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REPORT ON

Foundation Investigation and Design Highway 401 Embankment Widening Cataraqui Wetlands Kingston, Ontario G.W.P. 78-99-00

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REPORT

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PART A

FOUNDATION INVESTIGATION
HIGHWAY 401 EMBANKMENT WIDENING
CATARAQUI WETLANDS
KINGSTON, ONTARIO
G.W.P. 78-99-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by McCormick Rankin Corporation on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a foundation investigation associated with the Highway 401 expansion in Kingston, Ontario. The section of Highway 401 included in this assignment (G.W.P. 78-99-00) extends from just west of Montreal Street to about 1.8 km east of the CNR structure.

Foundation investigation services are required for the following components:

- CNR bridge rehabilitation/widening;
- Highway 401 embankment widening – Cataraqui wetlands;
- Montreal Street Underpass replacement;
- Overhead signs (total of 2); and,
- Noise Barrier Wall.

This report addresses the Highway 401 embankment widening component at the Cataraqui wetlands.

The terms of reference for the original scope of work are outlined in the MTO's Request for Proposal (RFP) dated April 2008. The work was carried out in accordance with Golder's Quality Control Plan dated November 2008.

2.0 SITE DESCRIPTION

The Highway 401 embankment widening is proposed for about 1.8 km of the highway, extending easterly from the existing CNR bridge structure (Mile 171.10 of the Kingston Subdivision) at Station 26+700 to just west of the existing Cataraqui River Bridge at Station 28+450.

The highway grade in the vicinity of the site generally declines from about elevation 84 m at the CNR bridge structure to about elevation 78 m east of Station 27+350, and increases back to elevation 84 m at the Cataraqui River bridge structure.

The existing Highway 401 embankments are constructed of rock fill and range in height from about 11 m at the east approach to the CN Rail overpass (26+700) to 3 m high or less between 27+500 and 28+000. Where slopes are greater than 3 m in height, the existing side slopes are at between 1.25H:1V and 1.5H:1V. In areas where slopes are less than 3 m high, existing side slopes are as shallow as 4H:1V. No signs of embankment instability were observed at the time of the field investigation, although some evidence of historic slope instability was observed in air photographs.

Within the site boundaries, Highway 401 is currently 2 lanes wide in each direction (4-lane highway). The proposed widening of the existing embankments to accommodate a 6-lane highway will require placement of up to about 3.5 m (vertical thickness) of new fill on the existing side slopes. Widening will only be required for the embankments located within the higher portions of the highway (i.e., elevations of greater than 79 m. It is understood that the embankment widening will be within the area of the Cataraqui wetlands.

The following table summarizes the approximate locations of the proposed embankment fills:

Fill Area	Station
Westbound Widening	Stations 26+700 to 27+400
Eastbound Widening	Stations 26+700 to 27+500
Eastbound Widening	Stations 28+200 to 28+450

3.0 INVESTIGATION PROCEDURES

A subsurface investigation was carried out at the proposed embankment widening locations between February 2 and March 10, 2009, at which time 38 boreholes (numbered W1 to W15 for the northern widening, and E2 to E24 for the southern widening, inclusive) were advanced about 4 m away from the toe of the existing westbound and eastbound embankment slopes at roughly 50 m spacings. An additional 4 boreholes (numbered S1 to S4) were advanced through the existing embankment at the north and south shoulders of Highway 401 between February 8 and 11, 2010. In February, 2011, excavation of test pits at 19 locations was attempted at the toe of the eastbound and westbound slopes. The borehole and test pit locations are shown on Drawings 1 to 3.

The boreholes put down near the toe of the existing embankment were advanced using portable/manual drilling equipment supplied and operated by OGS Drilling Services of Appleton, Ontario. These boreholes were advanced to depths ranging from 2.1 to 14.0 m below the existing ground surface. Boreholes W5, W8, and E5 were advanced past the sampling depth using dynamic cone penetration techniques to assess the approximate bedrock surface (i.e., refusal) reached at depths ranging from 19.9 to 21.5 m below ground surface.

The boreholes put down through the existing embankment at the north and south shoulders of the highway were advanced using a truck-mounted hollow-stem auger drill rig supplied and operated by Marathon Drilling. These boreholes were advanced to depths ranging from 9 to 15 m below the existing road surface to assess the characteristics of the embankment fill and the extent of organic deposits beneath the roadway.

Soil samples were obtained nearly continuously, using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) ASTM D1586 procedures. In-situ vane testing (N vane) was carried out within the cohesive deposits, where possible. Relatively undisturbed, 75 mm diameter thin-walled Shelby tube (ASTM D1587) samples of the cohesive soils were retrieved using a fixed piston sampler, where possible.

In addition, borehole E6A was advanced to obtain two Shelby tube samples of an organic clayey silt layer, and borehole E23B was advanced by continuous sampling to assess the probable shallow bedrock surface (i.e., refusal). Furthermore, boreholes E14C1, E14C2, E23A, E24A, and E24B were extended beyond sampling using dynamic cone penetration techniques to assess the probable shallow bedrock surface (i.e., refusal).

The boreholes were backfilled with bentonite pellets, mixed with native soils, and the site conditions restored following completion of work.

Test pits dug into the toe of the existing embankment slope were advanced using an 8 ton Kubota KX080 excavator operated by TWD Roads Management Inc. of Kingston, Ontario. Where possible, these test pits were advanced to depths ranging from 0.5 to 3.4 m below the existing ground surface. Of the 19 locations selected by MTO, four of the test pit locations on the south side (western-most test pits) were inaccessible due to open water. On the north side of the embankment, frozen ground conditions combined with the presence of large rock slabs limited the advance of five test pits. Grab samples were obtained from test pits.

The field work was supervised throughout by members of our engineering and technical staff, who located the boreholes, supervised the drilling, excavation, sampling and in-situ testing operations, logged the boreholes and test pits, and examined and cared for the soil samples.

The soil samples were identified in the field, placed in labelled containers and transported to Golder's Mississauga and Ottawa laboratories for further examination and laboratory testing. Index and classification tests consisting of water content determinations, Atterberg limits testing and grain size distribution analyses were carried out on selected soil samples. Laboratory oedometer consolidation testing was carried out on five specimens of the silty clay deposit from boreholes W3, E5, E10, S3 and S4.

The borehole locations and ground surface elevations were determined by Golder personnel at the site using a Trimble R8 GPS unit. 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 Drawings 1 to 3.

Borehole Number	Borehole Location	Approx. Station	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
W1	Toe of westbound embankment	26+725	4904355.6	307285.1	75.8
W2	Toe of westbound embankment	26+750	4904361.4	307310.1	75.8
W3	Toe of westbound embankment	26+800	4904375.9	307356.4	76.0
W4	Toe of westbound embankment	26+850	4904389.5	307407.6	76.5
W5	Toe of westbound embankment	26+900	4904404.0	307451.1	75.6
W6	Toe of westbound embankment	26+950	4904417.9	307497.2	75.5
W7	Toe of westbound embankment	27+000	4904432.1	307544.5	75.4
W8	Toe of westbound embankment	27+050	4904446.9	307595.5	75.5
W9	Toe of westbound embankment	27+100	4904462.0	307639.8	75.4
W10	Toe of westbound embankment	27+150	4904475.4	307687.8	75.6
W11	Toe of westbound embankment	27+200	4904492.0	307735.9	75.6
W12	Toe of westbound embankment	27+250	4904507.7	307785.2	75.5
W13	Toe of westbound embankment	27+300	4904522.7	307834.5	76.6
W14	Toe of westbound embankment	27+350	4904536.9	307880.7	77.3
W15	Toe of westbound embankment	27+400	4904551.7	307926.3	77.2
E2	Toe of eastbound embankment	26+720	4904279.4	307296.6	75.5
E3	Toe of eastbound embankment	26+750	4904291.4	307330.3	75.1
E4	Toe of eastbound embankment	26+800	4904307.2	307374.7	75.3
E5	Toe of eastbound embankment	26+850	4904328.0	307424.2	76.0
E6	Toe of eastbound embankment	26+900	4904341.7	307466.8	75.5
E6A	Toe of eastbound embankment	26+900	4904340.9	307467.0	75.6
E7	Toe of eastbound embankment	26+950	4904360.8	307513.4	75.7
E8	Toe of eastbound embankment	27+000	4904378.9	307564.6	75.1
E9	Toe of eastbound embankment	27+050	4904389.8	307612.9	75.3

Borehole Number	Borehole Location	Approx. Station	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
E10	Toe of eastbound embankment	27+100	4904408.0	307654.3	75.5
E11	Toe of eastbound embankment	27+150	4904429.0	307709.2	75.1
E12	Toe of eastbound embankment	27+200	4904443.7	307753.6	75.4
E13	Toe of eastbound embankment	27+250	4904461.3	307804.2	75.2
E14	Toe of eastbound embankment	27+300	4904473.4	307848.6	75.1
E15	Toe of eastbound embankment	27+350	4904490.0	307901.8	75.0
E16	Toe of eastbound embankment	27+400	4904505.2	307944.3	75.0
E17	Toe of eastbound embankment	27+450	4904524.2	307992.5	75.0
E18	Toe of eastbound embankment	27+500	4904527.8	308038.5	75.4
E19	Toe of eastbound embankment	28+200	4904744.9	308704.2	75.8
E20	Toe of eastbound embankment	28+250	4904771.2	308753.9	75.1
E21	Toe of eastbound embankment	28+300	4904783.0	308797.6	75.0
E22	Toe of eastbound embankment	28+350	4904801.4	308850.1	77.1
E23	Toe of eastbound embankment	28+400	4904814.1	308903.4	76.6
E23A	Toe of eastbound embankment	28+400	4904813.3	308903.6	76.6
E23B	Toe of eastbound embankment	28+400	4904813.3	308902.0	76.7
E24	Toe of eastbound embankment	28+440	4904820.5	308930.1	75.1
E24A	Toe of eastbound embankment	28+440	4904819.9	308931.4	75.1
E24B	Toe of eastbound embankment	28+440	4904818.7	308929.8	75.1
S1	Right shoulder of westbound lanes	26+850	4904370.0	307410.0	83.0
S2	Right shoulder of westbound lanes	26+950	4904400.0	307503.2	81.3
S3	Right shoulder of eastbound lanes	27+150	4904443.0	307704.1	78.8
S4	Right shoulder of eastbound lanes	28+250	4904788.9	308748.0	81.4

The test pit locations were selected and laid out by MTO. The depths of the different stratigraphic units shown on the test pit logs are relative to the ground surface at the toe of the slope. The offsets of the test pits from the centreline of Highway 401 were estimated based on the nearest embankment cross section provided by MRC, or, in flatter areas, were measured relative to stakes provided by MTO. The approximate locations of the test pits are shown on in plan on Drawings 1 and 2.

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

The site is located in the southern portion of the physiographic region known as the Napanee Plain, and just west of the Leeds Knobs and Flats, as delineated in The Physiography of Southern Ontario[1].

The Napanee Plain is flat to undulating, and is characterized by relatively shallow soil deposits overlying bedrock. Geologic mapping[2] indicates that the bedrock within the Napanee Plain consists of grey limestone/dolostone of the Gull River Formation (of the Trenton-Black River Group), which contains some shale partings and seams.

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

The Leeds Knobs and Flats are characterized by knobs of Precambrian rock surrounded by clay flats. The clay is grey in colour, and very weakly calcareous.

In particular, the study area lies within the western limits of the Cataraqui River. The Cataraqui River is characterized by a number of lakes joined by the river. This river flows southerly towards Kingston and is one of two major rivers in the area.

4.2 Site Stratigraphy

The detailed subsurface soil and groundwater conditions as encountered in the boreholes advanced during this investigation, together with the results of the laboratory tests carried out on selected soil samples, are given on the attached Record of Borehole and Record of Test Pit sheets. The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling and in-situ vane testing and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions in the area beyond the existing toe of the highway embankment consist of up to about 1.7 m of fill material and/or up to 4.6 m of peat overlying up to about 4.8 m of organic deposits of silt, silty clay, clayey silt or silt, where present. The peat, fill and/or organic deposits are generally underlain by up to about 14.0 m of silty clay, clayey silt and/or clay. The silty clay/clayey silt/clay deposit is underlain by a thin silty sand till deposit, and auger refusal was encountered at depths of 1.8 to 21.5 m below existing ground surface (i.e., elevations of 54.1 to 73.4 m).

A more detailed description of the subsurface conditions encountered in the boreholes and test pits put down at each embankment fill location is provided in the following sections, and stratigraphic profiles along the embankments are shown on Drawings 1 to 3.

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

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

4.2.1 Westbound and Eastbound Embankments, Stations 26+700 to 27+500

The borehole locations and ground surface elevations for boreholes W1 to W15, inclusive, as well as the soil stratigraphy section projected along the proposed westbound embankment from Stations 26+700 to 27+400 are shown on Drawing 1. The borehole locations and ground surface elevations for boreholes E2 to E18, inclusive, as well as the soil stratigraphy section projected along the proposed eastbound embankment from Stations 26+700 to 27+500 are shown on Drawing 2. The borehole locations for boreholes S1 through S3 and for test pits attempted along the north and south toe of slope are shown on Drawings 1 and 2.

4.2.1.1 Pavement Structure

At boreholes S1, S2, and S3 put down on the right shoulder of the eastbound and westbound lanes, 0.4 m of road base consisting of crushed grey sand and gravel was encountered. At boreholes S1 and S3, the road base was underlain by 1.1 and 0.8 m, respectively, of subbase consisting of brown sand with some gravel. At S1, the subbase contained cobbles.

4.2.1.2 Embankment Fill

The pavement structure at boreholes S1 through S3 is underlain by embankment fill which increases in thickness as the highway approaches the CNR bridge structure. The embankment fill primarily consists of fine rock fill, with coarser rock fill and larger rock slabs encountered at test pits advanced at the toe of the slope. At borehole S2, the upper portion of the embankment fill is sand with some fine rock fill. At borehole S1, the middle portion of the fine rock fill from 7 to 11.5 m depth has a silty matrix. Sample recovery within split spoons in the embankment fill was generally poor (ranging from 8% to 54%, but generally less than 25%) and recovered rock fill samples comprised broken rock fragments (predominantly limestone) ranging in particle size from a coarse sand to gravel. The results of grain size distribution testing on two samples of this material are provided on Figure 1. It is important to note that the size of the samplers used during the field investigation limits the maximum retrieved particle size to about 35 mm diameter. Careful examination of the larger diameter gravel samples within the split spoon suggests that much of the gravel component of these samples represent fragments of larger gravel, cobbles and possibly boulders broken up during advancement of the spoon. Rough augering and grinding, deflection of the augers, and refusal to penetration (e.g., at 3.4 m depth at borehole S1), together with gravel-sized limestone rock fragments within the split spoon sampler indicate the presence of cobble and possibly boulder-sized material within the embankment fills. Coarser rock fill and larger rock slabs were encountered at test pits advanced at the toe of slope, as is evident on the embankment side slopes.

The Standard Penetration Test (SPT) "N" values measured within embankment fill above elevation 74.5 m typically range from 3 to 15 blows per 0.3 m of penetration, indicating that the fill has a very loose to compact relative density. Locally higher SPT "N" values of 25, 36, 39 and 50 blows recorded in the upper portions of S1, S2, and S3 may be indicative of cobbles or boulders present within the embankment fill. The SPT "N" values measured in embankment fills below about elevation 74.5 m range from about 12 to 31 blows per 0.3 m of penetration, indicating that this portion of the fill is generally compact.

4.2.1.3 Grade Fill

Grade fill was encountered at ground surface at the toe of the embankments at boreholes W1, W4, W13 to W15, E5, and E7, inclusive, with a thickness between about 0.3 and 1.7 m. The grade fill consists of variable amounts of sand, silty sand, sandy silt, silty clay, and/or clayey silt.

The Standard Penetration Test (SPT) "N" values measured within the cohesionless fill range from 3 to 8 blows per 0.3 m of penetration, indicating that the fill has a very loose to loose relative density. The SPT "N" values measured within the cohesive fill layers range from about 3 to 14 blows per 0.3 m of penetration, indicating that this portion of the fill has a soft to very stiff consistency.

4.2.1.4 Peat, Silty Peat and Sandy Peat

Peat was encountered at ground surface or below water/ice or fill at boreholes W2, W3, W5, W6, W8 to W12, and E2 to E18, inclusive, with a thickness that ranges from about 0.5 and 4.2 m. Peat was also encountered beneath the embankment fills at boreholes S1, S2 and S3. At S1 the peat was not fully penetrated, but was at least 0.1 m thick. At S2 and S3, the peat ranged in thickness from 0.6 to 1.0 m. West of Station 27+350 at all locations where test pits were advanced beyond the surficial rock fill, peat was encountered and ranged in thickness from 0.1 to 1.8 m in thickness (average thickness 0.9 m). The peat is generally fibrous and, at times, contains rootlets, decomposed wood fragments, shells, and traces of organics, clay, silt, sand, and/or gravel. Silty peat was encountered at boreholes W4, E5, E6, and E15 with thicknesses ranging from about 0.5 to 4.6 m. Sandy peat was encountered at borehole W7 with a thickness of about 1.2 m.

The measured SPT "N" values in the peat, silty peat and sandy peat ranges from 'weight of hammer' to about 60 blows per 0.3 m of penetration, indicating a very soft to hard consistency. However the SPT "N" values more generally range from 'weight of hammer' to about 12 blows per 0.3 m of penetration, indicating a very soft to stiff consistency. The measured natural water content of the peat, silty peat and sandy peat ranges from approximately 55 to 2,644 percent, but more generally ranges from approximately 220 to 685 percent.

The percentage of organic matter was measured in several samples of the peat. The measured organic content ranges from about 4 to 78 percent, but more generally is in excess of 15 percent.

4.2.1.5 Organic Silty Clay, Organic Silt, Organic Clayey Silt, and Organic Clay

Organic silty clay was encountered at ground surface or below the fill and peat at boreholes W2, W13, W15, E9, E13, and S2 with thicknesses ranging from about 0.4 to 4.8 m. Marl of 2.2 m thickness was encountered in borehole E6 and at test pits advanced along the westbound toe of slope at Stations 26+850 and 26+900, where it was proven for 0.3 to 1.0 m thickness. The measured SPT "N" values in the organic silty clay generally range from about 2 to 8 blows per 0.3 m of penetration, indicating a very soft to stiff consistency. One SPT "N" value of 43 blows per 0.3 m of penetration was measured at borehole E9; however this value is not considered representative of the entire deposit. The results of Atterberg limit testing on five samples of the organic silty clay indicate a plasticity index which varies from about 22 to 29 percent and a liquid limit which varies from about 49 to 88 percent, as shown on Figure 2, indicating a high to very high plasticity. The measured natural water content of the organic silty clay ranges from about 38 to 216 percent. The measured organic content in six samples of the organic silty clay ranges from about 6 to 14 percent. The measured organic content of a marl sample was 6 percent, with water contents ranging from 117 to 196 percent.

Organic silt was encountered below the fill and/or peat at boreholes W4 and W10, with a thickness of about 0.9 and 0.4 m, respectively. The measured SPT "N" values in the organic silt were approximately 9 and 3, indicating stiff and very soft consistencies, respectively. The results of grain size distribution testing on one sample of this material are provided on Figure 3. The results of Atterberg limit testing on two samples of the organic silt indicate plasticity index values of about 17 and 29 percent and liquid limit values of about 63 and 103 percent, as shown on Figure 2, indicating a high to very high plasticity. The measured natural water content of the organic silt was about 95 and 132 percent, which is in excess of the measured liquid limit.

Organic clayey silt was encountered below the peat at boreholes W5, W7, E2, E5, and E6A, with thicknesses ranging from about 1.2 to 1.3 m. The measured SPT "N" values in the organic clayey silt range from about 'weight of hammer' to 7 blows per 0.3 m of penetration, indicating a very soft to firm consistency. The results of Atterberg limit testing on two samples of this deposit indicate a plasticity index ranging from about 7 to 30 percent and a liquid limit ranging from about 72 to 112 percent, as shown on Figure 2, indicating a high to very high plasticity. The measured natural water content of the organic clayey silt ranges from about 139 to 282 percent, which is in excess of the measured liquid limit. The measured organic content in two samples of the organic clayey silt was about 6 and 9 percent.

