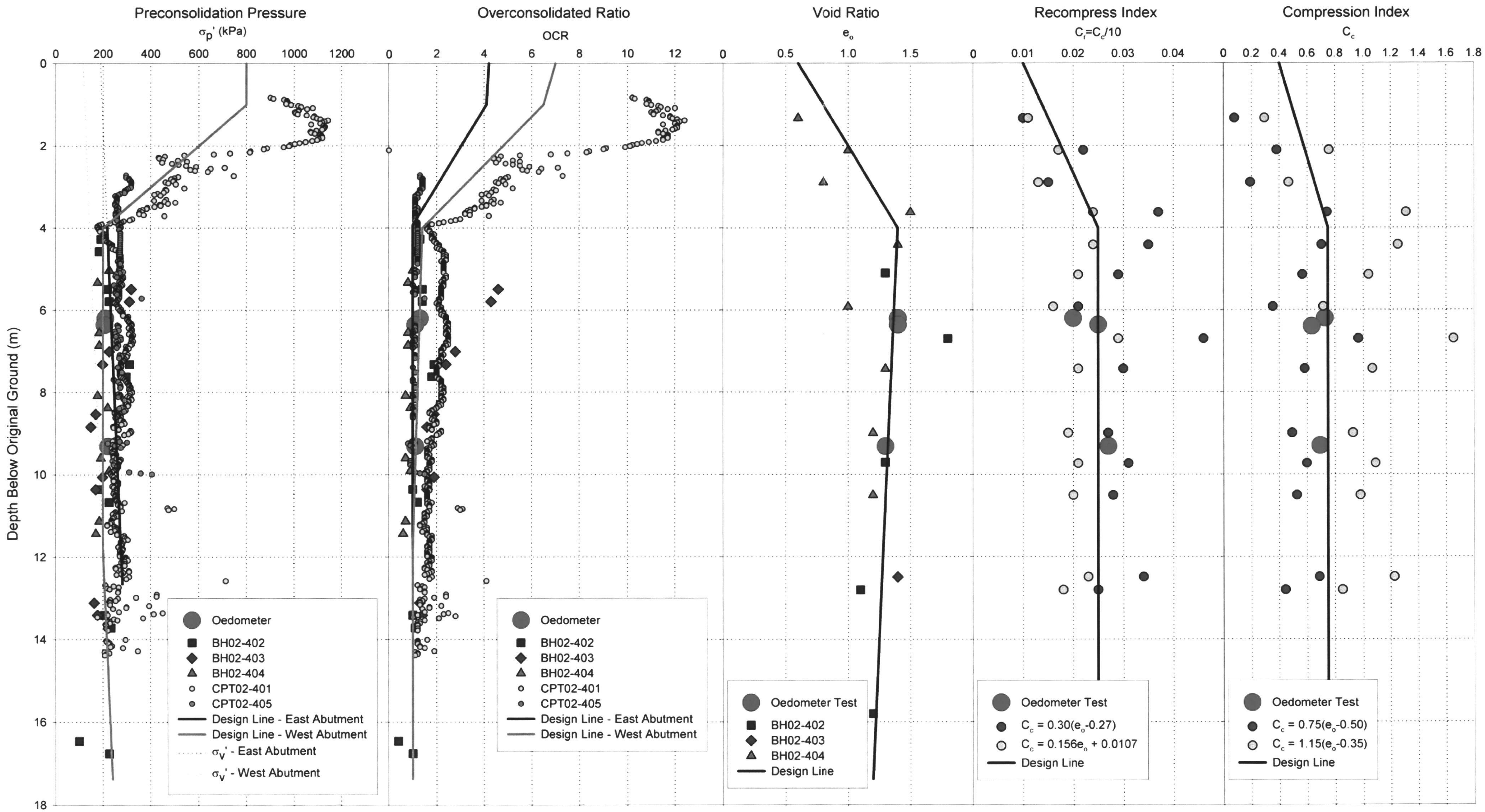
Drawn: RJ

Chkd: CN

02111554FIG01.cdr

SUMMARY OF CONSOLIDATION PROPERTIES

FIGURE 2



Date: MARCH 2004

Project: 021-1155-4

Golder Associates

Drawn: RJ

Chkd: CN

0211554FIG02.cdr