Finally, organic clay was encountered below the peat at borehole E14, with a thickness of about 0.5 m. One measured SPT "N" value in the organic clay was about 11 blows per 0.3 m of penetration, indicating a stiff consistency. The results of Atterberg limit testing on one sample of the organic clay indicate a plasticity index of about 33 percent and a liquid limit of about 54 percent, as shown on Figure 2, indicating a high plasticity. The measured natural water content of one sample of the organic clay was approximately 38 percent, which is below the measured liquid limit. The measured organic content in one sample of the organic clay was about 8 percent.

Many of the Atterberg limit results plotted below the "A"-line on Figure 2, typical of organic samples.

4.2.1.6 Silty Clay, Clayey Silt and/or Clay

The fill, peat and organic deposits are underlain by a relatively thick deposit that ranges from silty clay to clayey silt to clay. The silty clay, clayey silt, and/or clay was encountered at all of the borehole locations with the exception of borehole E9. The silty clay/clayey silt/clay deposit was fully penetrated at boreholes W13 and E17 to depths of about 9.6 and 6.7 m below the existing ground surface level, respectively. The silty clay/clayey silt/clay deposit was not fully penetrated in the other boreholes, but proven for depths that range from about 2.7 to 14.0 m below the existing ground surface level (i.e., elevations of 72.3 to 61.6 m, respectively) at boreholes put down at the toe of the existing embankment. At boreholes S2 and S3 put down through the existing embankments, the depth to the top of the silty clay/clayey silt/clay deposit was 9.2 and 6.3 m below existing ground surface (i.e., elevations of 72.2 and 72.6 m, respectively).

The silty clay/clayey silt/clay deposit is unweathered and typically grey to brown in colour. The measured SPT "N" values within this deposit ranged between 'weight of hammer' and about 171 blows per 0.3 m of penetration, but more generally from 'weight of hammer' to about 60 blows per 0.3 m of penetration. In situ vane testing in this material measured undrained shear strengths varying widely from about 6 to greater than 120 kilopascals. Very soft to soft silty clay (with undrained shear strength of 6 to 20 kPa) was encountered at boreholes W4, W5, E5 and E6. Along the remainder of the embankment in this section, results indicate a generally firm to very stiff consistency. The results of grain size distribution testing on samples of the silty clay, clayey silt and clay are provided on Figures 4 to 8.

The results of Atterberg limit testing on samples of the silty clay indicate a plasticity index which varies from about 17 to 29 percent and a liquid limit which varies from about 35 to 55 percent, as shown on Figures 9A, 9B and 10, indicating the silty clay deposit to generally have a moderate plasticity. The measured natural water content of the silty clay ranges from about 23 to 72 percent, but more generally ranges from about 25 to 45 percent, and is generally below the measured liquid limit.

The results of Atterberg limit testing on samples of the clayey silt indicate a plasticity index which varies from about 12 to 18 percent, and a liquid limit which varies from about 28 to 33 percent, as shown on Figure 11, indicating the clayey silt deposit to have a low plasticity. The measured natural water content of the clayey silt ranges from about 22 to 37 percent, and is generally below the measured liquid limit.

The results of Atterberg limit testing on samples of the clay indicate a plasticity index which varies from about 28 to 41 percent, and a liquid limit which varies from about 50 to 64 percent, as shown on Figure 12, indicating the clay deposit to have a high plasticity. The measured natural water content of the clay ranges from about 29 to 57 percent, and is generally below the measured liquid limit.

Oedometer consolidation testing was carried out on five thin-walled Shelby tube samples of the silty clay. The results of that testing are provided on Figures 13 to 17, inclusive, and are summarized in the table below. Test results indicate that the silty clay at the toe of the existing embankment is close to normally consolidated, with a preconsolidation pressure ranging from about 25 to 35 kPa and an overconsolidation ratio ranging from approximately 1.1 to 1.4. Consolidation testing results on samples of silty clay from beneath the embankment are overconsolidated, with preconsolidation pressures ranging from 350 to 450 kPa and an overconsolidation ratio of 2.5 to 3.9.

Borehole/ Sample Number	**Sample Depth/ Elevation (m)**	**Unit Weight (kN/m^3)**	**σ_P' (kPa)**	**σ_{VO}' (kPa)**	**$\sigma_P' - \sigma_{vo}'$ (kPa)**	**C_c**	**C_r**	**e_o**	**OCR**	**c_v (cm^2/s)**
W3 / 9	6.4 / 69.6	18.5	35	25	10	0.12	0.057	1.0	1.4	0.0014
E5 / 10	6.4 / 69.6	17.9	25	17.5	7.5	0.34	0.065	1.2	1.4	0.0007
E10 / 7	4.7 / 70.8	17.5	25	22.5	2.5	0.34	0.081	1.2	1.1	0.0013
S3 / 8	7.0 / 71.8	20.1	450	115	335	0.13	0.0066	0.65	3.9	0.015
S4 / 6	7.8 / 73.6	19.3	350	142	208	0.22	0.017	0.83	2.5	0.0053

Notes: For boreholes W3, E5, E10, at the toe of slope, c_v is for stress range $10 \le \sigma_v' \le 100$ kPa

For boreholes S3, S4, beneath embankment, c_v is for stress range $70 \le \sigma_v' \le 500$ kPa

σ_P' - Apparent preconsolidation pressure
σ_{VO}' - Computed existing vertical effective stress
C_c - Compression index
C_r - Recompression index
e_o - Initial void ratio
OCR - Overconsolidation ratio
c_v - Coefficient of consolidation

4.2.1.7 Silty Sand Till

At boreholes E17 and W13, the silty clay/clayey silt/clay deposit is underlain by a deposit of silty sand which is inferred to be glacial till. The silty sand was proven to depths of about 6.9 and 9.8 m, below the existing ground surface (i.e., elevations of 68.1 and 66.8 m) at boreholes E17 and W13, respectively. The results of grain size distribution testing on one sample of the silty sand till from borehole E17 are provided on Figure 18.

The deposit was only penetrated for a thickness of about 0.2 m before encountering spoon refusal or terminating the borehole. Standard penetration testing could not, therefore, be effectively carried out.

The measured natural water content of two samples of the silty sand was 12 and 22 percent.

4.2.1.8 Spoon/Dynamic Cone Refusal

Practical refusal to spoon advancement or refusal to penetration of the dynamic cone was encountered at boreholes W2, W5, W11, W14, E9, E14, and E17. The depth to refusal and refusal elevations are summarized in the following table:

Borehole Number	Ground Surface Elevation (m)	Depth to Spoon/DCPT Refusal (m)	Spoon/DCPT Refusal Elevation (m)
W2	75.8	11.7	64.1
W5	75.6	21.5	54.1
W11	75.6	6.5	69.1
W14	77.3	6.3	71.0
E9	75.3	5.6	69.7
E14	75.1	2.7	72.4
E14C1	75.1	3.1	72.0
E14C2	75.1	2.9	72.2
E17	75.0	6.9	68.1

4.2.1.9 Groundwater Conditions

Ice and/or water was encountered at the ground surface at boreholes W7 to W9, E2 to E4, E15, E17, and E18, with thicknesses ranging from about 0.1 to 1.7 m.

Upon completion of the drilling at the toe of the embankment in February, 2009, the groundwater levels in the boreholes at the toe were measured at between ground surface and about 2.9 m (about elevation 75.5 m) below the existing ground surface, during the short time they remained open prior to backfilling. At boreholes S1, S2, and S3, advanced through the existing embankment in February, 2010, groundwater levels measured at the time of drilling were at about 74 m elevation.

It should be noted that groundwater levels in the area are subject to fluctuations both seasonally and with precipitation events.

4.2.2 Eastbound Embankment, Station 28+200 to 28+450

The borehole locations and ground surface elevations for boreholes E19 to E24, inclusive, and boreholes E23A, E23B, E24A, and E24B, as well as the soil stratigraphy section projected along the proposed eastbound embankment from stations 28+200 to 28+450 are shown on Drawing 3.

4.2.2.1 *Pavement Structure*

At borehole S4, 0.2 m of road base consisting of grey crushed sand and gravel was encountered. The road base was underlain by 0.5 m of subbase consisting of grey-brown sand and gravel.

4.2.2.2 *Embankment Fill*

The pavement structure at borehole S4 is underlain by embankment fill. The upper 2.4 m of fill consists of moist, brown sand with some gravel. This fill is underlain by 3.8 m of fine rock fill. Sample recovery within split spoons in the embankment fill was generally poor (ranging from 25% to 33%) and recovered samples comprised broken rock fragments (predominantly limestone) ranging in particle size from a coarse sand to gravel. Within 0.5 m of the fill/peat interface, the rock fill contains traces of organic material. It is important to note that the size of the samplers used during the field investigation limits the maximum retrieved particle size to about 35 mm diameter. Careful examination of the larger diameter gravel samples within the split spoon suggests that much of the gravel component of these samples represent fragments of larger gravel, cobbles and possibly boulders broken up during advancement of the spoon. Rough augering and deflection of the augers, together with gravel-sized limestone rock fragments within the split spoon sampler indicates the presence of cobbles and possibly boulders within the embankment fills.

The Standard Penetration Test (SPT) "N" value measured within the upper embankment fill at borehole S4 was 21 blows per 0.3 m of penetration, indicating that the fill is compact. The Standard Penetration Test (SPT) "N" values measured within the fine rock fill at borehole S4 range from 6 to 26 blows per 0.3 m of penetration, indicating that the rock fill is loose to compact.

4.2.2.3 *Grade Fill*

Grade fill was encountered at ground surface at boreholes E23 and E23B put down at the toe of the proposed embankment slope, with a thickness of about 0.6 m. The fill consists of clayey silt with trace to some sand and rootlets.

The result of two Standard Penetration Test (SPT) "N" values measured within the fill was about 3 blows per 0.3 m of penetration, indicating that the fill has a soft consistency.

4.2.2.4 *Peat*

Fibrous peat with a trace of sand was encountered at ground surface or below water/ice at boreholes E20 and E21, with a thickness of about 1.5 and 0.6 m, respectively. About 0.3 m of fibrous peat was also encountered beneath the embankment fill at borehole S4.

The measured SPT "N" values in this deposit at the toe of the existing embankment ranged from 'weight of hammer' to 5 blows per 0.3 m of penetration, indicating a very soft to firm consistency. At borehole S4, the measured SPT "N" value was 14 blows per 0.3 m of penetration, indicating a stiff consistency. The measured natural water content of three samples of the peat ranged from 110 to 375 percent.

4.2.2.5 *Organic Clay and Organic Clayey Silt*

Organic clay was encountered below the peat at borehole E20 and S4 with a thickness of about 2.8 m and 0.5 m respectively. The measured SPT "N" values in the organic clay at E20 range from 2 to 5 blows per 0.3 m of penetration, indicating a very soft to firm consistency. Beneath the embankment at S4, the measured SPT "N" value was 23 blows per 0.3 m of penetration indicating a very stiff consistency. The results of Atterberg limit testing on three samples of the organic clay indicate a plasticity index of 31 to 57 percent and a liquid limit of 60 to 89 percent, as shown on Figure 19, indicating a high plasticity. The measured natural water contents of three samples of the organic clay were approximately 52 to 73 percent. The measured organic content in one sample of the organic clay was about 9 percent.

Organic clayey silt was encountered below the peat at borehole E21 with a thickness of about 1.2 m. The measured SPT "N" values in the organic clayey silt were about 4 to 7 blows per 0.3 m of penetration, indicating a firm consistency. The measured natural water content of one sample of the organic clayey silt was approximately 46 percent. The measured organic content in one sample of the organic clayey silt was about 6 percent.

4.2.2.6 Silty Clay, Clayey Silt and/or Clay

Silty clay, clayey silt and/or clay was encountered at the ground surface at borehole E22 and below the ice/water, fill, peat, and/or organic clayey silt at the other borehole locations in this section. The silty clay/clayey silt/clay deposit was fully penetrated at borehole E24 to a depth of about 1.9 m below the existing ground surface level. The silty clay/clayey silt/clay deposit was not fully penetrated in the other boreholes, but proven for depths that range from about 3.1 to 7.3 m below the existing ground surface level at boreholes put down at the toe of the embankment and at 7.5 m depth below the embankment fill at borehole S4.

The silty clay/clayey silt/clay deposit is unweathered and typically grey to brown in colour. The measured SPT "N" values within this deposit ranged between about 12 and 165 blows per 0.3 m of penetration. In situ vane testing in this material measured undrained shear strengths varying widely from about 54 to 98 kilopascals. These results indicate a firm to hard consistency. The results of grain size distribution testing on samples of the silty clay and clayey silt are provided on Figures 20 and 21, respectively.

The results of Atterberg limit testing on samples of the silty clay indicate a plasticity index which varies from about 21 to 29 percent, and a liquid limit which varies from about 39 to 49 percent, as shown on Figure 22, indicating the silty clay deposit to have a moderate plasticity. The measured natural water content of the silty clay ranges from about 30 to 41 percent, and is generally below the measured liquid limit.

The results of Atterberg limit testing on samples of the clay indicate a plasticity index which varies from about 30 to 33 percent, and a liquid limit which varies from about 53 to 56 percent, as shown on Figure 23, indicating the clay deposit to have a high plasticity. The measured natural water content of the clay ranges from about 28 to 40 percent, and is generally below the measured liquid limit.

4.2.2.7 Silt

The silty clay at borehole E24 is underlain by a thin deposit of silt. The silt was proven to a depth of about 2.1 m, below the existing ground surface (i.e., elevation 73.0 m).

The deposit was only penetrated for a thickness of about 0.2 m before encountering spoon refusal or terminating the borehole. Standard penetration testing could not, therefore, be effectively carried out.

The results of Atterberg limit testing on one sample of the silt indicates a plasticity index of about 2 percent and a liquid limit of about 15 percent, as shown on Figure 24, indicating a low plasticity. The measured natural water content of one sample of the silt was approximately 13 percent.

4.2.2.8 *Spoon/Dynamic Cone Refusal*

Practical refusal to spoon advancement or refusal to penetration of the dynamic cone was encountered at boreholes E23, E23A, E23B, E24, E24A, and E24B. The depth to refusal and refusal elevations are summarized in the following table:

Borehole Number	Ground Surface Elevation (m)	Depth to Spoon/DCPT Refusal (m)	Spoon/DCPT Refusal Elevation (m)
E23	76.6	3.2	73.4
E23A	76.6	3.2	73.4
E23B	76.7	3.1	73.6
E24	75.1	2.1	73.0
E24A	75.1	1.8	73.3
E24B	75.1	2.0	73.1

4.2.2.9 *Groundwater Conditions*

Ice and/or water was encountered at the ground surface at boreholes E19 and E21, with thicknesses of about 0.0 and 0.6 m, respectively.

Upon completion of the drilling, the groundwater levels in the boreholes put down at the toe of the embankment were between ground surface and 2.1 m (about elevation 75) below the existing ground surface, during the short time they remained open prior to backfilling. The groundwater level measured in open borehole S4 put down through the existing embankment was at elevation 75.5 m.

It should be noted that groundwater levels in the area are subject to fluctuations both seasonally and with precipitation events.

5.0 CLOSURE

This report was prepared by Ms. Kim Lesage, EIT and Ms. Erin O'Neill, P.Eng., under the direction of the Project Manager, Mr. Michael Snow, P.Eng. Mr. Fintan Heffernan, Golder's Designated MTO Contact for this project, conducted a technical and independent quality control review of the report.

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PART B

FOUNDATION INVESTIGATION
HIGHWAY 401 EMBANKMENT WIDENING
CATARAQUI WETLANDS
KINGSTON, ONTARIO
G.W.P. 78-99-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides an interpretation of the geotechnical data obtained during the investigation and recommendations on the foundation aspects of design of the proposed works.

The recommendations provided herein are based on interpretation of the factual data obtained from boreholes advanced during a subsurface investigation at the toe and, to a lesser extent, at the crest, of the existing Highway 401 embankment across the Cataraqui wetlands. The interpretation and recommendations provided are intended to provide the design engineers with sufficient information for geotechnical design of the widened embankments. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project, and for which special provisions or operational constraints may be required in the Contract Documents. Those requiring information on 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.

Section 6.1 provides a summary of the existing embankment geometry, available subsurface information and the proposed widening. For each design section, this information is also summarized in Table 1. Section 6.2.1 outlines the stability and settlement analysis methodology, parameter selection, and performance requirements used for the widened embankment, together with recommendations for stable embankment geometry and embankment fills. A summary of foundation engineering parameters for each design section is provided in Table 2. Section 6.3 provides a general discussion and recommendations related to potential alternatives for mitigating stability and settlement-related design and construction problems. A summary of applicability/feasibility, advantages and disadvantages, relative costs and risks/consequences for each mitigation option is provided in Table 3. The results of the analyses and recommendations on mitigating stability and time-dependent settlements are presented by design section in Section 6.4. A summary of the preferred mitigation options for each design section is provided in Table 4. Section 6.5 provides a discussion on the non-standard embankment geometries and construction methodologies selected by the design team to meet the overall project constraints. Subgrade preparation requirements and embankment construction recommendations are provided in Sections 6.6 and 6.7.

6.1 General Discussion

Golder Associates Ltd. (Golder) was retained by McCormick Rankin Corporation (MRC) to provide design recommendations on foundation aspects for the proposed widening of Highway 401 embankments over swamp/wet ground areas in the Greater Cataraqui Marsh between 20 m east of the CN Rail overpass (Station 26+720) and the Cataraqui River (Station 28+450), near Kingston, Ontario. The scope of work includes stability and settlement analyses, recommendations for stable embankment geometry and embankment fill materials, and implementation of ground improvement techniques that may be required as a means to minimize settlements and to improve stability (if necessary). The work also includes addressing specialized construction concerns and potential geotechnical problems associated with embankment construction, including sub-excavating soft/organic materials and placement of new fill materials.

6.1.1 Existing Embankment Geometry

The existing Highway 401 embankment was constructed through the Cataraqui Marsh in the 1950's. The embankment ranges in height from about 11 m at the east approach to the CN Rail overpass (Station 26+720) to 3 m high or less between Stations 27+500 and 28+000. Where slopes are greater than 3 m in height, the existing side slopes are at between 1.25H:1V and 1.5H:1V. In areas where slopes are less than 3 m high, existing side slopes are as shallow as 4H:1V. Between Station 28+000 and the Cataraqui River Bridge (Station 28+450), the embankment height increases from 3 to 8.5 m and the embankment slopes are at about 1.25H:1V. The highway throughout the study area is currently 4 lanes wide.

A summary of the height and side slopes of the existing embankment within each design section is presented in Table 1.

6.1.2 Summary of Subsurface Conditions

A series of boreholes put down at the eastbound and westbound toe of the proposed embankment indicate that the subsurface conditions outside of the existing embankment within the proposed area of widening generally comprise fibrous peat and fine organic deposits typically less than 3 m, but in places up to 5.5 m, thick overlying generally stiff to very stiff, but locally soft, silty clay and clayey silt. At boreholes put down between Stations 26+825 and 26+925, referred to herein as Section B, the upper 2 to 3 m of the clayey soils are very soft to soft (undrained shear strengths of 6 to 20 kPa) and may have been remoulded by some historical slope instability in the area. At depth, these deposits are underlain by glacial till and bedrock. Boreholes put down at select locations along the existing Highway 401 shoulder indicate that the embankment is constructed of fine rock fill underlain by a thin zone of organics over stiff to very stiff silty clay to clayey silt. The presence of a 0.3 to 1.3 m thick layer of peat beneath the embankment fill indicates that organic deposits were not completely removed prior to construction of the existing embankment.

The highway embankment through the Cataraqui wetlands was originally constructed in the 1950's. We understand from discussions with current and former MTO engineers that construction practice for embankments on swamps at that time was to dig out the organic deposits to firm ground or to the maximum reach of backhoes employed at the time, which ranged from about 3.5 to 4.5 m below ground surface. If organics or soft ground extended beyond the reach of excavation equipment, a rolling surcharge was sometimes used to create a mud wave and displace additional soft material as the embankment was built. The footprint of the excavation was typically extended laterally to the crest of the embankment plus an additional distance equal to the height of the embankment beyond the crest in both directions.

Attempts to obtain additional information on the lateral extent of rock fill (and the extent of compressible organics) beneath the toe and sideslopes of the existing embankment were made via a supplementary test pit investigation. Along the toe of the westbound lanes (the north slope) where excavations were possible, peat and organics (0.6 to 1.8 m thick) were encountered beneath the toe and sideslopes of the embankment west of Station 27+350. At Station 27+350 and east, no peat was encountered beneath the sideslopes. Along the toe of the eastbound slopes (south slopes), no additional information was obtained west of Station 27+020, because of accessibility issues. In the remaining testpits to the east, 0.8 m of peat was only encountered at one of the four locations (i.e., peat was likely removed at the other locations).

A summary of the relevant boreholes for each design section, and the simplified stratigraphy at the toe of slope within the design section, is presented in Table 1.

6.1.3 Proposed Widening of High Fill Embankments Over Swamps

The existing embankment is to be widened by up to 5 m on each side to accommodate the additional travel lane, fully paved shoulder and 4 horizontal to 1 vertical rounding, requiring placement of up to 3.5 m (vertical thickness) of new fill on the existing side slopes. Note that the stability and settlement mitigation options presented in Section 6.3 and the results of analyses presented in Section 6.4 were carried out using a road platform geometry (with 4H:1V roundings) based on standard MTO embankment widening techniques. To meet the overall project constraints, the embankment geometries were later changed (overall widening reduced) and some non-standard construction methodologies introduced. The implications of the changes in the embankment geometry and construction methodologies to the results of the settlement and stability analyses are provided in Section 6.5.

A summary of the proposed widening along each design section, for the original and modified embankment geometry, is presented in Table 1.

A high pressure gas main crosses beneath the existing embankment near Station 27+756, in an area of the highway which does not require widening. In addition, this section of high fill embankment crosses three culverts. Geotechnical recommendations for these culverts was previously provided in the Draft Foundation Investigation and Design Report for the Proposed Culvert Extensions at Stations 27+340, 27+675 and 28+037 under W.P. 78-99-00 (Golder Report No. 08-1111-0044-4800, dated October, 2012). Geotechnical recommendations for the approach embankments within 20 m of the CN Rail bridge structure were previously provided in the Foundation Investigation and Design Report for the CN Rail Overpass Structure under W.P. 78-99-00 (Golder Report No. 08-1111-0044-2, dated January, 2011).

6.2 Engineering Analysis Methodology and Parameter Selection

Section 6.2.1 provides recommendations for stable embankment geometry and embankment fills. Sections 6.2.2 and 6.2.3 of this report summarize the methods used for analysis of embankment stability and settlement resulting from the placement of additional fill for the embankment widening for critical sections of the embankment. A summary of foundation engineering parameters for each design section is provided in Table 2.

6.2.1 Embankment Fill Types and Benching Requirements

Different embankment fill alternatives (i.e., rock fill, granular fill and earth fill) provide relative advantages and disadvantages in terms of availability, weight (i.e., driving force and applied load to founding subsoils/bedrock), construction cost and time, ease of construction and post-construction performance.

It is understood that rock fill (generated from widening of the adjacent portion of Highway 401 in this contract section) is the preferred embankment fill material for this project. As such, the stability and settlement analyses discussed below have been carried out on the basis that the majority of the roadway embankments will be constructed of fine rock fill.

6.2.1.1 Rock Fill

The main advantage of constructing embankments using rock fill is the ability to achieve steeper side slopes (1.25H:1V), thus reducing the overall quantity of material required for the widened embankment and the amount of material placed in sub-excavated areas under water. Rock fill will also likely be available locally, either from widened excavations in adjacent bedrock cuts or from rock borrow areas close to the project limits. At this site, the added advantage of using rock fill in the widened sections is that the existing embankment is also constructed of rock fill, and using similar materials in the widened sections will minimize the need for chinking or special transition layers where the two materials meet. The disadvantage of using rock fill for the construction of high embankments is that some post-construction settlement of the embankment fill itself will occur, although mostly within about the first year of construction. Settlement of the rock fill is discussed further in Section 6.2.3.3.

For rock fill embankments, the incorporation of a 2 m wide bench into the uniform side slope profile is required wherever the embankment will exceed a height of 10 m (in accordance with MTO guidelines). With the exception of immediately adjacent to the CN rail overpass (from Station 26+720 to 26+760), embankments are less than 10 m in height and benches are not required.

6.2.1.2 Earth Fill

The main advantage of constructing embankments with earth fill (i.e., sand and gravel) is the ease of construction and negligible post-construction settlement within the embankment fill itself. However, this option would require a larger volume of fill and wider right-of-way because the side slopes of earth fill embankments (2H:1V) are flatter than those of rock fill. In sections with high embankments and marginal stability, this would also result in larger driving forces and potentially less stable slopes requiring additional sub-excavation and ground improvement.

For earth fill embankments, MTO guidelines require the incorporation of a 2 m wide bench into the uniform side slope profile wherever the embankment exceeds a height of 8 m.

6.2.2 Embankment Stability Analysis

The following sections outline the methodology used to evaluate embankment stability at the various embankment sections and the parameters used in the analyses. The results of the analyses, provided based on the original embankment widening geometry and assuming standard MTO construction practices, are presented in Section 6.4. For each design section, the analysis results are discussed in combination with recommendations regarding possible design and construction mitigation alternatives. Section 6.5 provides a discussion of the non-standard embankment geometries and construction methodologies subsequently proposed, and the implications of the changes on the results of the stability analyses provided in Section 6.4.

6.2.2.1 Methodology and Performance Requirements

Stability analyses were performed for the critical sections of the proposed high fill embankment widening. For this report, the critical sections were assumed to correspond to the highest embankment height and/or the maximum thickness of soft, grey, compressible clay.

The global stability of the embankment slopes was assessed using the commercially available program SLOPE/W produced by Geo-Slope International Ltd., employing the Morgenstern-Price method of analysis. For all analyses, the factor of safety of numerous potential failure surfaces was computed in order to establish the minimum factor of safety. A target factor of safety of 1.3 against deep-seated, global failure that would affect the operation of the highway is normally used for the design of embankment slopes under static conditions. This factor of safety is considered adequate for the embankment widenings at this site considering the design requirements and the field data available.

Pseudo-static seismic slope stability analyses for the above configuration were also carried out assuming a seismic coefficient of 0.05g (k_h = 0.5 A). The minimum target factor of safety used for seismic loading conditions was 1.1.

In all areas, the analysis assumes that the peat and near-surface organic soils (encountered at or below the ground surface during drilling operations) of significant thickness (i.e., greater than 0.1 m thick) will be removed from below the footprint of the widening as part of the embankment construction. For design purposes, the groundwater level is based on the piezometric conditions observed during drilling. In general, the groundwater level is at about the level of the original (i.e., swamp) ground surface.

6.2.2.2 *Parameter Selection*

For cohesionless soils, effective stress parameters were employed in the analyses assuming drained conditions for the soils. The effective stress parameters (effective friction angle and cohesion) for these soils were estimated from empirical correlations using the results of in situ Standard Penetration Tests (SPT), in conjunction with engineering judgement considering experience in similar soil conditions.

For cohesive deposits, total stress parameters were employed in the analyses assuming undrained conditions for these soils. The total stress parameters for the clayey silt to clay deposit were assessed based on the undrained shear strengths from vane shear strength tests, as well as shear strengths calculated from oedometer (consolidation) tests results based on the formula $s_u = 0.22\ \sigma_p'$, where s_u is the average mobilized undrained shear strength (kPa) and σ_p' is the preconsolidation pressure (kPa). For the cohesive deposits beneath the embankment it was assumed that the minimum total stress parameters corresponded to a normally consolidated clay (preconsolidation pressure equals existing effective stress), unless information from drilling at the embankment or toe indicated stiffer soils.

Drained (effective stress) parameters were used to model the long-term frictional resistance of the peat. Fibrous peats are frictional materials with high peak friction angles resulting from the interlocking of fibres within the deposit. Typical values of effective stress friction angles in fibrous peats from triaxial compression tests are in the range of 40 to 60 degrees (Mesri, 2007), but large shear deformations are required to mobilize the maximum frictional resistance. Measurements of at rest earth pressure in fibrous peat deposits are typically in the range of 0.30 to 0.35 (Mesri, 2007), corresponding to an at rest friction angle of between 40 and 44 degrees. As such, a value of 40 degrees for the effective friction angle of peat was used for the virgin peat deposits at the toe of slope and beyond the existing embankments for most of the slope sections. In Section B, the friction angle of the peat (48 degrees) was estimated from back analysis assuming an existing factor of safety of about 1.3.

The table below summarizes the soil parameters that have been used in the stability analyses. A more complete description of the foundation engineering parameters used for the slope stability analyses at each section is found in Table 2 appended to the end of this report.

Material	Unit Weight (kN/m^3)	Effective Friction Angle	Undrained Shear Strength (kPa)
Rock Fill for Embankment Widening	21	40°	
Existing Embankment Fill	21	40°	
Peat	9.6 - 10.7	40° - 48°	29°*
Organic Silt/Clay	12 - 14	27°	
Silty Clay to Clayey Silt to Clay	17.4 – 19.3		6 – 100**
Till	22	Impenetrable by failure surface	
Bedrock	24	Impenetrable by failure surface	

* As per Section 6.5.1, undrained shear strength equivalent to a short term friction angle of 29 degrees.

** A breakdown of the undrained shear strength for the cohesive soils with depth within each design section is provided in Table 2.

6.2.3 Settlement Analysis

The following sections outline the methods used to conduct the settlement analyses at the various embankment widening sections. In addition, the parameters used in the analyses for each of the critical section(s) are also presented. The results of the settlement analyses, provided based on the original embankment widening geometry and assuming standard MTO construction practices, are presented in Section 6.4. For each design section, the analysis results are discussed in combination with discussions regarding potential settlement mitigation measures. Section 6.5 provides a discussion of the non-standard embankment geometries and construction methodologies subsequently proposed, and the implications of the changes on the results of the settlement analyses provided in Section 6.4.

6.2.3.1 Methodology

To estimate the magnitude of the expected settlements that will occur along the Highway 401 embankment as a result of widening between Stations 26+700 and 27+500 and Stations 28+200 and 28+450, analyses were carried out on critical sections of the widened fill embankments using the commercially available program Settle3D (Version 2.0) produced by Rocscience Inc. and/or hand and spreadsheet calculations. Critical sections correspond to the greatest new embankment height and/or the maximum thickness of soft, compressible clay soils. The rate of settlement/consolidation of the cohesive foundation soils was assessed using Terzaghi's one-dimensional consolidation theory.

New loads imposed on the underlying foundation soils resulting from the proposed widening include:

- Increases in grade at the edge of the new fully paved shoulder (0.5 to 1.6 m of grade change);
- Increases in grade along the widened side slopes (1.0 to 3.8 m grade change);
- Increases in grade at the toe of the existing slope (up to 3.8 m grade change); and,
- Increases in loads from subexcavation of 1.8 to 5.5 m of peat and organic soils and replacement with heavier rock fill.

The sources of settlement resulting from the imposed loads which were considered to include:

- Primary time-dependent consolidation of soft cohesive deposits within the footprint of the proposed embankment (predominantly at the toe of the slope in the area to be widened);
- Recompression of the stiff cohesive deposits within the footprint of the widened embankment;
- Primary time-dependent consolidation of approximately 0.3 to 1.3 m of peat and organic soils remaining beneath the existing embankment fills which cannot be removed using OPSD 203.020;
- Secondary time-dependent (creep) consolidation of the cohesive deposits and, where present, peat and organic deposits (long-term) within a 10 year period following completion of construction;
- Immediate settlement of the existing embankment fills; and,
- Self-weight compression of the new embankment fill materials (rock fill) placed at the crest, side slopes and toe of the existing embankment slope.

The thickness of the compressible foundation soils and the height of the embankments vary along the length of the proposed embankment widening and as such the settlements along the length of a given alignment will similarly vary. Given that the analyses were carried out in the critical portions of each section, the settlements estimated will generally represent the upper bound value along a given section of the alignment.

Settlement estimates are provided for the various slope sections at the edge of the fully paved shoulder and at the toe of slope, and are divided into during-construction settlement (within about 2 to 3 months of fill placement) and post-construction settlement (for a 10-year period following the completion of construction). The settlement analyses assume that any surficial or near surface organic soils of significant thickness (i.e., greater than about 0.1 m thick) will be removed in their entirety from the footprint of the widening prior to construction of the new embankments (as per OPSD 203.020) and that rock fill will be used for replacement of subexcavated material. The piezometric conditions required in the analyses were based on the groundwater levels noted during drilling which was essentially located at about the level of the natural ground surface at most locations.

6.2.3.2 Parameter Selection

The general settlement parameters used in the analyses, including the simplified stratigraphy and associated deformation and time-rate consolidation parameters employed for the different native soil types and embankment fills for the critical sections in each section are given in Table 2. The settlement analyses were carried out using the Boussinesq method for distribution of stresses in the foundation soils.

The immediate compression of existing embankment fills was modelled by estimating an elastic modulus of deformation based on the SPT 'N'-values and using correlations proposed by Das (2002). These estimated values were compared with the typical range of expected values for similar soil types, as outlined in CHBDC (2006) and adjusted, if necessary.

The consolidation settlement of the cohesive deposits was assessed using the results of the laboratory consolidation test and/or in situ field vane tests to estimate the deformation parameters for the cohesive deposits. In addition, the results of the laboratory index testing were also employed to further assess deformation parameters (i.e., recompression and compression indices) using the following empirical correlations:

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

$$C_c = 0.5 \times G_S\,(PI / 100)$$

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

$$C_r = C_c / 10$$

Where: C_c = Compression index (kPa);

w_L = Plastic limit (%);

G_S = Specific gravity of the soil solids;

PI = Plasiticity index (%);

e_0 = Void ratio of the clayey silt to clay deposits; and,

C_r = Recompression index (kPa).

The preconsolidation pressure profile beneath the existing embankment and used in the settlement analyses was established using the results of the oedometer testing as well as correlations with the results of the in situ vane tests, based on a relationship between field vane shear strength and preconsolidation pressure proposed by Leroueil (1990) of:

$$\sigma_p' = s_u / 0.25$$

Where: s_u = Undrained shear strength (kPa); and,

σ_p' = Preconsolidation pressure (kPa)

Where the preconsolidation pressure of the cohesive deposit exceeds the applied effective stress (i.e., where the deposit is being recompressed), it was assumed that the subgrade settlements resulting from recompression of the cohesive deposit would occur quite rapidly, likely almost entirely during embankment construction. Primary consolidation of softer clayey deposits takes longer because pore pressures in the deposit must dissipate before settlement can occur. The rates of primary consolidation of these softer deposits were established using the coefficient of consolidation, c_v (cm^2/s), obtained from the results of the consolidation testing.

In addition to primary consolidation and recompression within cohesive deposits, secondary compression may also occur. Secondary compression is referred to as creep settlement and occurs over a long period of time, after full dissipation of excess pore pressure under a constant stress. The following relationships have been employed for estimating the magnitude of creep settlement per the design life following the completion of primary settlement at each location.

$$S_{secondary} = \frac{C_c}{1+e_0}\frac{C_a}{C_c}L_0 log\frac{t}{t_p}$$

Where : $S_{secondary}$ = Secondary consolidation (creep) settlement (mm);

C_c = Compression index (kPa);

e_0 = Void ratio of the clayey silt to clay deposits;

C_α = Secondary compression index of soil ($C_\alpha = 0.04\ C_c$);

L_0 = Initial thickness of compressible clay deposit (mm);

t = Post-construction period (10 years for this project); and,

t_P = Time to reach end of primary consolidation (years).

The consolidation settlement of the thin peat deposit beneath the existing embankment fill was assessed using the results of the laboratory index testing to estimate the deformation parameters for the organic deposits. The following empirical correlations proposed by Mesri (2007) were used:

$$C_c = w_n / 100$$

$$C_\alpha / C_c = 0.06$$

Where: C_c = Compression index (kPa);

w_n = Natural water content (%); and,

C_α = Secondary compression index of soil.

For purposes of the settlement analysis, it was assumed that the peat beneath the embankment is normally consolidated (preconsolidation pressure equal to existing effective stress) and laterally constrained (i.e., vertical settlement only, no lateral deformation). For time-rate settlement analysis, it was assumed that consolidation of the peat would take place quickly and that the time to "End of Primary" consolidation would be within the typical 2 to 3 month construction window.

6.2.3.3 Settlement of Embankment Fill

Where rock fill is used for the construction of the proposed embankment widening, there will be settlement due to compression of the rock fill itself under self weight, in addition to the settlement of the underlying foundation soils as described above. The magnitude of settlement of the rock fill depends on the following factors:

- Type of rock/strength of particles;
- Size and shape of rock particles;
- Gradation of rock fill;
- Total height/thickness of rock fill (stress level); and,
- Method of construction and sequence of placement (including, lift thickness, compactive effort, and state of packing).

Both immediate (i.e., during construction) and post-construction (i.e., creep) settlement of rock fill occurs as a result of re-arrangement of rock particles under load due to wetting and as a result of crushing of rock particles at point contacts (i.e., local crushing and degradation). The magnitude of both the immediate and post-construction settlement have been shown to be approximately a linear function of the fill height, the value of which depends on the method of placement (i.e., compacted versus dumped rock fill) as discussed below.

Compacted Rock Fill

Rock fill should be placed, wherever possible, in a controlled manner (i.e., not end dumped) in accordance with Special Provision 206S03 Earth Excavation, Grading. Blading, dozing and 'chinking' the rock to form a dense, compact mass will be required to minimize voids and bridging and should be used to construct rock fill embankments above the existing groundwater table. The immediate settlement of new granitic rock fill placed in this manner is expected to be nominal and the magnitude may be estimated as a function of the final fill height (i.e., conservatively up to about 0.5 percent of the fill height for fills up to 5 m high, up to about 0.75 percent for fills between 5 m and 10 m high (per "Post-Construction Rock Fill Settlement and Guidelines for Estimating Rock Fill Quantity", prepared by the MTO Research and Development Branch, dated 2010). For this project, it is assumed that limestone bedrock will be used for the widening of the high fill embankments. The self weight compression of compacted limestone fill is not expected to exceed the values predicted for granitic rock fill. This settlement is expected to occur during construction. In addition, some post-construction time-dependent settlement may occur but is expected to be negligible for compacted rock fill and can be estimated as a function of the fill height per log-cycle of time (i.e., less than 0.1 percent of the fill height per log-cycle time).

Dumped Rock Fill

Rock fill that is end dumped into place with little or no control over the lift thickness and compactive effort would be used when backfilling sub-excavated areas below the groundwater table. The immediate settlement of rock fill placed in this uncontrolled manner may be greater than that described above (i.e., about 1 percent for fills up to 5 m thick, about 2 percent for fills between 5 m and 10 m thick and about 3 percent for fills between 10 m to 15 m thick). This settlement is expected to occur during construction. In addition, post-construction time-dependent settlement of dumped rock fill will likely occur but is expected to be relatively small although greater than that of compacted rock fill (i.e., up to 0.3 percent of the fill height per log-cycle time).

6.2.3.4 Settlement Performance Requirements

The following criterion was used as a guide for assessing the mitigation requirements for the design of embankments. Allowable settlements reflect post-paving settlements at the edge of the new fully paved shoulder.

Distance Behind Abutment	Allowable Settlement
0 m to 30 m	10 mm to 25 mm
30 m to 70 m	25 mm to 50 mm
70 m to 170 m	50 mm to 100 mm
Greater than 170 m	100 mm to 200 mm

This performance criterion forms part of the overall design performance for each section of the embankment widening. In general, the recommended mitigation option for each site has been selected such that the remaining primary consolidation settlement and secondary consolidation (creep) settlement is limited to the above criteria over a 10-year period following completion of construction. It has been assumed that construction will take place over a 2 month period. Because of the limited amount of new rock fill being placed at the road level, post-construction settlement due to compression of the rock fill under self weight has only been included in predictions of settlement along the side slopes and at the toe of the embankment.