APPENDIX A

LABORATORY TEST DATA

TABLE A1

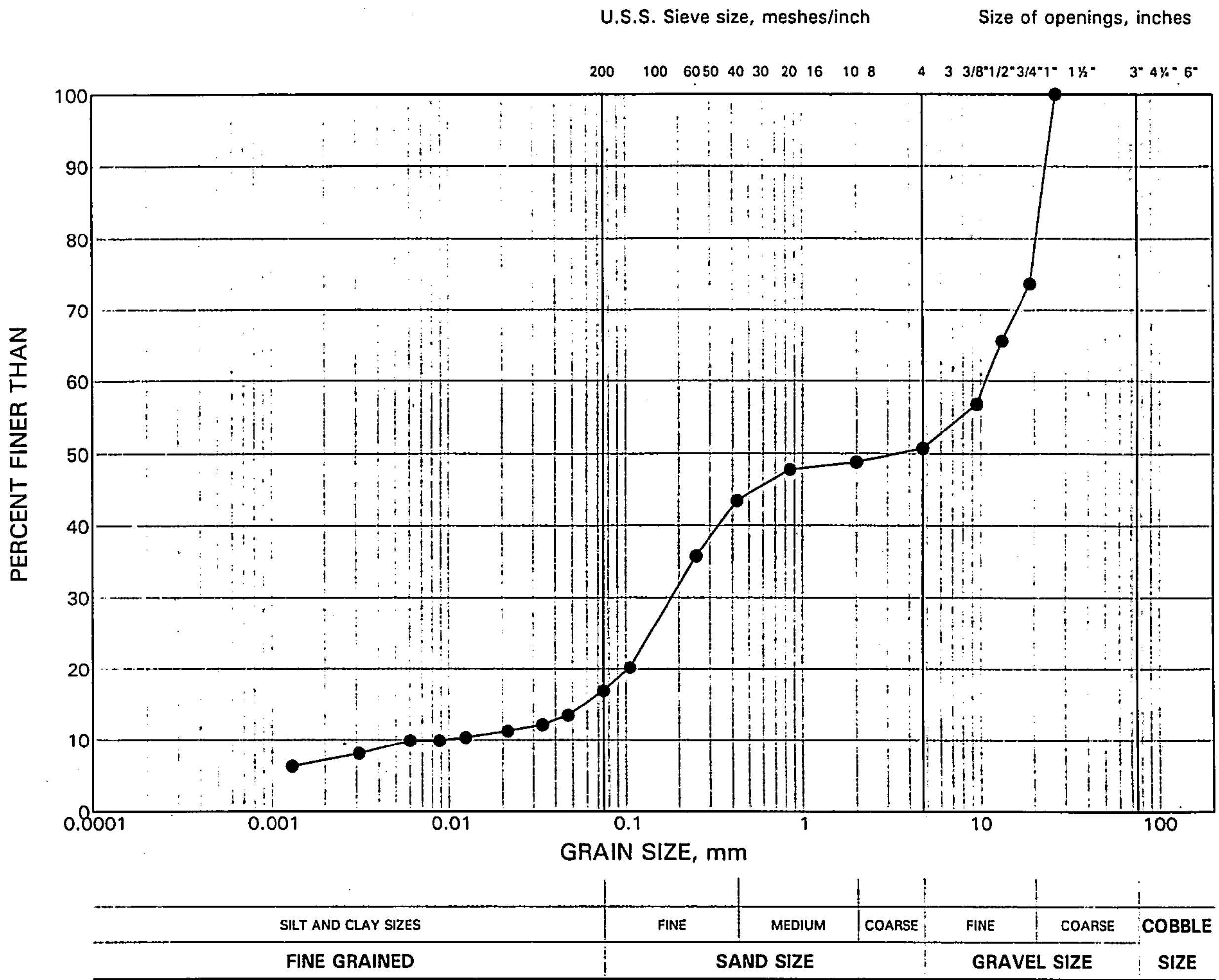
SUMMARY OF WATER CONTENT DETERMINATIONS

PROJECT NUMBER	021-1155
PROJECT NAME	MMM / Design / Hwy 7
DATE TESTED	December, 2002