6.3 Stability and Settlement Mitigation Options

Along the length of the high fill embankment/swamp crossing, the settlement and stability of the widened embankment has been assessed based on existing subsurface conditions and the proposed widening. If left in place, weak, compressible peat and organic-rich soils along the toe of most of the embankment are expected to lead to the potential for instability or unacceptably large settlements with the placement of fills. In some areas, the presence of very soft cohesive soils at the toe of the slope also contributes to instability and large predicted settlements. Settlement at the crest of the embankment is largely controlled by the presence of a thin horizon of peat and organics beneath the existing embankment fills and recompression of the underlying stiff to very stiff cohesive deposit which, in places, is up to 18 m thick.

There are a number of options for mitigating the potential for settlements and/or instability along the widened embankment. A brief general discussion on these alternatives is given below. The various alternatives to mitigate or reduce the magnitude of settlement and increase the stability of the slopes are also presented in Table 3, which provides a comparison of the advantages, disadvantages, and relative costs for the options. Based on a review of issues, it is considered that mitigation of post-construction settlements by subexcavation and replacement of surficial organic deposits is the most feasible alternative for most of the embankment sections. Where identified, settlement resulting from compression of "trapped" organics can be mitigated with preloading of the embankment widening areas, provided that there is sufficient time available during construction before the final paving.

In areas where the foundation soils consist of stiff clayey soils only, embankment stability issues are not anticipated, provided that all significant compressible organic layers at the toe of the slope (i.e., greater than about 0.1 m thick) are removed prior to construction, and that the MTO requirements for mid-height berms are incorporated into the design, as necessary. In areas where very soft cohesive soils are present at the toe of the slope, additional measures will need to be taken to improve both the temporary and long-term stability of the slopes.

A summary of the proposed works, the recommended embankment fill type and side slope, maximum depth of organics encountered, the assumed mitigation option for analysis, and the estimated settlement (during construction and post-construction) is provided in Table 4. Depending on the area, one alternative or a combination of alternatives may be more advantageous than others. The selected mitigation options, as per Section 6.5, are also listed.

6.3.1 Full Sub-Excavation of Compressible Organic Soils

Full sub-excavation of the weak and compressible organic soils underlying the footprint of the proposed widened embankment in advance of the placement of rock fill is a viable option for improving the stability and controlling long-term settlement of the widened embankments along much of this site. The removal of the organics in accordance with OPSD.203.020 would result in improved long-term stability and significantly reduce settlements at the toe of the slope. Given the thickness of the underlying cohesive deposit at this site (up to 18 m in some sections), and its generally stiff to very stiff consistency, it is not considered necessary or practical to fully subexcavate and replace the deposit as a means of reducing long-term settlement or improving settlement.

This option has the advantage that, along most of the corridor, construction of the above-grade widening could proceed upon completion of sub-excavation and replacement of compressible organic soils without concerns for short-term instability. However, sub-excavation will produce a large volume of spoil material for disposal and will require a large volume of rock fill replacement.

Based on the results of this subsurface investigation, the depth to the bottom of the soft, compressible organic soils within the widened swamp crossings varies across the project site, ranging from about 0.6 m to 5.5 m below existing ground surface, but typically less than about 4 m. We understand that, based on MTO field experience on similar highway construction projects, the practical maximum depths that can be reached with conventional and long-stick excavator equipment is about 6 m and 12 m, respectively. As such, sub-excavation of organic deposits and replacement with rock fill is considered a generally feasible option for construction of the roadway embankments and would result in enhanced long-term stability and reduced long-term settlement of the widened embankments.

This option is most suited to areas where there is a limited thickness of soft, compressible soils underlying the footprint of the proposed embankment widening, provided that the requirements for setbacks and adequate right-of-way are available, and there are no conflicts with encroachment on existing adjacent features. In areas where the depth of subexcavation exceeds about 4 m and the existing embankment heights are greater than 4.5 m in height, special precautions will need to be taken to improve the stability of the temporary excavation at the toe of the slope and protect the existing roadway from failures due to temporary loss of toe support.

The advantages of this option are:

- Reduced total settlements of the embankment side slopes and toe;

- Reduced total settlements of travelled portions of the highway for future widening; and,
- Improved long-term embankment stability.

The disadvantages of this option are:

- Risk of instability of existing embankment slopes without appropriate temporary protection measures or limitations on the size, staging, and methodology of the excavation;
- Generation of a large volume of excavation spoil requiring disposal/management;
- Some increase in settlement of existing embankment due to additional loads at toe resulting from replacement of light organic soils with heavier embankment fills;
- The need for a larger corridor of land acquisition; and,
- Greater quantities of rock fill required.

6.3.2 Partial Sub-Excavation of Soft Cohesive Soils

In areas where the organic deposits at the toe of the slope are directly underlain by very soft, relatively thin cohesive deposits which pose a significant stability or settlement concern, consideration could be given to their removal (i.e., partial sub-excavation) and replacement in addition to the full sub-excavation and replacement of overlying organic soils.

Partial sub-excavation would be most appropriate at the western end of the embankment, from about Station 26+825 to 26+925 (Section B), where the soft clays extend to a depth of 8.5 to 9.5 m below ground surface at the toe of the embankment and embankment are high (between 6.5 and 8.5 m in height). In these locations, the width of partial sub-excavation may need to extend beyond the toe of the 1.25H:1V slope to achieve the required minimum factors of safety for global long-term stability of the embankment.

The advantages of this option are:

- Improved long-term stability of widened embankment;
- Reduced requirement for ground improvement, minimizing need for additional right-of-way;
- Reduced total settlements of the embankment side slopes and toe; and,
- Reduced settlements of travelled portions of the highway for future widening.

The disadvantages of this option are:

- Reduced stability of existing embankment during temporary excavation of soils at toe of slope without appropriate and considerable temporary protection;
- Significant difficulties in working below the water table to install temporary excavation support;
- Added costs associated with temporary shoring and dewatering (which may be required during installation of shoring);
- Increased delay in construction associated with excavating the soft cohesive soils;
- Increased quantity of rock fill required; and,
- Increased generation of excess excavation spoil.

6.3.3 Stabilizing Toe Berms

In areas where thick and/or soft clayey deposits (which cannot practically be removed) coincide with high embankment fills, resulting in embankment stability or settlement issues, it may be possible to construct stability berms along the embankment toe and/or to place the embankment fill in stages in layers of limited thickness to ensure that the stability of the widened embankment is maintained. Toe berms consist of rock fill placed along the toe of the widened embankment fill. These berms could be constructed as berms above the ground surface buttressed against the toe of the slope, or as an extended zone of subexcavation and replacement of organic deposits beyond the toe of the embankment. These configurations produce a similar effect (i.e., increased stability) to using flatter embankment slopes but often require less fill material and apply less load (subsequently inducing less settlement) at the crest of the embankment. Depending on the subsurface conditions and the existing embankment height, toe berms will typically be on the order of about one third to one half of the height of the final embankment. The lateral extent (width) of toe berms will vary depending on the results of the stability analyses, but could range from one-half to one times the highway embankment height or greater.

Where strength gain is required within the underlying clayey soils before the full embankment can be constructed, consideration could be given to applying a preload or surcharge (see Section 6.3.5) to the toe berm and allowing a suitable time interval to allow pore pressures to dissipate and strength gain to occur in the underlying clayey soils while limiting the potential for instability of the embankment. In some cases, where lightweight organic soils beyond the toe of the slope provide minimal resisting load and, where the foundation soils are very soft, toe berms may contribute additional driving forces to the slope and result in further instability. In these cases, toe berms are not a feasible way of mitigating stability issues.

The advantages of this option are:

- Improved long-term stability of existing embankment if new geometry with toe berm is stable;
- Improved temporary stability of existing embankment (excavation depth less than with excavation and removal of soft cohesive soils at depth);
- Requires less fill than using flatter slopes; and,
- Reduced post-construction settlements of travelled portions of the highway for future widening.

The disadvantages of this option are:

- Potential reduction in long-term stability of existing embankment in cases where toe berms add additional driving forces which cannot be resisted by lightweight organic soils and soft cohesive soils;
- Increased generation of excess excavation spoil if toe berm is placed as an extended zone of subexcavation and replacement below existing grade;
- Increased quantity of rock fill required for toe berms;
- Small increase in settlements of the toe and lower side slopes of the embankment due to added load of berm;
- Construction is delayed to allow for primary consolidation to be completed and possibly for staged construction (if required); and,
- Additional right-of-way may be required to accommodate the berm.

6.3.4 Preloading

In addition to sub-excavation and replacement of the compressible organic soils (and possibly soft cohesive soils) at the toe of the existing embankment, preloading may be considered for reducing post-construction settlements of the proposed embankments both at the crest and toe of the slope. Preloading refers to the placement of rock fill to the proposed height of embankment (in one or more stages) in advance of pavement construction in order to preconsolidate the underlying compressible soils. Preloading reduces the magnitude of long-term, post-construction settlements by promoting such settlements to occur under embankment fill loads in advance of final grading of the embankment. It also increases the strength of the clayey subsoils underlying the embankment footprint, thereby improving stability and may be used in combination with staged construction to improve the strength of underlying weak cohesive deposits.

Preloading may also be advantageous as a means of identifying and mitigating the additional settlements resulting from the potential presence of a "trapped" wedge of organics beneath the embankment (see Section 6.6.4). To mitigate settlement of these potentially highly compressible materials, the embankment could be constructed and allowed to sit with appropriate monitoring for a period of time equal at least the expected "End of Primary" for the underlying soft clayey soils at the toe (i.e., about 1 to 2 months). Given the permeability of highly organic soils, it is expected that settlement would take place relatively quickly. If significantly more settlement is realized at the end of this time frame than would be expected from compression of the soft silty clay deposits alone, it is likely that such a trapped zone exists, and that additional preloading may be required to minimize post-construction settlement.

Preloading requires placement of embankment fill and monitoring of settlements, and possibly pore pressures, for a period of time corresponding either to approximately the 'End of Primary' (EoP) consolidation of clayey subsoils or, where primary consolidation occurs quickly, a time period sufficient to reduce secondary settlements to within allowable tolerances. Required preloading times will vary depending on the properties of the clayey subsoils, the thickness of the clayey deposits, and the degree of the embankment widening. Once the estimated EoP consolidation (or target settlement) has occurred, final grading for construction can proceed. Long-term secondary consolidation (creep) settlements will still continue to occur over the design life of the embankment, however, such settlements would be less than primary consolidation settlements. Where secondary consolidation (creep) settlements are considered to be large enough to effect the long-term performance of the roadway, these can be further reduced by surcharging as discussed in Section 6.3.5.

Preloading is most suited for areas where "trapped" organics are observed at the toe of the slope which cannot be removed safely prior to embankment construction. This option can also be considered for areas where post-construction settlements are in excess of acceptable tolerances but where, with sufficient time, settlements could be reduced to within acceptable levels, or where increases in shear strengths are required in underlying clays to meet stability requirements. Delays to the construction schedule will need to be accommodated, where necessary.

The advantages of this option are:

- Reduced magnitude of long-term, post-construction settlements by promoting such settlements to occur under embankment fill loads in advance of final grading of the embankment;
- Cost effective method of dealing with settlements; and,
- Improved strength of underlying cohesive soils at the toe, resulting in improved short and long-term stability of the widened embankment.

The disadvantages of this option are:

- Construction is delayed to allow for primary consolidation to be completed and possibly for staged construction (if required);
- An instrumentation and monitoring program would be required to assess when EoP consolidation is reached (as discussed in Section 6.3.7); and,
- Regrading is required to account for settlement prior to construction of the final pavement structure.

6.3.5 Surcharging

Similar to preloading, surcharging refers to the placement of embankment fill in advance of final pavement construction to reduce long-term, post-construction settlements (including creep). The difference between preloading and surcharging is the amount of fill placed and the time required for consolidation to be achieved. With surcharging, the preload is placed as described above, followed by an additional lift of fill (the surcharge) above that required to construct the final embankment geometry. This additional lift of fill applies greater stress to the underlying clayey soils and increases the rate of primary consolidation over that achieved by preloading only, resulting in over-consolidation of the underlying compressible foundations soils. At the EoP consolidation, the portion of the surcharge fill remaining above the required embankment height (sub-base level) is removed. The surcharge fill can also be left in place for a longer duration to reduce the long-term, secondary consolidation (creep) settlements.

If surcharges are placed at the crest of the slope, it may be necessary to construct toe berms or stage the placement of preload and surcharge to limit the potential for instability. Upon completion, the removed surcharge may be re-used on other parts of the site.

Surcharging is most suited to those areas considered appropriate for preloading, but where sufficient time for primary consolidation settlements to occur under preload fill loads alone is not available. Surcharging is most effective in shorter embankments where the increase in load resulting from the surcharge felt by the compressible soils at depth is significant and where the stability of the higher surcharged embankment can be practically maintained by reasonably sized toe berms or staged construction. Surcharging is also suited to the toe of the slope, where strength gain in the clay away from the toe of the embankment is required to improve the embankment stability before the widened embankment can be constructed.

The advantages of this option are:

- Reduced magnitude of long-term, post-construction settlements;
- Decreased delay time for construction over preloading alone;
- Improved stability of embankment resulting from improved strength of soft cohesive soils at the toe of slope, if surcharge is placed at the toe; and,
- Reduced width of toe berm (if required) if toe berm is surcharged, and associated reductions in excavation spoil, quantity of rock fill and time to construct toe berm.

The disadvantages of this option are:

- Ineffective in higher embankments with narrow widenings;

- Construction is delayed, albeit less than for preloading, to allow for primary consolidation to occur;
- Longer construction time if staged construction is required;
- Larger quantity of rock fill if toe berms are required for stability, as compared to preloading alone;
- An instrumentation and monitoring program would be required to assess when EoP consolidation is reached (as discussed in Section 6.3.7); and,
- Increased handling of rock fill (or Granular 'B') to remove the surcharge.

6.3.6 Lightweight Fill

Another alternative for reducing the magnitude of long-term settlement and improving stability in areas of soft, compressible foundation soils is to use lightweight fill, such as expanded polystyrene (EPS) or slag, for embankment construction.

The use of lightweight fill reduces the load applied to the foundation soils due to the low density of the fill materials. This in turn reduces the magnitude of post-construction settlement and reduces the potential for instability.

Lightweight fill is not considered a practical option for general use over large areas due to the expense and/or shipping costs for the supply of these types of fills. It is also not considered a practical option for use below the water table (EPS) or in environmentally sensitive watersheds (slag). Rather, lightweight fill is most suited where existing structures (e.g., culvert beneath the embankment) cannot tolerate differential settlement, or for areas underlain by deep compressible subsoil conditions, where sub-excavation is not practical or feasible, and where there is no available time in the construction schedule for a preload or surcharge period.

The advantages of this option are:

- Improved stability;
- Reduced total settlements at the crest and toe of slope;
- No significant delay in construction; and,
- Elimination of the need for stabilizing toe berms.

The disadvantages of this option are:

- Significant additional expense of lightweight fill (depending on the volume required);
- Not feasible to install in low height embankments (due to minimum conventional soil cover requirements over EPS); and,
- Cannot be used below the water table or in environmentally sensitive watersheds.

6.3.7 Instrumentation and Monitoring

For some areas where the preloading and/or surcharging options are adopted and in all areas where staged construction is adopted, the magnitude and time rate of settlement as well as dissipation of pore pressures during and after construction of embankments should be assessed with monitoring instrumentation. Such monitoring could consist of installing settlement pins (SPs) and/or settlement rods (SRs) within the existing embankment (and at the toe) and taking regular measurements/readings at given intervals of time during and after construction of the widened embankment for the duration of the preloading/surcharging period. In addition, standpipe piezometers (SSPs) may be required and are usually installed to provide background pore pressure readings for the vibrating wire piezometers. This monitoring instrumentation is particularly important where it is considered necessary to carefully monitor the stability of the subsoils during staged placement of fill.

In areas where slope stability is a concern, slope inclinometers can be used to monitor relative horizontal movement of soil layers.

The extent of instrumentation and the frequency of monitoring required will depend on the foundation treatment alternative chosen for a given section and the height of the embankment. Specifications for the type, number and layout of the instrumentation, together with the supply, installation, protection and monitoring should be included as Special Provisions in the Contract Documents.

6.3.8 Ground Improvement

In areas where more conventional stabilization methods are not feasible, consideration could be given to ground improvement as a means of stabilizing the widened slope and reducing excessive settlement at the toe of the slope. Various ground improvement alternatives (e.g., driven timber piled foundations, rammed aggregate piers, stone columns, jet grouted columns, deep soil mix columns) could be considered as a means of improving the strength and stiffness of the weak and compressible soils.

This option is most suited for areas where weak and/or compressible soils result in unstable slopes and these soils cannot be removed without significant temporary shoring to stabilize the existing embankment. This option can also be considered for areas where post-construction settlements are in excess of acceptable tolerances.

The advantages of this option are:

- Improved long-term stability of existing and widened embankment;
- Minimal risk to the existing embankment during construction of the improved zone;
- Improved temporary stability of existing embankment by reducing (or eliminating) the depth of subexcavation and replacement at the toe;
- Reduced magnitude of long-term, post-construction settlements by shedding loads to stiffer members (where column-supported ground improvement is used); and,
- Minimal impact on construction schedule, as work could be completed in advance of the larger scale embankment widening operations.

The disadvantages of this option are:

- Depending on the selected method of ground improvement, this option may be more costly than temporary shoring;
- Additional investigation, testing and design will be required to develop the most cost-effective design;
- A specialty contractor would need to be retained to carry out ground improvement works; and,
- A separate instrumentation and monitoring, and testing program would be required to verify the results of the ground improvement.

6.4 Results of Analysis (Original Embankment Geometry and Standard Construction Practices)

The results of the stability and settlement analyses for the high fill embankment widening through the Cataraqui wetlands based on the original embankment widening geometry and assuming standard MTO construction practices are provided in Section 6.4 below. Section 6.5 provides a discussion of the non-standard embankment geometries and construction methodologies subsequently proposed, and the implications of the changes on the stability and settlement analyses results and proposed mitigation measures provided below.

For ease of assessment, the embankment has been divided into six sections from west to east. A summary of the embankment widening, including the existing embankment height, side slopes and proposed widening, relevant boreholes and simplified subsurface conditions for each section is presented in Table 1.

Where the results of stability or settlement analyses indicate that standard embankment design and construction practices will not adequately meet performance requirements, the options and recommendations for achieving the target factor of safety for embankment stability and/or for minimizing the time dependent, post-construction settlements to within acceptable tolerances are also discussed. The advantages, disadvantages, relative costs, and risks/consequences for these mitigation options are summarized in Table 3. A summary of the preferred stability and/or settlement mitigation option, as augmented by the discussions presented in Section 6.5, for each design section is provided in Table 4.

In areas where the foundation soils consist of stiff to very stiff cohesive deposits only, it is anticipated that there will be no significant risk of long-term instability associated with widening of the embankments. Similarly, the settlement of the foundation soils resulting from placement of additional fills in these areas is expected to occur predominantly as recompression during or shortly after construction, with some time-dependent secondary settlement (creep) over time. In some of these areas, the presence of weak/soft cohesive deposits at the toe of slope constitutes zones of potential instability and large time-dependent settlement of the proposed embankments. In these areas, consideration must be given to an enhanced design and/or to follow a construction sequence that will achieve the minimum target factor of safety of 1.3 for the proposed new embankment height and geometry and limit the post-construction settlements and subsequent maintenance on the new roadway pavement structure.

For embankments widened with rock fill, or where organic soils are subexcavated and replaced with rock fill, settlement of the rock fill is also expected due to compression of the rock fill itself (see Section 6.2.3.3). The thickness of new fills at the crest of the embankment slope is generally negligible and as such, post-construction settlements of rock fill at the road level are expected to be negligible. Post-construction rock fill settlements are expected to be greatest along the embankment side slopes near the existing toe of slope, where up to 3.5 m of widening rock fill and up to 5.5 m of new rock fill (for peat replacement) will be placed.

6.4.1 Highway 401 STA 26+720 to 26+825 WBL and EBL (Section A)

In Section A, along Highway 401 westbound and eastbound from about STA 26+720 to 26+825 (immediately east of the CN Rail Overpass Structure), the existing embankment is between about 9 and 11 m in height with side slopes of about 1.25H:1V. The embankment is to be widened by about 2 to 5 m along the westbound lanes (WBL) and by about 1 to 3 m along the eastbound lanes (EBL). The increase in grade at the new edge of fully paved shoulder is between 1.2 and 1.6 m WB and 0.8 to 1.0 m EB, with maximum increase in fill heights on the side slopes of between 2.5 and 3.8 m.

The subsoils in this area at the toe of the existing embankment consist of about 1.8 to 4.6 m of peat and organics (generally increasing in thickness to the east) overlying silty clay. No additional information was obtained on the amount of peat beneath the side slopes at Station 26+800 in Section A, because the south side of the embankment was inaccessible, and excavations were unable to penetrate the large rock slabs near surface on the north side. The underlying silty clay is stiff to hard in the western portion of Section A, but the upper 1 to 3 m is very soft to firm towards the east end of the section at W3 and E3. At borehole W1 near the existing bridge and railway, 1.2 m of loose silty sand and stiff clayey silt fill was encountered directly overlying hard silty clay. The clayey silt stratum is inferred to be at least 7 m thick based on spoon refusal which was met at W2 at 11.7 m depth, possibly indicating the glacial till or bedrock surface at depth. At borehole B1, put down through the embankment as part of the neighbouring CNR Bridge Widening investigation, the fine rock fill extended to a depth of 4.1 m below ground surface and was underlain by generally stiff to very stiff silty clay overlying glacial till at a depth of about 11.4 m. The lower 1.2 m of the silty clay is firm, with an undrained shear strength of about 40 kPa.

In keeping with Section 6.2.1 on embankment fill types, the widened embankment was analysed assuming a rock fill composition and 1.25H:1V side slopes. The stability and settlement analyses assume that the organic soils encountered at or below ground surface have been removed (in accordance with OPSD 203.020) prior to construction of the widened embankment. Because of the proximity to the CNR overpass structure, allowable settlements within this section were limited to less than 25 to 50 mm at the west end and 50 to 100 mm further east. The simplified stratigraphy and the associated unit weight, strength, deformation and time-rate consolidation parameters employed for the different soil types encountered in this area are summarized in Table 2. The piezometric condition used in the analyses was a water table at ground surface at the toe of the slope, based on groundwater levels noted during drilling.

6.4.1.1 Stability

Based on the results of the subsurface investigation and review of the profile drawings, the critical section (i.e., greatest embankment height and/or maximum thickness of soft, compressible foundation soils) in Section A is located at STA 26+800. The stability analysis performed indicates that after the completion of construction (including removal and replacement of the organic deposits), the embankment at the critical section will have a factor of safety (FoS) of greater than 1.3 against deep-seated, global failure surfaces that would impact the operation of the roadway.

Results of pseudo-static seismic slope stability analyses for the above configuration also indicate that the embankment side slopes will have factors of safety of greater than 1.1.

6.4.1.2 Settlement

To estimate the magnitude of the anticipated settlements due to new construction, analysis was carried out on the critical section(s) representative of the subsurface conditions within the area, at about STA 26+720, 26+740 and STA 26+800. Because of the adjacent CNR overpass abutment, allowable post construction settlements at STA 26+720 are limited to 10 to 25 mm. Allowable post construction settlements at STA 26+740 are limited to 25 to 50 mm, increasing to 50 to 100 mm east of STA 26+770.

Based on the results of the analysis of the critical sections at STA 26+720 and 26+740, the total amount of settlement of the foundation soils during construction resulting from widening of the embankment by about 4 m and excavation and replacement of up to 4.6 m of peat and organic soils is predicted to be about 55 mm at the edge of the new fully paved shoulder (FPS). This settlement is comprised of:

- About 35 mm of primary consolidation of peat and organics beneath the embankment;
- About 10 mm of recompression of the stiff to very stiff cohesive deposit; and,
- About 10 mm of immediate settlement of the embankment fills.

The total amount of post-construction settlement of the foundation soils resulting from widening, excavation and replacement of peat is predicted to be about 25 mm at the edge of the new fully paved shoulder. This settlement is comprised of:

- Up to about 25 mm of secondary compression (creep) of the cohesive deposit in the 10 year period following construction.

The amount of settlement within the existing and new travelled lanes is expected to be significantly less than that estimated at the edge of the fully paved shoulder during and within the 10 year period following construction.

Settlement of the foundation soils in the lower portions of the side slopes near the toe of the embankment is predicted to be significantly more than that at the crest of the slope. This is in large part due to the presence of layer of firm cohesive soils at the toe of the existing slope and the substantially larger relative increase in effective stress due to widening and replacement of peat with rock fill immediately above these soils. The total amount of predicted settlement at the toe of the slope is about 260 mm, comprised of:

- Up to 45 mm of primary consolidation of the soft cohesive soils at toe of slope;
- Up to about 65 mm of immediate self-weight compression of the rock fill at the toe;
- About 40 mm of recompression of the underlying stiff to very stiff cohesive deposit;
- About 30 mm of secondary compression (creep) of the cohesive deposit during construction;
- About 40 mm of secondary compression (creep) of the cohesive deposit after construction; and,
- About 40 mm of rock fill settlement within the 10 years following construction.

Based on an average coefficient of consolidation (c_v) of about 0.002 cm^2/s estimated for the cohesive deposit for the imposed loading conditions and assuming two-way drainage of the approximately 1.6 m thick firm cohesive deposit, it is estimated that 90 percent of the primary consolidation settlement at the toe of the slope will be completed in about 2 to 3 months and, as such, will likely be largely completed during construction. It is assumed that the recompression of the cohesive deposit will take place largely within the construction window.

Rock fill settlement during construction is estimated to be up to about 65 mm at the critical section(s) based on placement of up to 3.8 m of rock fill on the side slopes and up to an additional 4.6 m at the toe after removal of the organic deposits. The magnitude of post-construction settlement of the rock fill is estimated to be about 20 mm per log-cycle of time for this area and as such, approximately 40 mm of rock fill settlement is expected to occur over a 10-year period following completion of construction. Little to no rock fill settlement is expected near the crest of the slope, given the limited thickness of fills being placed at the road level.

As such, about 180 mm of the predicted settlement is expected to take place within the construction window, with about 80 mm of post construction settlement remaining in the 10 years to follow.

Settlements at the north east portion of Section A are expected to be greater than those presented above, but less than those predicted in Section 6.4.2.2 (Section B). However, because of the increased distance to the CNR overpass abutments, post-construction settlement at the edge of the fully paved shoulder in this portion of Section A are expected to be less than the allowable tolerances of 50 – 100 mm.

6.4.1.3 Mitigation of Stability Issues and/or Time Dependent Settlements

Within 30 to 70 m of the existing bridge, the presence of an up to 1.2 m thick firm cohesive deposit and 6.1 m thick stiff clayey silt to silty clay deposit beneath the existing embankment fills influences the magnitude of post-construction settlement of the widened embankment. Predicted settlements at the edge of the fully paved shoulder and within the existing roadway are just within the allowable post-construction settlement tolerances as outlined in Section 6.2.3.4 for the section which is 30 to 70 m from the CNR overpass abutments. Larger settlements are expected at the toe of the slope (particularly where organics at the toe will be excavated and replaced), but are largely expected to take place during construction.

The alternatives have been evaluated on the basis of the advantages, disadvantages, relative costs and risk/consequences and, in this section, the preferred mitigation method is the full sub-excavation of organics at the toe. Provided the up to 4.6 m thick organic deposits at the toe of the slope are removed and replaced with rock fill prior to embankment widening, no additional stability or settlement mitigation measures are required. However, considering the depth of the organic deposits and proximity of the excavation to the existing highway embankment, staged excavation will be required to maintain temporary stability and to protect the existing roadway and temporary protection systems may be required. Details regarding the recommendations for staged excavation of organics are provided in Section 6.6.

Full Sub-Excavation of Compressible Organic Soils

Prior to the construction of the embankment, the removal and replacement with rock fill of up to 4.6 m of organic deposits will be required. Considering the depth of the organic deposits and proximity of the excavation to the existing highway embankment, staged excavation will be required to maintain temporary stability and to protect the existing roadway. Temporary protection systems are recommended along the WB toe where removal depths are greater than 4 m in thickness. Details regarding the recommendations for staged excavation of organics are provided in Section 6.6.

The cohesive deposit beneath the embankment is stiff to very stiff and extends up to about 7.3 m below existing ground surface within the proposed embankment footprint at this location. Full sub-excavation of the cohesive deposit to this depth in this area is not considered feasible and is not considered as suitable settlement mitigation option.

Partial Sub-Excavation of Soft Cohesive Soils

The cohesive soils within the new embankment footprint are generally stiff to very stiff and the widened embankment required no additional stability mitigation aside from removal of organics at the toe of the slope. The cohesive soils are sufficiently stiff that the imposed loads resulting from the widening generally do not exceed the preconsolidation pressure of the cohesive deposit except for the lower 2.6 m of the deposit. As such, partial sub-excavation of a portion of the silty clay is not practical and would only result in a decrease of post-construction settlements of about 30 mm.

Preloading

The cohesive soils beneath the travelled portion of the embankment footprint are generally stiff to very stiff. Recompression of these deposits due to additional loading is expected to take place quickly and predicted settlements at the crest are expected to be within allowable tolerances.

Surcharging

Given the absence of stability or settlement issues associated with the western portion of the proposed embankment geometry, surcharging is not considered appropriate for this section.

Lightweight Fill

Given the absence of stability or settlement issues associated with the proposed embankment geometry, the use of expensive lightweight fill (i.e., expanded polystyrene (EPS)) is not considered necessary or practical for this area.

Ground Improvement

Given the absence of stability or settlement issues associated with the proposed embankment geometry, ground improvement is not considered necessary or practical for this area.

6.4.2 Highway 401 STA 26+825 to 26+925 WBL and EBL (Section B)

In Section B, along Highway 401 westbound and eastbound from about STA 26+825 to 26+925, the existing embankment is between about 6.5 and 8.5 m in height with side slopes of about 1.25H:1V to 1.5H:1V. The embankment is to be widened by about 3 to 4 m along the eastbound and westbound lanes. The increase in grade at the new edge of fully paved shoulder is about 1.2 m WB and 0.6 to 0.8 m EB, with maximum increase in fill heights on the side slopes of about 3.2 m.

The subsoils in this area at the toe of the existing embankment consist of about 4.3 to 6.1 m of peat and organics overlying silty clay. At boreholes W4 and E5, the peat was overlain by 0.6 m of sandy fill. At test pits advanced at the base of the north (westbound) slopes, 0.3 to 1.2 m of peat and greater than 0.3 or 1 m of marl was encountered below the sideslopes, although further advance below 2.1 and 3.0 m depth was limited by the presence of rock slabs near surface. Along the south (eastbound slopes), the base of the embankment side slopes could not be accessed due to open water. The upper 3 to 4 m of silty clay underlying the peat and organics at the toe is very soft to soft (undrained shear strengths between 6 and 20 kPa), becoming stiff to very stiff (undrained shear strengths greater than 60 kPa) with depth. The clayey silt stratum is inferred to be some 16 m thick at the toe of slope based on dynamic cone penetration testing (DCPT) at boreholes W5 and E5. The DCPT met refusal at 21.5 m depth at W5, possibly indicating the glacial till or bedrock surface at depth. At borehole S1 put down through the embankment in Section B, the embankment was found to be underlain by 7.5 m of additional embankment fill below the adjacent ground surface elevation. The depth of embankment fill beneath the existing ground surface is significant and the elevation of the base of the fills corresponds well with the base of the soft silty clay at the EB and WB toe of slope in this section. This indicates that both the compressible organics and the underlying soft cohesive soils were subexcavated and replaced during the original construction of the embankment. At borehole S1, the embankment fill is underlain by peat which was not fully penetrated but, based on the results of similar boreholes through the embankment; we have assumed is 1 m in thickness. Below the peat, we have assumed that the silty clay beneath the embankment is stiff to very stiff, with undrained shear strengths in excess of 100 kPa. The total thickness of the silty clay beneath the embankment is inferred from DCPT refusal at W5 to be 13 m thick.

In keeping with Section 6.2.1 on embankment fill types, the widened embankment was analysed assuming a rock fill composition and 1.25H:1V side slopes. The stability and settlement analyses assume that the organic soils encountered at or below ground surface have been removed (in accordance with OPSD 203.020) prior to construction of the widened embankment. The allowable settlements within this section were limited to no more than 50 to 100 mm along the northwestern portion of this section and 100 to 200 mm along the remainder of the section. The simplified stratigraphy and the associated unit weight, strength, deformation and time-rate consolidation parameters employed for the different soil types encountered in this area are summarized in Table 2. The piezometric condition used in the analyses was a water table at ground surface at the toe of the slope, based on groundwater levels noted during drilling.

6.4.2.1 Stability

Based on the results of the subsurface investigation and review of the profile drawings, the critical sections (i.e., greatest embankment height and/or maximum thickness of soft, compressible foundation soils) in Section B are located at STA 26+850 and STA 26+900.

The existing stability of Section B is also thought to be marginal (FoS 1.25 to 1.37), but is largely contingent on the assumptions made about the location of the soil/rock interface beneath the embankment sideslopes. The lower FoS indicated by the results of stability analysis (for both the existing and widened slopes Section B) is further supported by evidence of previous stability issues at this location, including:

- Very soft, and potentially remoulded clay encountered at the toe of slope beneath the thick organic soils in this area;
- The significant overexcavation of the subsoils beneath the embankment and replacement of the upper zone of the silty clay (which corresponds in elevation to the same zone of very soft clay encountered at the toe of slope north and south of the existing embankment), as indicated by borehole S1; and,
- A bowl shaped section of raised peat at the toe of slope, which is visible in air photographs dating back to about the time of construction of the original highway, but is not visible before that time.

The stability analysis performed on the critical section(s) indicates that after the completion of construction (including removal and replacement of the organic deposits), the widened embankment will also have a factor of safety (FoS) of **less than** 1.3 or greater for deep-seated, global failure surfaces that would impact the operation of the roadway.

As such, special mitigation measures will need to be taken to improve the stability of the widened embankment.

6.4.2.2 Settlement

To estimate the magnitude of the anticipated settlements due to new construction, analysis was carried out on the critical section(s) representative of the subsurface conditions within the area, at about STA 26+850. Because of the adjacent CNR overpass abutment, allowable post construction settlements between STA 26+770 and 26+870 are limited to 50 to 100 mm, increasing to 100 to 200 mm east of STA 26+870.

Based on the results of the analysis of the critical section, the total amount of settlement of the foundation soils during construction resulting from widening of the embankment by about 4 m, excavation and replacement of 5.5 m of peat is predicted to be about 65 mm at the edge of the new fully paved shoulder (FPS). This settlement is comprised of:

- About 25 mm of primary consolidation of peat and organics beneath the embankment;

- About 30 mm of recompression of the stiff to very stiff cohesive deposit; and,
- About 10 mm of immediate settlement of the embankment fills.

The total amount of post-construction settlement of the foundation soils resulting from widening, excavation, and replacement of peat is predicted to be about 90 mm at the edge of the new fully paved shoulder. This settlement is comprised of:

- About 50 mm of secondary compression (creep) of the peat and organics beneath the embankment in the 10 year period following construction; and,
- About 40 mm of secondary compression (creep) of the cohesive deposit in the 10 year period following construction.

The amount of settlement within the existing and new travelled lanes is expected to be significantly less than that estimated at the edge of the fully paved shoulder during and within the 10 year period following construction. At the edge of pavement of the outside travelled lane, settlements are predicted to be about 60 percent of those at FPS (40 mm during construction and 55 mm post-construction). At the centres of the three travelled lanes, settlement is predicted to decrease from about 45 percent at the centre of the outside lane to about 20 percent at the centre of the inside lane (i.e., 30 to 15 mm of settlement during construction, and 40 to 20 mm in 10 years following construction).

Settlement of the foundation soils in the lower portions of the side slopes near the toe of the embankment is predicted to be significantly more than that at the crest of the slope. This is in large part due to the presence of soft compressible cohesive soils at the toe of the existing slope and the substantially larger relative increase in effective stress due to widening and replacement of peat with rock fill immediately above these soils. The total amount of predicted settlement at the toe of the slope is about 550 mm, comprised of:

- Up to 250 mm of primary consolidation of the soft cohesive soils at toe of slope;
- Up to about 75 mm of immediate self-weight compression of the rock fill at the toe;
- About 75 mm of recompression of the underlying stiff to very stiff cohesive deposit;
- About 20 mm of secondary compression (creep) of the cohesive deposit during construction;
- About 90 mm of secondary compression (creep) of the cohesive deposit after construction; and,
- About 40 mm of rock fill settlement within the 10 years following construction.

Based on an average coefficient of consolidation (c_v) of about 0.006 cm^2/s estimated for the cohesive deposit for the imposed loading conditions and assuming two-way drainage of the approximately 2.4 m thick soft cohesive deposit, it is estimated that 90 percent of the primary consolidation settlement at the toe of the slope will be completed in about 1 to 2 months and, as such, will likely be mostly completed during construction. It is assumed that the recompression of the cohesive deposit will also take place largely within the construction window.

Rock fill settlement during construction is estimated to be up to about 75 mm at the critical section(s) based on placement of up to 3.2 m of rock fill on the side slopes and up to an additional 5.5 m at the toe after removal of the organic deposits. The magnitude of post-construction settlement of the rock fill is estimated to be about 20 mm per log-cycle of time for this area and as such, approximately 40 mm of rock fill settlement is expected to occur over a 10-year period following completion of construction. Little to no rock fill settlement is expected near the crest of the slope, given the limited thickness of fills being placed at the road level.

As such, about 420 mm of the predicted settlement is expected to take place within the construction window, with about 130 mm of post construction settlement remaining in the 10 years to follow.

The settlement predicted above is considered to be the upper bound for the section, as the critical section takes into account both the greatest amount of widening and thickest organic zone to be subexcavated and replaced. Settlements along the remainder of the section should be less than those indicated.

6.4.2.3 Mitigation of Stability Issues and/or Time Dependent Settlements

The presence of the 3 to 4 m thick layer of very soft to soft silty clay (undrained shear strengths between 6 and 20 kPa) at the toe of the slope, in combination with the added loads imposed by the widened embankment and replaced organic soils, results in an unstable slope configuration which does not meet the minimum required factor of safety against failure of 1.3 under static loading conditions.

The presence of an up to 1 m thick organic deposit and 13 m thick stiff clayey silt to silty clay deposit beneath the existing embankment fills influences the magnitude of post-construction settlement of the widened embankment. Predicted settlements at the edge of the fully paved shoulder and within the existing roadway are near the upper limit of allowable post-construction settlement tolerances as outlined in Section 6.2.3.4 for the eastern limits of the section within 170 m of the new bridge abutments, and within tolerances for the remainder of the section. Significant settlement is expected at the toe of the slope, but it is largely expected to take place during construction.

In order to achieve a safe slope geometry and minimize post-construction settlements, the alternatives presented below can be considered. The alternatives have been evaluated on the basis of the advantages, disadvantages, relative costs and risk/consequences. In order to improve the stability of the slope and minimize post-construction settlements at the toe within this section of the embankment, the preferred mitigation method is the ground improvement of the weak silty clays and full-subexcavation or improvement of the organics at the toe of the slope.

Full Sub-Excavation of Compressible Organic Soils

Prior to the construction of the embankment, the removal and replacement with rock fill of up to 5.5 m of organic deposits will be required. Considering the depth of the organic deposits and proximity of the excavation to the existing highway embankment, staged excavation will be required to maintain stability and to protect the existing roadway. Where excavation depths exceed 4 m, temporary protection systems are recommended. Details regarding the recommendations for staged excavation of organics are provided in Section 6.6.