Borehole No.	Sample No.	Depth (ft)	Depth (m)	Water Content (%)	Atterberg Limits LL, PL, PI
02-402	1	0.5-2.0	0.15-0.61	19.8%	
02-402	2	3.0-5.0	0.91-1.52	7.9%	
02-402	5	20.0-22.0	6.10-6.71	39.4%	
02-402	6	25.0-27.0	7.62-8.23	42.6%	LL=38.0, PL=18.2, PI=19.8
02-402	7	27.5-29.5	8.38-8.99	27.1%	
02-402	8	30.0-32.0	9.14-9.75	28.5%	
02-402	9	35.0-37.0	10.67-11.28	45.6%	
02-402	10	40.0-42.0	12.19-12.80	65.0%	LL=65.9, PL=23.5, PI=42.4
02-402	11	50.0-52.0	15.24-15.85	47.1%	
02-402	12	60.0-62.0	18.29-18.90	39.7%	
02-402	13	70.0-72.0	21.34-21.95	45.0%	
02-403	3	7.5-9.5	2.29-2.90	21.3%	
02-403	5	12.5-14.5	3.81-4.42	24.7%	
02-403	8	20.0-22.0	6.10-6.71	51.0%	
02-403	10	30.0-32.0	9.14-9.75	49.9%	
02-403	12	45.0-47.0	13.72-14.33	51.5%	
02-404	1	5.0-7.0	1.52-2.13	21.5%	
02-404	4	20.0-22.0	6.10-6.71	4.3%	
02-404	8	35.0-37.0	10.67-11.28	22.0%	
02-404	9	37.5-39.5	11.43-12.04	36.6%	
02-404	10	40.0-42.0	12.19-12.80	27.5%	LL=70.5, PL=22.4, PI=48.1
02-404	11	42.5-44.5	12.95-13.56	54.2%	
02-404	12	45.0-47.0	13.72-14.33	52.3%	
02-404	13	50.0-52.0	15.24-15.85	35.4%	
02-404	14	55.0-57.0	16.76-17.37	46.4%	LL=45.3, PL=26.4, PI=18.9
02-404	15	60.0-62.0	18.29-18.90	42.0%	
02-404	16	65.0-67.0	19.81-20.42	43.7%	