While full sub-excavation of the compressible organic soils within the widened section will minimize settlements at the toe of the slope, full sub-excavation of organics alone will not increase the factor of safety of the widened embankment sufficiently to meet the minimum required factor of safety.

Partial Sub-Excavation of Soft Cohesive Soils

The cohesive soils within the new embankment footprint are generally stiff to very stiff; however, the presence of a 3 to 4 m zone of soft cohesive at the toe of the slope contributes to the instability of the slopes in this section which will require additional stability mitigation in addition to the standard removal of organics at the toe of the slope. Partial sub-excavation of the soft portion of the silty clay at the toe of the slope (to an elevation of about 67 m) would significantly reduce the amount of predicted settlement at the toe of the slope and would improve the stability of the slope to meet the specified targets, however, a deep excavation (up to 9.5 m deep) adjacent to an embankment which is 6.5 to 8.5 m in height would be required.

In order to carry out this excavation safely without impacting the existing highway, temporary shoring will be required to maintain an adequate temporary factor of safety against failure of the existing embankment. Temporary shoring within this 100 m section on both the east and west side of the highway would likely comprise sheet piling driven to refusal at depth within the stiff to very stiff silty clay or underlying glacial till soils. Given the high lateral loads imposed by the existing embankment and the depth of excavation required in front of the temporary shoring, at least one and possibly two rows of tie-back anchors will be required to secure the sheet piling. Anchors would need to be extended into the bedrock, which is expected at about 21.5 m depth (based on DCPT refusal at boreholes W5 and E5). In addition, vertical members (e.g., steel H-piles) may need to be driven in front of the sheet piles to provide added stiffness. To maximize the resisting forces on the downslope side of the shoring, it is recommended that partial sub-excavation of the soft cohesive soils be carried out in-the-wet. To facilitate installation of the tie back anchors, some temporary dewatering of slot-trenches may be required (particularly for the lower row, if required).

Prior to subexcavation and replacement of the organic and soft cohesive soils, we recommend that the slope be instrumented (see Section 6.3.7) and that regular monitoring be carried out for movements along the existing embankment.

Preloading

The cohesive soils beneath the travelled portion of the embankment footprint are generally stiff to very stiff. Recompression of these deposits due to additional loading is expected to take place quickly and predicted settlements at the crest are expected to be within allowable tolerances.

At the toe of the slope, the soft cohesive deposit contributes much of the settlement at the toe. With an estimated coefficient of consolidation (c_v about 0.006 cm^2/s) for the deposit, it is estimated that 90 percent of the predicted 250 mm of primary consolidation settlement of this deposit will be completed in about 1 to 2 months (i.e., within the construction window), and settlements resulting from secondary consolidation of the deposit are relatively small (about 50 mm in 10 years). As such, preloading within this deposit is not necessary to control post-construction settlement.

In this section, where slope stabilization measures are required, preloading could serve to improve the shear strengths of the underlying cohesive soils, thus improving the overall stability of the embankment. However, the required increase in shear strength of the underlying soils is such that preloading alone (or even widening in stages) would not be sufficient to improve the overall stability of the slope.

Stabilizing Toe Berms

As discussed above in Section 6.4.2.1, the stability of the widened embankment within Section B does not meet the minimum target factors of safety outlined in Section 6.2.2.1. The low factor of safety in this section is largely controlled by the presence of 3 to 4 m of weak cohesive soils (S_u of 6 to 20 kPa) beneath up to 5.5 m of peat at the toe of the slope. Also contributing to the stability issues are the relatively high embankment heights (6.5 to 8.5 m), steep existing side slopes (1.25-1.5H:1V) and the relatively large widening (3 to 4 m).

Analyses were carried out to develop a suitable toe berm geometry which would improve the stability of the slope to meet the minimum target factors of safety. Models were developed for toe berms consisting of an extended zone of subexcavation of the peat and organic deposits below ground surface for a length of up to about 30 m beyond the toe of the new embankment, as well as for above-grade toe berms up to half the embankment height and extending 30 m or more beyond the toe of the new embankment. Based on the results of the analyses it was determined that, due to the presence of lightweight organic soils at the toe of the slope and weak cohesive soils at and beyond the toe of the slope, any berm placed at the toe of the slope would act as an added driving force, further destabilizing the slope. As such, it was concluded that, toe berms would not provide a suitable method of mitigating the stability of the widened embankment.

Surcharging

In view of the minimal settlement at the crest of the slope and the relatively short duration required for end-of-primary consolidation of the clayey deposits at the toe, surcharging is not considered to be the most appropriate mitigation option for reducing settlements in this section.

Consideration was given to surcharging the weak clayey soils at the toe of the slope as a means of improving the shear strengths of the underlying cohesive soils, thus improving the overall stability of the embankment before it is constructed. However, the analysis indicates that the increase in shear strength required within the weak clayey soils at the toe of the slope cannot be reasonably achieved with surcharging and that application of surcharge loads could cause further instability of the existing slope. As such, surcharging is not considered a suitable method for mitigating stability issues within Section B.

Lightweight Fill

Given the extensive area requiring stabilization, it is not considered practical to use expensive lightweight fill (i.e., expanded polystyrene (EPS)) as a means of mitigating stability issues in this section.

Ground Improvement

Analysis indicates that the 3 to 4 m of weak cohesive soils beneath the organic soils at the toe of the slope in Section B creates an unstable slope when this 100 m section is widened by 3 to 4 m. As discussed above, the presence of weak cohesive soils at depth make more conventional means of slope stabilization (toe berms, preloading/surcharging) ineffective and subexcavation of these soils up to 9.5 m deep adjacent to the existing embankment will require extensive temporary shoring to maintain a safe slope.

An alternative to these more conventional stabilization methods which could be considered for this 100 m section of highway is improvement of the shear strength of the 3 to 4 m of weak silty clay deposit by ground improvement. Given the constraints at this site (e.g., thick overlying organic deposits, high water table, weak silty clays extending to up to 9.5 m below ground surface), we have reviewed the various ground improvement alternatives (e.g., driven timber piled foundations, rammed aggregate piers, stone columns, jet grouted columns, deep soil mix columns) and consider deep soil mixing to be the preferred mitigation strategy for this portion of the embankment.

Deep soil mixing (DSM) involves the additions of binders (most commonly combinations of cement, lime, and gypsum) to the in situ soils to create a chemical reaction product which bonds the soil particles together and results in improved soil mass consistency, strength and deformation characteristics. Because DSM methods are fast, clean, flexible, and have minimal impact to the surroundings, such methods are being used in a growing number of civil applications. The improved soil strength and stiffness within the DSM zone can improve the factor of safety against failure resulting from widening of the Highway 401 embankment under both static and seismic loads.

Deep soil mixing methods (DSM) rely on the physical mixing of in place soils with binders using mixing shafts or other means to form in situ columns or panels of strengthened soils. Traditional column-type deep mixing machines, are constructed with mixing shafts consisting of auger cutting heads, discontinuous auger flights and mixing paddles, and can vary from single to 8-shaft configurations depending on the purpose of the deep mixing. The machines lower augers from the surface into the existing ground, the augers are turned and binders are added to the soil in situ, forming a strengthened column of ground. Other technologies allow for the installation of rectangular-shaped panels using cutter wheels to mix the in situ soil with binders. The main advantage of rectangular panel installations over traditional column type installations is the reduced amount of overlap between panels and increased effective width of improvement. The advantage of the cutter over auger soil mixers is the improved ability to key the improved zone into stiffer underlying soils.

The composite strength of the deep soil mix zone is a function of the obtainable field soil-cement strengths and the layout of the columns or panels within the improvement zone (which changes the replacement ratio). For slope stabilization such as that required in this section, treated zones could be constructed using a cellular box pattern involving the construction of a continuous line of DSM columns or panels connected with perpendicular "rib" or "web" segments, as a single wall with perpendicular support members at regular spacings, or as individual barrettes evenly spaced across the area to be improved.

Depending on the layout of the treated zone, the achievable strength of the improved columns or panels, and the replacement ratio required to achieve the required minimum factor of safety, the improved zone would likely span about 10 to 15 m in width from the 1H:1V toe of the existing embankment (similar to that required for standard subexcavation and replacement treatment), and would extend to the base of the weak clays (at about elevation 67 m). To provide basal fixity, the DSM should be embedded a minimum of 1.0 m into the underlying very stiff cohesive soils. Consideration could be given to removing the organics deposits above the weak clays, or to extending the improved zone to the existing ground surface and placing a geogrid above the improved section to span the improved columns and transfer the loads imposed by the widening onto the stiffer members, thus minimizing settlement of the untreated peat and clayey soils and avoiding costs related to removal and replacement of these organics. A schematic section is provided in Drawing 4.

To optimize the design of the improvement zone, additional field and laboratory testing, together with more detailed numerical modelling and analysis would need to be carried out. Golder has previously been involved in design and construction of similar deep soil mix stabilization walls in Canada and has the expertise in house to carry out this work.

6.4.3 Highway 401 STA 26+925 to 27+075 WBL / 27+025 EBL (Section C)

In Section C, along Highway 401 westbound and eastbound from about STA 26+925 to 27+075 WBL and 27+025 EBL, the existing embankment is between about 4.8 and 6.5 m in height with side slopes of between 1.25H:1V and 2H:1V. The embankment is to be widened by about 4 m along the westbound lanes (WBL) and by about 3 to 5 m along the eastbound lanes (EBL). The increase in grade at the new edge of fully paved shoulder is about 1.2 m WB and 0.6 to 0.8 m EB, with maximum increase in fill heights on the side slopes of between 2.6 and 3.2 m.

The subsoils in this area at the toe of the existing embankment consist of about 1.8 to 4.1 m of peat and organics (generally decreasing in thickness to the east) overlying silty clay. Of the two test pits advanced at the base of the westbound (north) embankment side slopes, one met refusal in frozen ground, and one encountered

1.8 m of peat beneath about 0.5 m of rock fill. On the south (eastbound) side of the embankment, the testpit at Station 26+960 was inaccessible due to open water, and the testpit at 27+020 indicated that all but about 0.1 m of peat had been removed. The underlying silty clay is typically stiff to hard, with undrained shear strengths (s_u) typically greater than 60 kPa. At W6, the upper 1 m of the silty clay is firm (s_u = 20 to 35 kPa). At borehole E7, 0.3 m of firm silty clay fill was encountered overlying the peat. The clayey silt stratum is inferred to be up to 18 m thick based on dynamic cone penetration refusal met at 20 m depth at borehole W8 and at 21.5 m depth at borehole W5. The clay is likely underlain by glacial till and/or bedrock. At W7, the top of the peat was 1.2 m below water and ice. Borehole S2, put down at the WB shoulder of the existing embankment at the west end of Section C indicates that the embankment fill extends to about 2 m below the elevation of the toe of slope and is underlain by 1.3 m of firm to stiff peat and organic silty clay overlying very stiff silty clay.

In keeping with Section 6.2.1 on embankment fill types, the widened embankment was analysed assuming a rock fill composition and 1.25H:1V side slopes. The stability and settlement analyses assume that the organic soils encountered at or below ground surface have been removed (in accordance with OPSD 203.020) prior to construction of the widened embankment. Allowable settlements within this section were limited to no more than 100 to 200 mm. The simplified stratigraphy and the associated unit weight, strength, deformation and time-rate consolidation parameters employed for the different soil types encountered in this area are summarized in Table 2. The piezometric condition used in the analyses was a water table at ground surface at the toe of the slope, based on groundwater levels noted during drilling.

6.4.3.1 Stability

Based on the results of the subsurface investigation and review of the profile drawings, the critical section (i.e., greatest embankment height and/or maximum thickness of soft, compressible foundation soils) in Section C is located at STA 26+950. The stability analysis performed on the critical section(s) indicates that after the completion of construction (including removal and replacement of the organic deposits), the embankment will have a factor of safety (FoS) of 1.3 or greater for deep-seated, global failure surfaces that would impact the operation of the roadway.

The results of pseudo-static seismic slope stability analyses for the above configuration indicate that the embankment side slopes will have factors of safety of greater than 1.1 under seismic loading.

6.4.3.2 Settlement

To estimate the magnitude of the anticipated settlements due to new construction, analysis was carried out on the critical section(s) representative of the subsurface conditions within the area, at about STA 26+950.

Based on the results of the analysis of the critical section, the total amount of settlement of the foundation soils during construction resulting from widening of the embankment by about 5 m and excavation and replacement of 4.1 m of peat is predicted to be about 95 mm at the edge of the new fully paved shoulder (FPS). This settlement is comprised of:

- About 40 mm of primary consolidation of peat and organics beneath the embankment;
- About 10 mm of secondary consolidation of the peat and organics beneath the embankment;
- About 35 mm of recompression of the stiff to very stiff cohesive deposit; and,
- About 10 mm of immediate settlement of the embankment fills.

The total amount of post-construction settlement of the foundation soils resulting from widening, excavation and replacement of peat is predicted to be about 110 mm at the edge of the new fully paved shoulder. This settlement is comprised of:

- About 75 mm of secondary compression (creep) of the peat and organics beneath the embankment in the 10 year period following construction; and,
- About 35 mm of secondary compression (creep) of the cohesive deposit in the 10 year period following construction.

The amount of settlement within the existing and new travelled lanes is expected to be significantly less than that estimated at the edge of the fully paved shoulder during and within the 10 year period following construction. At the edge of pavement of the outside travelled lane, settlements are predicted to be about 35 percent of those at FPS (35 mm during construction and 42 mm post-construction). At the centres of the three travelled lanes, settlement is predicted to decrease from about 20 percent at the centre of the outside lane to about 6 percent at the centre of the inside lane (i.e., 20 to 6 mm of settlement during construction, and 25 to 7 mm in 10 years following construction).

Settlement of the foundation soils in the lower portions of the side slopes near the toe of the embankment is predicted to be significantly more than that at the crest of the slope. This is in large part due to the presence of soft compressible cohesive soils at the toe of the existing slope and the substantially larger relative increase in effective stress due to widening and replacement of peat with rock fill immediately above these soils. The total amount of settlement at the toe of slope is predicted to be about 265 mm, comprised of:

- About 120 mm of recompression of the underlying stiff to very stiff cohesive deposit;
- Up to 15 mm of primary consolidation of the soft cohesive soils at toe of slope;
- About 5 mm of secondary compression (creep) of the cohesive deposit during construction;
- Up to about 60 mm of immediate self-weight compression of the rock fill at the toe;
- About 35 mm of secondary compression (creep) of the cohesive deposit after construction; and,
- About 30 mm of rock fill settlement within the 10 years following construction.

Based on an average coefficient of consolidation (c_v) of about 0.002 cm^2/s estimated for the cohesive deposit for the imposed loading conditions and assuming two-way drainage of the approximately 0.6 m thick soft cohesive deposit, it is estimated that 90 percent of the primary consolidation settlement at the toe of the slope will be completed in about 1 to 2 months and, as such, will likely be mostly completed during construction. It is assumed that the recompression of the cohesive deposit will also take place largely within the construction window.

Rock fill settlement during construction is estimated to be up to about 60 mm at the critical section(s) based on placement of up to 3.2 m of rock fill on the side slopes and up to an additional 4.1 m at the toe after removal of the organic deposits. The magnitude of post-construction settlement of the rock fill is estimated to be about 15 mm per log-cycle of time for this area and as such, approximately 30 mm of rock fill settlement is expected to occur over a 10-year period following completion of construction. Immediate settlement of new rock and/or roadway fills at the edge of the new fully paved shoulder is expected to be negligible given that the grade change in this section at the edge of pavement is minimal.

As such, about 200 mm of the predicted settlement is expected to take place within the construction window, with about 65 mm of post construction settlement remaining in the 10 years to follow.

The settlement predicted above is considered to be the upper bound for the section, as the critical section takes into account both the greatest amount of widening and thickest organic zone to be subexcavated and replaced. Settlements along the remainder of the section should be less than those indicated.

6.4.3.3 Mitigation of Stability Issues and/or Time Dependent Settlements

The presence of an up to 1.3 m thick organic deposit and 18 m thick stiff clayey silt to silty clay deposit beneath the existing embankment fills influences the magnitude of post-construction settlement of the widened embankment. Predicted settlements at the edge of the fully paved shoulder and within the existing roadway are within the allowable post-construction settlement tolerances as outlined in Section 6.2.3.4 for the section more than 170 m from bridge abutments. Settlement is expected at the toe of the slope, but it is largely expected to take place during construction.

The alternatives have been evaluated on the basis of the advantages, disadvantages, relative costs and risk/consequences and, in this section, full sub-excavation of organics at the toe is recommended. Provided the up to 4.1 m thick organic deposits at the toe of the slope are removed and replaced with rock fill prior to embankment widening, no additional stability or settlement mitigation measures are required. However, considering the depth of the organic deposits and proximity of the excavation to the existing highway embankment, staged excavation will be required to maintain temporary stability and to protect the existing roadway. Details regarding the recommendations for staged excavation of organics are provided in Section 6.6.

Full Sub-Excavation of Compressible Organic Soils

Prior to the construction of the embankment, the removal and replacement with rock fill of up to 4.1 m of organic deposits will be required. Considering the depth of the organic deposits and proximity of the excavation to the existing highway embankment, staged excavation will be required to maintain temporary stability and to protect the existing roadway. Details regarding the recommendations for staged excavation of organics are provided in Section 6.6.

The cohesive deposit beneath the embankment is stiff to very stiff and extends up to about 18 m below existing ground surface within the proposed embankment footprint at this location. Full sub-excavation of the cohesive deposit to this depth in this area is not considered feasible and is not considered as a suitable settlement mitigation option.

Partial Sub-Excavation of Soft Cohesive Soils

The cohesive soils within new embankment footprint are generally stiff to very stiff and the widened embankment required no additional stability mitigation aside from removal of organics at the toe of the slope. The cohesive soils are sufficiently stiff that the imposed loads resulting from the widening generally do not exceed the preconsolidation pressure of the cohesive deposit except for the upper 0.5 m of the deposit. As such, partial sub-excavation of a portion of the silty clay would only reduce the amount of predicted settlement at the toe of slope by about 15 mm.

Preloading

The cohesive soils beneath the travelled portion of the embankment footprint are generally stiff to very stiff. Recompression of these deposits due to additional loading is expected to take place quickly and predicted settlements at the crest are expected to be within allowable tolerances.

Surcharging

Given the absence of stability or settlement issues associated with the proposed embankment geometry, surcharging is not considered appropriate for this section.

Lightweight Fill

Given the absence of stability or settlement issues associated with the proposed embankment geometry, the use of expensive lightweight fill (i.e., expanded polystyrene (EPS)) is not considered necessary or practical for this area.

Ground Improvement

Given the absence of stability or settlement issues associated with the proposed embankment geometry, ground improvement is not considered necessary or practical for this area.

6.4.4 Highway 401 STA 27+075/025 to 27+175 WBL and EBL (Section D)

In Section D, along Highway 401 westbound and eastbound from about STA 27+075/025 to 27+175 WBL and EBL, the existing embankment is between about 4.0 and 4.8 m in height with side slopes of between 1.25H:1V and 1.75H:1V. The embankment is to be widened by about 4 to 5 m along the westbound lanes (WBL) and by about 3 to 4 m along the eastbound lanes (EBL). The increase in grade at the new edge of fully paved shoulder is less than 0.5 m WB and EB, with maximum increase in fill heights on the side slopes of between 2.0 and 2.6 m.

The subsoils in this area at the toe of the existing embankment consist of about 2.4 to 4.0 m of peat and organics overlying silty clay. On the north side of the embankment, test pits were attempted between Stations 27+120 and 27+137, but could not be advanced because of refusal in frozen ground. On the south (eastbound) side of the embankment, testpits advanced in close proximity to culverts indicated that peat had been removed from the sideslopes in these areas. The silty clay ranges in stiffness from very soft to hard, with undrained shear strengths as low as 10 kPa in the upper 1 to 2 m, increasing in stiffness to greater than 60 kPa with depth. The clayey silt stratum is at least 4 to 6 m thick, but is likely substantially thicker (17 m thick) based on dynamic cone penetration testing carried out in Section C. At W9, the top of the peat was 1.7 m below water and ice.

In keeping with Section 6.2.1 on embankment fill types, the widened embankment was analysed assuming a rock fill composition and 1.25H:1V side slopes. East of STA 27+075 WBL, the slopes were analysed using 2H:1V side slopes, as drawn in the sections provided by MRC. The stability and settlement analyses assume that the organic soils encountered at or below ground surface have been removed (in accordance with OPSD 203.020) prior to construction of the widened embankment. Allowable settlements within this section were limited to no more than 100 to 200 mm. The simplified stratigraphy and the associated unit weight, strength, deformation and time-rate consolidation parameters employed for the different soil types encountered in this area are summarized in Table 2. The piezometric condition used in the analyses was a water table at ground surface at the toe of the slope, based on groundwater levels noted during drilling.

6.4.4.1 *Stability*

Based on the results of the subsurface investigation and review of the profile drawings, the critical section (i.e., greatest embankment height and/or maximum thickness of soft, compressible foundation soils) in Section D is located at STA 27+100. The stability analysis performed on the critical section(s) indicates that after the completion of construction (including removal and replacement of the organic deposits), the embankment will have a factor of safety (FoS) of 1.3 or greater for deep-seated, global failure surfaces that would impact the operation of the roadway.

The results of pseudo-static seismic slope stability analyses for the above configuration also indicate that the embankment side slopes will have factors of safety of greater than 1.1 under seismic loading

6.4.4.2 *Settlement*

To estimate the magnitude of the anticipated settlements due to new construction, analysis was carried out on the critical section(s) representative of the subsurface conditions within the area, at about STA 27+100.

Based on the results of the analysis of the critical section, the total amount of settlement of the foundation soils during construction resulting from widening of the embankment by about 4.5 m and excavation and replacement of 4.0 m of peat is predicted to be about 110 mm at the edge of the new fully paved shoulder (FPS). This settlement is comprised of:

- About 60 mm of primary consolidation of peat beneath the embankment;
- About 10 mm of secondary consolidation of the peat beneath the embankment;
- About 30 mm of recompression of the stiff to very stiff cohesive deposit; and,
- About 10 mm of immediate settlement of the embankment fills.

The total amount of post-construction settlement of the foundation soils resulting from widening, excavation and replacement of peat is predicted to be about 80 mm at the edge of the new fully paved shoulder. This settlement is comprised of:

- About 50 mm of secondary compression (creep) of the peat beneath the embankment in the 10 year period following construction; and,
- About 30 mm of secondary compression (creep) of the cohesive deposit in the 10 year period following construction.

The amount of settlement within the existing and new travelled lanes is expected to be significantly less than that estimated at the edge of the fully paved shoulder during and within the 10 year period following construction. At the edge of pavement of the outside travelled lane, settlements are predicted to be about 50 percent of those at FPS (55 mm during construction and 40 mm post-construction). At the centres of the three travelled lanes, settlement is predicted to decrease from about 33 percent at the centre of the outside lane to about 11 percent at the centre of the inside lane (i.e., 35 to 15 mm of settlement during construction, and 26 to less than 10 mm in 10 years following construction).

Settlement of the foundation soils in the lower portions of the side slopes near the toe of the embankment is predicted to be significantly more than that at the crest of the slope. This is in large part due to the presence of soft compressible cohesive soils at the toe of the existing slope and the increase in effective stress due to widening and replacement of peat with rock fill immediately above these soils. The total amount of settlement predicted at the toe of the slope is about 345 mm, comprised of:

- About 75 mm of recompression of the underlying stiff to very stiff cohesive deposit;
- Up to 130 mm of primary consolidation of the soft cohesive soils at toe of slope;
- About 10 mm of secondary compression (creep) of the cohesive deposit during construction;
- Up to about 60 mm of immediate self-weight compression of the rock fill at the toe;
- About 40 mm of secondary compression (creep) of the cohesive deposit after construction; and,
- About 30 mm of rock fill settlement within the 10 years following construction.

Based on an average coefficient of consolidation (c_v) of about 0.004 cm^2/s estimated for the cohesive deposit for the imposed loading conditions and assuming two-way drainage of the approximately 1.6 m thick soft cohesive deposit, it is estimated that 90 percent of the primary consolidation settlement at the toe of the slope will be completed in about 2 months and, as such, will likely be mostly completed during construction. It is assumed that the recompression of the cohesive deposit will also take place largely within the construction window.

Rock fill settlement during construction is estimated to be up to about 60 mm at the critical section(s) based on placement of up to 2.6 m of rock fill on the side slopes and up to an additional 4.0 m at the toe after removal of the organic deposits. The magnitude of post-construction settlement of the rock fill is estimated to be about 15 mm per log-cycle of time for this area and as such, approximately 30 mm of rock fill settlement is expected to occur over a 10-year period following completion of construction. Immediate settlement of new rock and/or roadway fills at the edge of the new fully paved shoulder is expected to be negligible given that the grade change in this section at the edge of pavement is minimal.

As such, about 275 mm of the predicted settlement is expected to take place within the construction window, with about 70 mm of post construction settlement remaining in the 10 years to follow.

The settlement predicted above is considered to be the upper bound for the section, as the critical section takes into account both the greatest amount of widening and thickest organic zone to be subexcavated and replaced. Settlements along the remainder of the section should be less than those indicated.

6.4.4.3 Mitigation of Stability Issues and/or Time Dependent Settlements

The presence of an up to 17 m thick clayey silt to silty clay deposit and 1.0 m thick peat deposit beneath the existing embankment fills influences the magnitude of post-construction settlement of the widened embankment. Predicted settlements are below allowable post-construction settlement tolerances as outlined in Section 6.2.3.4. Settlement is expected at the toe of the slope, but it is largely expected to take place during construction.

The alternatives have been evaluated on the basis of the advantages, disadvantages, relative costs and risk/consequences and, for this section, the full sub-excavation of compressible organic soils is recommended. Provided the up to 4.0 m thick organic deposits at the toe of the slope are removed and replaced with rock fill prior to embankment widening, no additional stability or settlement mitigation measures are required. However, considering the depth of the organic deposits and proximity of the excavation to the existing highway embankment, staged excavation will be required to maintain temporary stability and to protect the existing roadway. Details regarding the recommendations for staged excavation of organics and weak/soft deposits are provided in Section 6.6.

Full Sub-Excavation

Prior to the construction of the embankment, the removal and replacement with rock fill of up to 4.0 m of organic deposits will be required. Considering the depth of the organic deposits and proximity of the excavation to the existing highway embankment, staged excavation will be required to maintain stability and to protect the existing roadway. Details regarding the recommendations for staged excavation of organics are provided in Section 6.6.

The cohesive deposit beneath the embankment is stiff to very stiff and extends up to about 17 m below existing ground surface within the proposed embankment footprint at this location. Full sub-excavation of the cohesive deposit to this depth in this area is not considered feasible and is not considered as suitable settlement mitigation option.

Partial Sub-Excavation

The cohesive soils within new embankment footprint are generally stiff to very stiff and the widened embankment required no additional stability mitigation aside from removal of organics at the toe of the slope. The cohesive soils are sufficiently stiff that the imposed loads resulting from the widening generally do not exceed the preconsolidation pressure of the cohesive deposit, except for the upper 1.6 m of the deposit at the toe of the slope. Partial sub-excavation of this soft portion of the silty clay could significantly reduce the amount of predicted settlement at the toe of slope by about 130 mm, however, removal would extend the depth of excavation to some 6 m adjacent to the embankment which, without appropriate roadway protection, could lead to potential instability of the existing embankment during excavation.

Preloading

The cohesive soils beneath the travelled portion of the embankment footprint are generally stiff to very stiff. Recompression of these deposits due to additional loading is expected to take place quickly and predicted settlements at the crest are expected to be within allowable tolerances. Settlements at the toe are predicted to be significant, however, primary consolidation settlements are expected to take place quickly (based on c_v about 0.004 cm^2/s) and post-construction settlements are expected to be relatively small. Given the time to realize most of the settlement at the crest and toe, preloading to minimize settlement is not required.

Surcharging

Given the absence of stability or settlement issues associated with the proposed embankment geometry, surcharging is not considered appropriate for this section.

Lightweight Fill

Given the absence of stability or settlement issues associated with the proposed embankment geometry, the use of expensive lightweight fill (i.e., expanded polystyrene (EPS)) is not considered necessary or practical for this area.

Ground Improvement

Given the absence of stability or settlement issues associated with the proposed embankment geometry, ground improvement is not considered necessary or practical for this area.

6.4.5 Highway 401 STA 27+175 to 27+500 WBL and EBL (Section E)

In Section E, along Highway 401 westbound and eastbound from about STA 27+175 to 27+500 WBL and EBL, the existing embankment is between about 3 and 4 m in height with side slopes range between 1.25H:1V and 4H:1V. Along the westbound lanes, the embankment is to be widened by 3 to 4 m between STA 27+175 and 27+275 at slopes ranging from 1.25H:1V to 2H:1V,and reducing to less than 1 m of widening at 4H:1V west of STA 27+300. Eastbound lanes in this section are to be widened 3 to 4 at a side slope of 1.25H:1V between 27+175 and 27+300, and up to 6 m at a side slope of 4H:1V east of STA 27+300. The increase in grade at the new edge of fully paved shoulder is less than 0.5 m WB and EB, with maximum increase in fill heights on the side slopes of between 0.2 and 2.2 m.

Drawings provided by MRC indicate that a rigid frame open culvert (RFO) crosses perpendicular to the highway at about STA 27+340 m. According to the drawings, the culvert dimensions are 1.83 m x 1.22 m x 38.27 m. Based on the proposed widening, the existing culvert will likely need to be lengthened by about 4 to 6 m on the EB toe of slope and by about 2 m on the WB toe of slope. No information was provided on the elevation of the culvert, but it is assumed that the outlet is near ground surface at the toe of the existing embankment. The design of this widened culvert is provided in a separate report.

The subsoils in this area at the toe of the existing embankment consist of fill and/or peat and organics overlying silty clay. Surficial fill encountered at the toe of the WBL (boreholes W13 through W15) ranges in composition from sandy clayey silt to silty clay. The surficial fills are underlain by up to 1 m of firm organic silty clay, which in turn are underlain by inorganic silty clay to clayey silt. Along the remainder of the section, the subsoils typically consist of about 1.7 to 2.4 m of peat, underlain by silty clay which, near the peat/silty clay interface, is frequently organic. At borehole E16 and E17, the peat is underlain by a 0.6 m thick zone of soft to firm clay. The silty clay along the remainder of this section of embankment of ranges from firm to hard, but is typically very stiff, with undrained shear strengths of 100 kPa or more. The silty clay stratum is at least 4 to 6 m thick in most locations, although at E14, refusal to spoon and dynamic cone penetration was met at about 3 m. From borehole W12, it is inferred that the silty clay is about 8 m thick and is likely underlain by glacial till or bedrock. On the north (westbound) side of the embankment, test pits were advanced at three locations to observe the subsurface conditions beneath the embankment sideslopes. At the testpit advanced as Station 27+240 WB, 1.3 m of peat was encountered beneath the rock fill. East of Station 27+350, no peat was indicated. On the south (eastbound) side of the embankment at Station 27+350, 0.8 m of peat was encountered beneath the side slopes. No peat was encountered at Station 27+600 (EB).

In keeping with Section 6.2.1 on embankment fill types, the widened embankment was analysed assuming a rock fill composition. Because the design slopes range in steepness from 1.25H:1V to 4H:1V, the stability analyses were carried out for the steepest slopes, while settlement analyses were carried out for the largest grade increase. The stability and settlement analyses assume that the organic soils encountered at or below ground surface have been removed (in accordance with OPSD 203.020) prior to construction of the widened embankment. Allowable settlements within this section were limited to no more than 100 to 200 mm. Within 25 m of the culvert, differential settlement should be limited to 25 mm. The simplified stratigraphy and the associated unit weight, strength, deformation and time-rate consolidation parameters employed for the different soil types encountered in this area are summarized in Table 2. The piezometric condition used in the analyses was a water table at ground surface at the toe of the slope, based on groundwater levels noted during drilling.

6.4.5.1 Stability

Based on the results of the subsurface investigation and review of the profile drawings, the critical section (i.e., greatest embankment height and/or maximum thickness of soft, compressible foundation soils) in Section E is located at STA 27+250. The stability analysis performed on the critical section(s) indicates that after the completion of construction (including removal and replacement of the organic deposits), the embankment will have a factor of safety (FoS) of 1.3 or greater for deep-seated, global failure surfaces that would impact the operation of the roadway.

The results of pseudo-static seismic slope stability analyses for the above configuration also indicate that the embankment side slopes will have factors of safety of greater than 1.1 under seismic loading.

6.4.5.2 Settlement

To estimate the magnitude of the anticipated settlements due to new construction, analysis was carried out on the critical section(s) representative of the subsurface conditions within the area, at about STA 27+300.

Based on the results of the analysis of the critical section, the total amount of settlement of the foundation soils during construction resulting from widening of the embankment by about 4 m and excavation and replacement of 2.4 m of peat is predicted to be about 70 mm at the edge of the new fully paved shoulder (FPS). This settlement is comprised of:

- About 40 mm of primary consolidation of peat beneath the embankment;
- About 5 mm of secondary consolidation of the peat beneath the embankment;
- About 15 mm of recompression of the stiff to very stiff cohesive deposit; and,
- About 10 mm of immediate settlement of the embankment fills.

The total amount of post-construction settlement of the foundation soils resulting from widening, excavation and replacement of peat is predicted to be about 65 mm at the edge of the new fully paved shoulder. This settlement is comprised of:

- About 50 mm of secondary compression (creep) of the peat beneath the embankment in the 10 year period following construction; and,
- About 15 mm of secondary compression (creep) of the cohesive deposit in the 10 year period following construction.

The amount of settlement within the existing and new travelled lanes is expected to be significantly less than that estimated at the edge of the fully paved shoulder during and within the 10 year period following construction. At the edge of pavement of the outside travelled lane, settlements are predicted to be about 50 percent of those at FPS (35 mm during construction and post-construction). At the centres of the three travelled lanes, settlement is predicted to decrease from about 25 percent at the centre of the outside lane to about 5 percent at the centre of the inside lane (i.e., 20 to 5 mm of settlement during construction, and 15 to less than 5 mm in 10 years following construction).

Settlement of the foundation soils in the lower portions of the side slopes near the toe of the embankment is predicted to be more than that at the crest of the slope due in large part due to the increase in effective stress due to widening and replacement of peat with rock fill immediately above these soils. The total amount of predicted settlement at the toe of the slope is about 115 mm, comprised of:

- About 45 mm of recompression of the underlying stiff to very stiff cohesive deposit;
- About 15 mm of secondary compression (creep) of the cohesive deposit after construction;
- Up to about 35 mm of immediate self-weight compression of the rock fill at the toe; and,
- About 20 mm of rock fill settlement within the 10 years following construction.

Recompression of the stiff cohesive deposit at the toe of slope and immediate self-weight compression of the rock fill is expected to be completed within a 2-month construction window.

Rock fill settlement during construction is estimated to be up to about 35 mm at the critical section(s) based on placement of up to 2.2 m of rock fill on the side slopes and up to an additional 2.4 m at the toe after removal of the organic deposits. The magnitude of post-construction settlement of the rock fill is estimated to be about 10 mm per log-cycle of time for this area and as such, approximately 20 mm of rock fill settlement is expected to occur over a 10-year period following completion of construction. Immediate settlement of new rock and/or roadway fills at the edge of the new fully paved shoulder is expected to be negligible given that the grade change in this section at the edge of pavement is minimal.

As such, about 80 mm of the predicted settlement is expected to take place within the construction window, with about 35 mm of post construction settlement remaining in the 10 years to follow.

The settlement predicted above is considered to be the upper bound for the section, as the critical section takes into account both the greatest amount of widening and thickest organic zone to be subexcavated and replaced. Settlements along the remainder of the section should be less than those indicated.

6.4.5.3 Mitigation of Stability Issues and/or Time Dependent Settlements

The presence of an up to 1.0 m thick peat deposit beneath the existing embankment influences the magnitude of post-construction settlement of the widened embankment. However, predicted settlements are below allowable post-construction settlement tolerances as outlined in Section 6.2.3.4. Settlement is expected at the toe of the slope, but it is largely expected to take place during construction.

In order to minimize post-construction settlements and differential settlement between new and existing lanes, the alternatives presented below can be considered. The alternatives have been evaluated on the basis of the advantages, disadvantages, relative costs and risk/consequences and, in this section, full sub-excavation of compressible organic soils is recommended. Provided the up to 2.4 m thick organic deposits at the toe of the slope are removed and replaced with rock fill prior to embankment widening, no additional stability or settlement mitigation measures are required.

Full Sub-Excavation of Compressible Organic Deposits

Prior to the construction of the embankment, the removal and replacement with rock fill of up to 2.4 m of organic deposits will be required. Details regarding the recommendations for staged excavation of organics are provided in Section 6.6.

The cohesive deposit beneath the embankment is stiff to very stiff and extends up to about 8 m below existing ground surface within the proposed embankment footprint at this location. Full sub-excavation of the cohesive deposit to this depth in this area is not considered feasible and is not considered as suitable settlement mitigation option.

Partial Sub-Excavation of Soft Cohesive Soils

The cohesive soils within new embankment footprint are generally stiff to very stiff and the widened embankment required no additional stability mitigation aside from removal of organics at the toe of the slope. The cohesive soils are sufficiently stiff that the imposed loads resulting from the widening do not exceed the preconsolidation pressure of the cohesive deposit. As such, no partial sub-excavation is required in this section.

Preloading

The cohesive soils beneath the travelled portion of the embankment footprint are generally stiff to very stiff. Recompression of these deposits due to additional loading is expected to take place quickly and predicted settlements at the crest are expected to be within allowable tolerances.

Surcharging

Given the absence of stability or settlement issues associated with the proposed embankment geometry, surcharging is not considered appropriate for this section.

Lightweight Fill

Given the absence of stability or settlement issues associated with the proposed embankment geometry, the use of expensive lightweight fill (i.e., expanded polystyrene (EPS)) is not considered necessary or practical for this area.

Ground Improvement

Given the absence of stability or settlement issues associated with the proposed embankment geometry, ground improvement is not considered necessary or practical for this area.

6.4.6 Highway 401 STA 28+200 to 28+450 WBL/EBL (Section F)

In Section F, along Highway 401 eastbound from about STA 28+200 to 28+450 EBL, the existing embankment is between about 5.8 and 8.5 m in height with side slopes range between 1.25H:1V and 2H:1V (and as shallow as 4H:1V at the west end of the section). Along the westbound lanes, the embankment is to be widened by up to 0.5 m at slopes of 1.25H:1V or 4H:1V, depending on the existing embankment slopes. Eastbound lanes in this section are to be widened by 1 to 4 m at side slopes of 1.25H:1V. The increase in grade at the new edge of fully paved shoulder is up to 0.9 m, with maximum increase in fill heights on the side slopes of between 1 and 3.5 m.

The subsoils in this area at the toe of the existing embankment consist of fill and/or peat and organics overlying silty clay. At boreholes E20 and E21, the subsoils consist of between 1.8 and 4.3 m of peat and organics and up to 0.6 m of soft clay (at E20) overlying generally stiff silty clay (undrained shear strengths > 80 kPa). Boreholes put down west and east of E20/E21 indicate very stiff to hard silty clay at surface. At the eastern portion of this section (E23, E24 near the Cataraqui River Bridge), 0.6 m of surficial fill consisting of clayey silt or peat is underlain by very stiff to hard silty clay. The silty clay stratum is at least 3 to 4 m thick in most locations. At borehole E23, refusal to spoon and dynamic cone penetration was met at about 3 m, likely indicating the top of glacial till or bedrock. Borehole S4 put down at the crest of the embankment in Section F indicates that the embankment is underlain by about 1 m of rock fill, overlying 0.3 m of peat and 0.5 m of organic clay. These potentially compressible deposits overlie very stiff silty clay extending to a depth of about 4 m below original ground surface.