GRAIN SIZE DISTRIBUTION
Sand and Gravel (FILL) ←

FIGURE A2

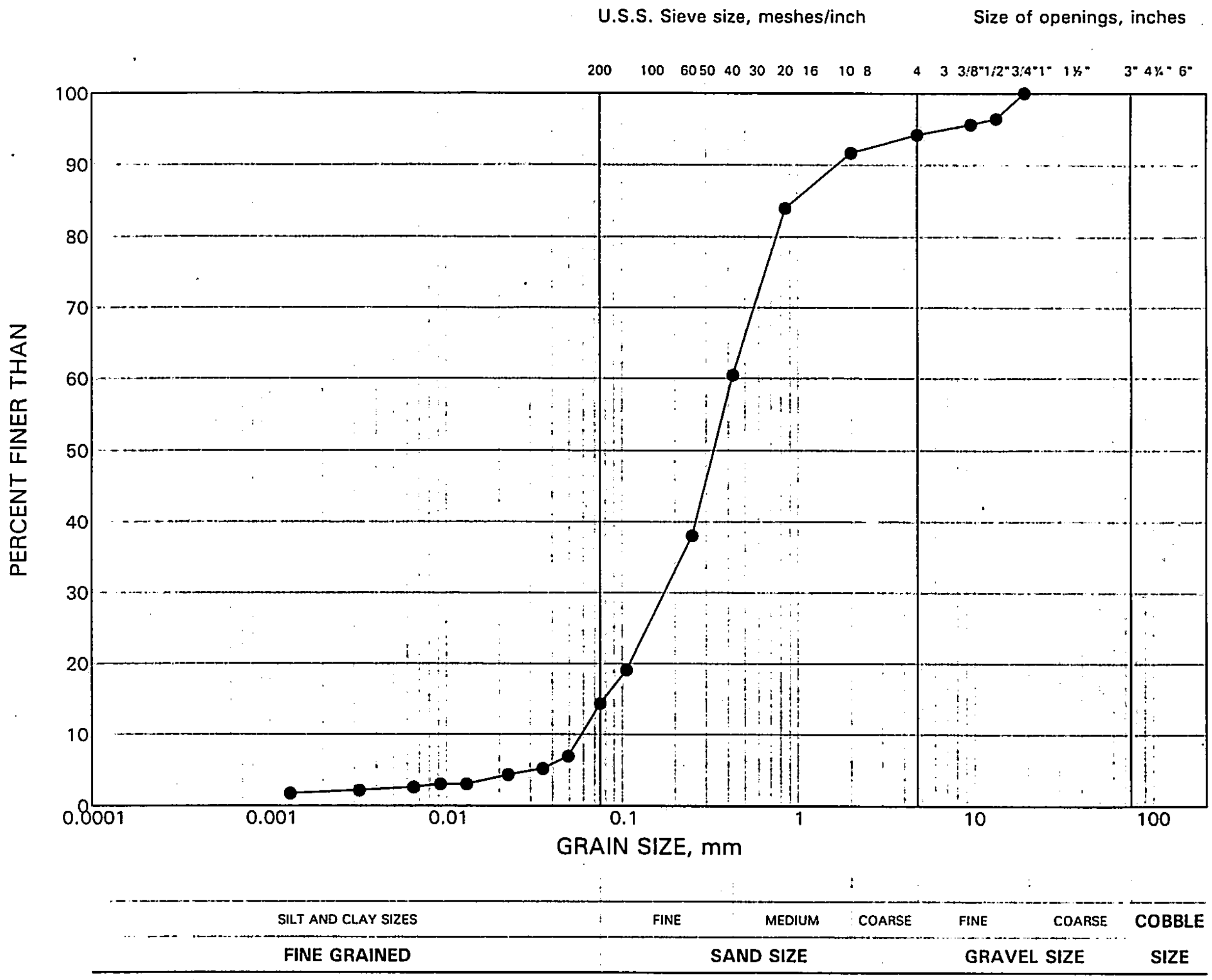


LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	02-402	2	1.5

GRAIN SIZE DISTRIBUTION
Sand (FILL) ←

FIGURE A3



LEGEND

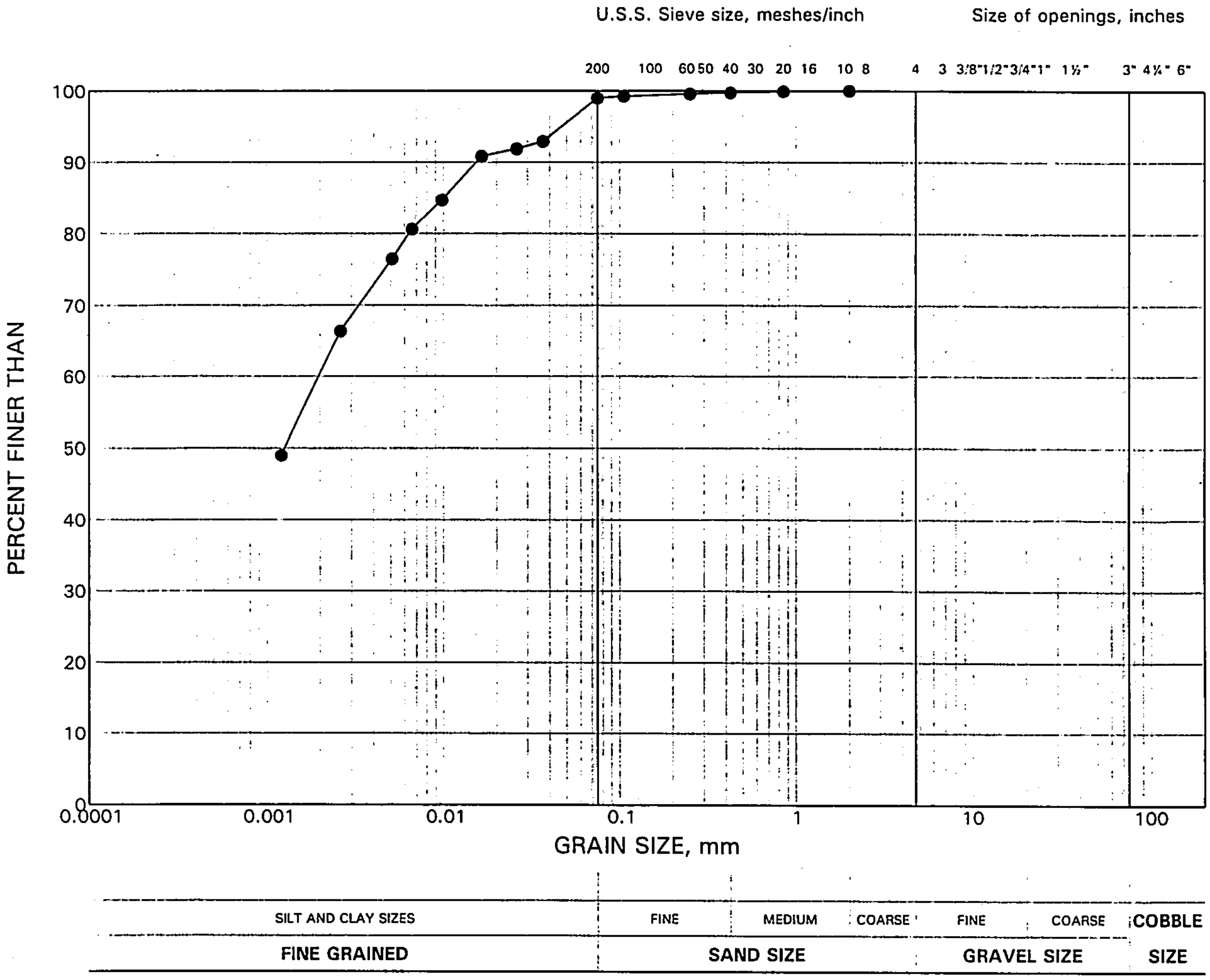
SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	02-404	4	6.7

Project 021-1155

Golder Associates

GRAIN SIZE DISTRIBUTION
Clay

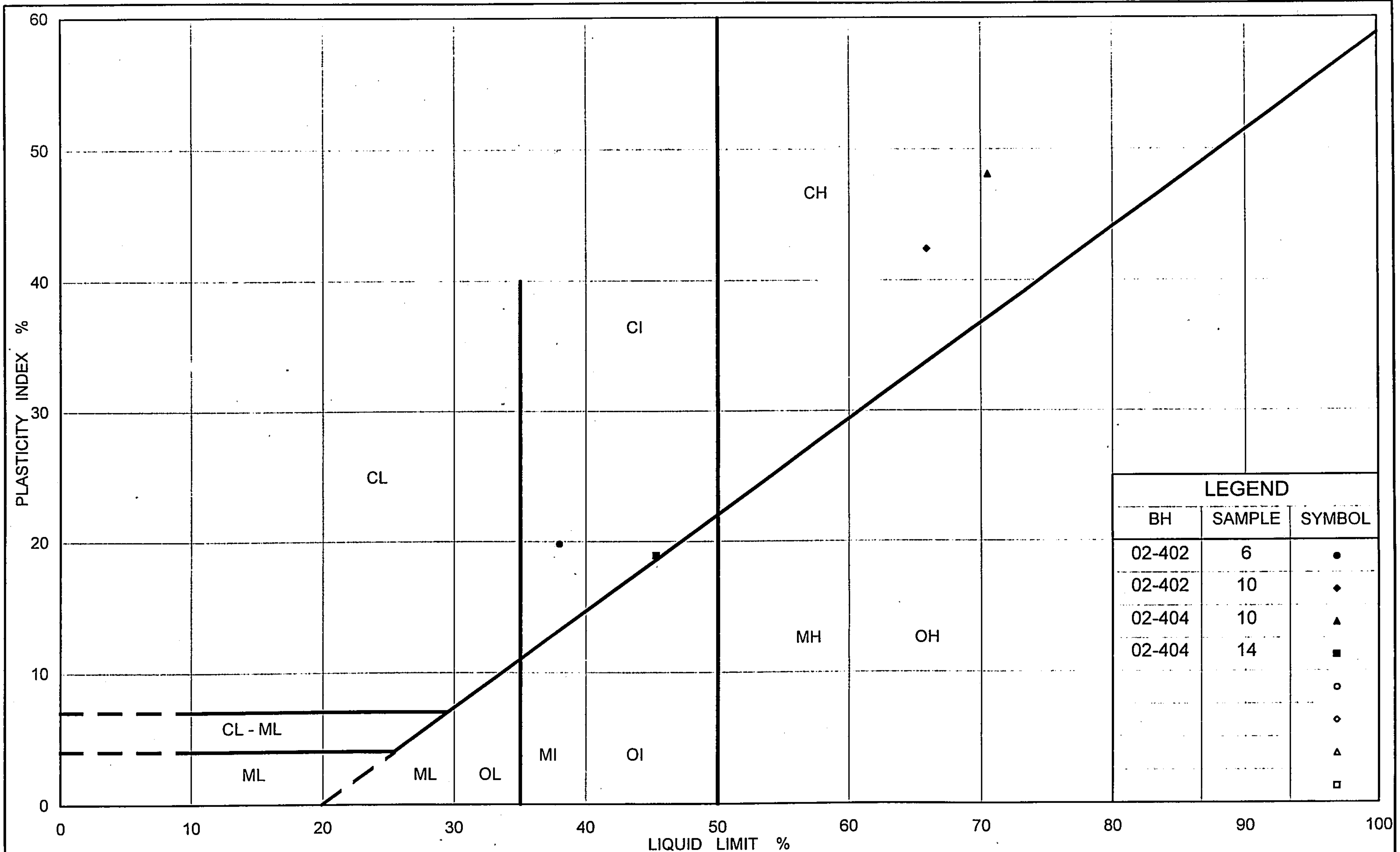
FIGURE A4



LEGEND

SYMBOL	BOREHOLE	SAMPLE	DEPTH (m)
●	02-402	10	12.8

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART

FIG No. A5

Project No. 021-1155