In keeping with Section 6.2.1 on embankment fill types, the widened embankment was analysed assuming a rock fill composition and 1.25H:1V side slopes along the EBL and between 1.25H:1V and 4H:1V along the WBL, as drawn in the sections provided by MRC. The stability and settlement analyses assume that the organic soils encountered at or below ground surface have been removed (in accordance with OPSD 203.020) prior to construction of the widened embankment. Allowable settlements within this section were limited no more than 50 to 100 mm for most of the section, decreasing to between 10 to 25 mm immediately adjacent to the existing Cataraqui River Bridge. The simplified stratigraphy and the associated unit weight, strength, deformation and time-rate consolidation parameters employed for the different soil types encountered in this area are summarized in Table 2. The piezometric condition used in the analyses was a water table at ground surface at the toe of the slope, based on groundwater levels noted during drilling.

6.4.6.1 Stability

Based on the results of the subsurface investigation and review of the profile drawings, the critical section (i.e., greatest embankment height and/or maximum thickness of soft, compressible foundation soils) in Section F is located at STA 28+200. The stability analysis performed on the critical section(s) indicates that after the completion of construction (including removal and replacement of the organic deposits), the embankment will have a factor of safety (FoS) of 1.3 or greater for deep-seated, global failure surfaces that would impact the operation of the roadway.

The results of pseudo-static seismic slope stability analyses for the above configuration also indicate that the embankment side slopes will have factors of safety of greater than 1.1 under seismic loading.

6.4.6.2 Settlement

To estimate the magnitude of the anticipated settlements due to new construction, analysis was carried out on the critical section(s) representative of the subsurface conditions within the area, at about STA 28+250.

Based on the results of the analysis of the critical section, the total amount of settlement of the foundation soils during construction resulting from widening of the embankment by about 4.5 m and excavation and replacement of 4.3 m of peat and organics is predicted to be about 70 mm at the edge of the new fully paved shoulder (FPS). This settlement is comprised of:

- About 40 mm of primary consolidation of peat and organics beneath the embankment;
- About 5 mm of secondary consolidation of the peat beneath the embankment;
- About 15 mm of recompression of the stiff to very stiff cohesive deposit; and,
- About 10 mm of immediate settlement of the embankment fills.

The total amount of post-construction settlement of the foundation soils resulting from widening, excavation and replacement of peat and organics is predicted to be about 45 mm at the edge of the new fully paved shoulder. This settlement is comprised of:

- About 35 mm of secondary compression (creep) of the peat and organics beneath the embankment in the 10 year period following construction; and,

- About 10 mm of secondary compression (creep) of the cohesive deposit in the 10 year period following construction.

The amount of settlement within the existing and new travelled lanes is expected to be significantly less than that estimated at the edge of the fully paved shoulder during and within the 10 year period following construction. At the edge of pavement of the outside travelled lane, settlements are predicted to be about 10 percent of those at FPS (less than 10 mm during construction and less than 5 mm post-construction). At the centres of the three travelled lanes, settlement is predicted to be less than 5 percent of settlements at the FPS (i.e., less than 5 mm during and following construction).

Settlement of the foundation soils in the lower portions of the side slopes near the toe of the embankment is predicted to be more than that at the crest of the slope due in large part due to the increase in effective stress due to widening and replacement of peat and organics with rock fill immediately above these soils, and in places, the presence of up to 0.6 m of soft clay beneath the replaced organics. The total amount of predicted settlement at the toe of the slope is about 135 mm, comprised of:

- About 5 mm of primary consolidation settlement of the soft cohesive soils;
- About 20 mm of recompression of the underlying stiff to very stiff cohesive deposit;
- Up to about 60 mm of immediate self-weight compression of the rock fill at the toe;
- About 10 mm of secondary compression (creep) of the cohesive deposit after construction; and,
- Up to about 40 mm of rock fill settlement within the 10 years following construction.

Based on an average coefficient of consolidation (c_v) of about 0.004 cm^2/s estimated for the cohesive deposit for the imposed loading conditions and assuming two-way drainage of the approximately 0.1 m thick soft cohesive deposit, it is estimated that 90 percent of the primary consolidation settlement at the toe of the slope will be completed in about 1 month, as such, will likely be mostly completed during construction. Recompression of the stiff cohesive deposit at the toe of slope and immediate self-weight compression of the rock fill is expected to be completed within a 2-month construction window.

Rock fill settlement during construction is estimated to be up to about 60 mm at the critical section(s) based on placement of up to 3.8 m of rock fill on the side slopes and up to an additional 4.6 m at the toe after removal of the organic deposits. The magnitude of post-construction settlement of the rock fill is estimated to be about 20 mm per log-cycle of time for this area and as such, approximately 40 mm of rock fill settlement is expected to occur over a 10-year period following completion of construction. Immediate settlement of new rock and/or roadway fills at the edge of the new fully paved shoulder is expected to be negligible given that the grade change in this section at the edge of pavement is minimal.

As such, about 85 mm of the predicted settlement is expected to take place within the construction window, with about 50 mm of post construction settlement remaining in the 10 years to follow.

The settlement predicted above is considered to be the upper bound for the section, as the critical section takes into account both the greatest amount of widening, thickest organic zone to be subexcavated and replaced, and softest foundation soils. Settlements along the remainder of the section should be less than those indicated. Immediately adjacent to the existing bridge structure, settlements associated with the widening are expected to be minimal (< 25 mm) given the limited amount of widening and the very stiff to hard foundation soils beneath the embankment.

6.4.6.3 Mitigation of Stability Issues and/or Time Dependent Settlements

The presence of an up to 1.3 m thick layer of compressible organics beneath the embankment influences the magnitude of post-construction settlement at the crest of the widened embankment. Predicted settlements are below allowable post-construction settlement tolerances as outlined in Section 6.2.3.4.

In order to minimize post-construction settlements, the alternatives presented below were considered. The alternatives were evaluated on the basis of the advantages, disadvantages, relative costs and risk/consequences and in this section, full subexcavation and replacement of the organics at the toe of slope is recommended. Provided that the up to 4.3 m thick layer of peat and organic deposits are removed and replaced with rock fill prior to embankment widening, no additional stability or settlement mitigation measures are required. However, considering the depth of the organic deposits and proximity of the excavation to the existing highway embankment, staged excavation will be required to maintain stability and to protect the existing roadway in portions of this Section. Additional temporary protection is recommended where excavations exceed 4 m depth. Details regarding the recommendations for staged excavation of organics and weak/soft deposits are provided in Section 6.6.

Full Sub-Excavation of Compressible Organic Deposits

Prior to the construction of the embankment, the removal and replacement with rock fill of up to 4.3 m of organic deposits will be required. Considering the depth of the organic deposits and proximity of the excavation to the existing highway embankment, staged excavation will be required to maintain stability and to protect the existing roadway in portions of this Section. Details regarding the recommendations for staged excavation of organics are provided in Section 6.6.

The cohesive deposit beneath the embankment is stiff to very stiff and extends up to about 7.5 m below existing ground surface within the proposed embankment footprint at this location. Full sub-excavation of the cohesive deposit to this depth in this area is not considered feasible and is not considered as suitable settlement mitigation option.

Partial Sub-Excavation of Soft Cohesive Soils

The cohesive soils within new embankment footprint are generally stiff to very stiff and the widened embankment required no additional stability mitigation aside from removal of organics at the toe of the slope. The cohesive soils are sufficiently stiff that the imposed loads resulting from the widening generally do not exceed the preconsolidation pressure of the cohesive deposit. As such, no partial sub-excavation is required in this section.

Preloading

The cohesive soils beneath the travelled portion of the embankment footprint are generally stiff to very stiff. Recompression of these deposits due to additional loading is expected to take place quickly and predicted settlements at the crest are expected to be within allowable tolerances.

Surcharging

Given the absence of stability or settlement issues associated with the proposed embankment geometry, surcharging is not considered appropriate for this section.

Lightweight Fill

Given the absence of stability or settlement issues associated with the proposed embankment geometry, the use of expensive lightweight fill (i.e., expanded polystyrene (EPS)) is not considered necessary or practical for this area.

Ground Improvement

Given the absence of stability or settlement issues associated with the proposed embankment geometry, ground improvement is not considered necessary or practical for this area.

6.4.7 Settlement of the Gas Main at Station 27+756

The existing gas main at Station 27+756 is outside of the area of widening for this project and, as such, no construction induced settlements are expected.

6.4.8 Settlement of Existing Culverts

Foundations input to the design and extension of the three existing structural culverts at Stations 27+340, 27+675 and 28+030 is provided in a separate report.

6.5 Results of Analysis (Non-Standard Embankment Geometries and Construction Methodologies)

In a meeting with MTO, MRC and Golder, the impacts and costs of following standard MTO design and construction practices were discussed and it was concluded by MTO and the design team that alternate non-standard embankment geometries and non-standard construction methodologies should be considered for the construction of the embankment through the Cataraqui wetlands. This decision to consider non-standard methodologies was driven by a number of issues, including:

- The large volumes of peat that would need to be removed as per OPSD 203.020 (high cost);
- The high cost and complexity of mitigating the stability issues in Section B;
- The depth of peat removal which, in some places, exceeds 3 to 4 m and would require temporary shoring and possible lane closures during construction;
- The challenges associated with staging the shaving of existing side slopes to 1H:1V to accommodate subexcavation and backfill in accordance with OPSD 203.020, which is made more challenging because of access issues (the toe of the slope can only be accessed from the east), materials management and adjacent land owner issues; and,
- The changes made to the embankment geometry (roundings changed from 4H:1V to 3H:1V) to minimize the footprint into the fisheries compensation areas.

The non-standard construction methodologies considered by the design team included:

- Staged construction of widened embankment with no removal of peat at toe of slope, to eliminate all subexcavation of organics at the toe of slope and beneath the embankment sideslopes;
- Construction of the widened embankment with only partial removal of peat at the toe of slope, by:
    - Limiting depth of removals to a maximum depth of 3 m to eliminate the need for temporary shoring and minimize the risks of instability to the existing embankment sideslopes; and/or,

  - Leaving a section of trapped organics beneath the existing sideslopes (i.e., changing the requirement for removals from OPSD 203.020 to 203.030); and/or,
  - Limiting the width of swamp excavation to extend only to the toe of the widened embankment slope, but not beyond (i.e., modification to OPSD 203.020, as shown on Drawing 5).
- Modifying the slope geometry to minimize the footprint of the widening such that only minimal new fill would be placed on the existing embankment slopes (particularly in Section B where the existing slope stability is marginal).

A summary of the advantages and disadvantages of these non-standard construction methodologies and embankment geometries have been added to Table 3.

Based on the results of that meeting, the modified embankment widening sections with 3H:1V roundings in Sections C through F, and 3 or 2H:1V roundings in Sections A and B, were assessed with respect to settlement and stability, and a summary of the findings is provided below.

6.5.1 Embankment Stability

The long-term stability of slopes with the original embankment geometry (4H:1V rounding slope) or modified 3H:1V slopes with peat removals at the toe of slope are stable in all sections but Section B, which is underlain by the thickest peat deposits and an underlying very weak clay layer. With further reductions of the slope geometry in Section B (i.e., a 2H:1V rounding), the amount of fill being placed on the existing side slopes (and in particular at the toe of the slope) is significantly reduced and there is only a slight reduction in the existing long-term slope stability of the slopes (1.24 to 1.28, compared to the existing conditions FOS 1.25 to 1.37).

To assess the short-term stability of the embankments constructed on peat (without subexcavation and removal), the undrained shear strengths of the material must be considered. Because the fill at the toe of slope is being placed on virgin peat deposits that are under low existing effective stresses and have not previously been compressed, these deposits have very low initial undrained shear strengths. Mesri (2007) provides a correlation between undrained shear strength and existing vertical consolidation pressure of about 0.55 for fibrous peat deposits with water contents of around 500 percent, which corresponds to an equivalent short-term friction angle of about 29 degrees.

Stability modeling of the proposed widening constructed to full height without peat removals using undrained shear strengths results in very low factors of safety (less than 1) and will result in significant lateral displacement (i.e., spreading type failures) of the peat in Sections C through F. In order to successfully construct on virgin peat without encountering significant stability problems, the widened embankments would need to be constructed in very short lifts (likely no more than 1 m of fill placed at a time), and each lift would need to be left in place until pore pressures dissipated (likely for 1 to 3 months per lift). Instrumentation would need to be installed to monitor pore pressures and lateral movements of the peat to reduce the risk of failures within the new embankment fill. Even if constructed in stages, the peat would undergo significant settlement and lateral displacement (both during and after construction) and significant amounts of extra rock fill will need to be placed below swamp grade as the peat is compressed or displaced laterally.

In Sections A and B, where there is limited fill being placed at the toe of slope in areas with peat and organics and where the widened fill will be placed above a geogrid-reinforced temporary access road at the toe of slope (refer to Section 6.8), very little load will be transmitted to the underlying peat and the short term factor of safety against instability is predicted to be about 1.2, and would increase in time as pore pressures within the underlying peat, organic and clayey soils dissipate.

6.5.2 Embankment Settlement

The settlement analysis presented in Section 6.4 above was carried out based on the original embankment widening geometry with a 4H:1V roundings and assuming full removal of peat beneath the side slopes and at the toe of slope as per OPSD203.020. Results indicated that the settlement at the edge of the fully paved shoulder was generally expected to be within acceptable ranges.

The following discussion provides a summary of the relative changes in predicted settlement outlined in Section 6.4 based on the following changes:

- The change in slope widening geometry from 4H:1V roundings to 3H:1V or 2H:1V roundings;
- The reduction (limitation) of the depth of subexcavation in Sections C to F to no more than 3 m;
- The incorporation of a minimum of 6 months preload period allowed for between construction of the widened embankment and final paving;
- The elimination of subexcavation and replacement of peat and organics with rock fill in Sections B and A;
- The construction of a 1 to 1.5 m high georid-reinforced construction access road at the toe of slope in Sections B and A;
- The potential for limiting the removals to beyond the existing toe of slope (OPSD 203.030), rather than cutting the existing slope at 1H:1V from the crest of slope, as specified in OPSD 203.020; and,
- The modification to OPSD 203.020 to limit the width of widening to extend only to the toe of the widened embankment slope.

The change in slope widening geometry from 4H:1V roundings to 3H:1V or 2H:1V roundings results in a significant reduction in the amount of material being placed on the side slopes (see Table 1 for summary of widths of widening with the 4H:1V and 3 or 2H:1V), and thus a reduction in the overall load being applied to the embankment, which would result in a reduction of the amount of predicted settlement, both at the toe of slope and at the edge of the paved shoulder.

At the toe of slope, settlement predictions provided in Section 6.4 were estimated assuming that all peat and organics were removed and replaced with rock fill. Sources of settlement included recompression or consolidation of the cohesive deposit under the added loads imposed by replacing the peat with rock fill, by constructing the widened embankment, and by the settlement of the rockfill itself. No settlement of the peat at the toe or side slopes was considered because it was to be removed. The reduction in the maximum subexcavation depths to 3 m along the toe of the embankments will remove most of the compressible organics from the toe of slope, however, in a few areas, would result in some compressible organic deposits being left in place beneath the new embankment fills at the toe of slope. As a result, some additional settlement of the embankment side slopes and toe (both during and post-construction) should be expected, although this additional settlement is unlikely to impact the performance of the road platform given that the new travelled lanes are entirely within the existing embankment width. Where not fully removed, ongoing "creep" settlement of the underlying peat/organic materials will occur following paving, and ongoing future maintenance of the sideslopes for the widened portions of the highway in these areas may be required.

Constructing the widened embankment as a preload (i.e., the full height of the embankment is constructed and allowed to sit for a period of at least 6 months to a year before paving) will further reduce the amount of post-construction settlement predicted in Section 6.4 at both the crest and toe of slope, and will allow additional settlement resulting from compression of any trapped organics to take place prior to paving. It is recommended that settlement be monitored over time using settlement pins placed at regular intervals along the widened embankment near the crest of slope. An NSSP for settlement pins is provided.

In Sections A and B, by incorporating a 2H:1V rounding to minimize the amount of additional load being placed on the existing sideslopes, the embankment widening is reduced considerably at the toe of slope (to a sliver widening in most locations underlain by peat and organics). In these sections, leaving some or all of the peat at toe of slope is thought to have minimal impact on predicted settlements at crest of slope because of the steepness and height of the slopes and the limited widening.

With a 1 to 1.5 m high temporary construction road constructed along the toe of slope in Sections A and B, the added loading from temporary construction road is about equivalent to the added load from subexcavation and replacement of the peat with rock fill, resulting in no increase in stress. No new loads added means no change in the predicted settlements at the edge of pavement in these sections as presented in Section 6.4.

Beneath the side slopes, settlements are expected to be between those expected at the edge of the fully paved shoulder and at the toe of slope and will be highly dependent on the amount of trapped organics left in place beneath the side slopes. If slopes are excavated as per OPSD 203.030 rather than OPSD 203.020, there is an increased risk of trapped organics beneath the side slopes, and additional settlement (both during and post-construction) should be expected. Increased post-construction settlement of the toe and side-slopes could result in some oversteepening of the embankment side-slopes. Even with preloading of the embankment widening areas, ongoing "creep" settlement of the underlying peat/organic materials would occur following paving, which may result in potential softening of the embankment shoulders, for which ongoing maintenance will likely be required. If the embankment were widened in the future, the future removal of these additional "trapped" organics would be more difficult and, if left in place, additional settlements of these "trapped" organics would impact the future travel lanes.

6.5.3 Conclusions

The results of the updated settlement and stability analyses were presented to MTO in a meeting with the design team, and the following conclusions were drawn:

- It was decided that standard subexcavation and replacement procedures in accordance with OPSD 203.020 should be followed in Sections C through F, but that subexcavation depths and lengths should be limited to 3 m or less to avoid the need for temporary shoring and reduce the risk of instability to the existing highway embankments. It was further decided that the width of subexcavation should be limited to the widened toe of slope and that steeper, near-vertical excavation sideslopes be used to accommodate property constraints.
- In Section B, where the subexcavation depths exceed practical excavation limits without significant temporary shoring and where the existing stability of the slope is marginal, it was decided by MTO that they could accept a lower factor of safety (around 1.2) to avoid the costs, staging issues and construction risks associated with standard construction practices, and that Section B could be constructed with 2H:1V roundings, without subexcavation of the organics at the toe of slope.

- Similarly, in Section A, where subexcavation of the organics would be challenging to stage and where the embankment widening in areas with peat and organics are of limited width, it was decided that, to avoid the costs, staging issues and construction risks associated with standard construction practices, this area could also be constructed without subexcavation of the organics at the toe of slope.
- To minimize the risk of post-construction settlement (particularly due to trapped organics beneath the side slopes), MTO Foundations requested that a minimum of 6 months preload be allowed for between construction of the widened embankment and final paving.

6.6 Subgrade Preparation Requirements

Surficial peat and organic soils are present along the majority of the proposed embankment widening, as discussed in Sections 4.2 and 6.4 of this report. It is not normal practice to carry out topsoil and organics stripping from below embankments that are greater than 1.2 m in height (OPSS 206 only requires stripping of topsoil below embankments of less than 1.2 m in height), however; if the peat/organic soils were left in place below these embankment widening areas, significant primary consolidation settlement of the peat would occur, followed by ongoing secondary ("creep") settlement would result in poor performance and ongoing maintenance of the widened portions of the highway. Even with preloading and/or surcharging of the embankment widening areas, ongoing "creep" settlement of the underlying peat/organic materials would occur following paving, again resulting in poor performance and ongoing maintenance for the widened portions of the highway in these areas. If the embankment were widened in the future, significant additional settlements should be expected from these "trapped" organics. The peat/organic soils are generally unsuitable for support of the proposed embankment widening and, therefore, it is recommended that the peat and organic soils be subexcavated (to a maximum depth of 3 m) from within the footprint of the new widening areas and replaced with embankment fill. In Sections A and B, where widenings are limited to slivers in areas with peat and organics, these deposits will remain in place.

The following sections address subgrade preparation (peat removal) requirements, including depth of excavation, subexcavation procedures, and associated construction concerns (supplemented with appropriate Operational Constraints) for the proposed embankment widening work.

6.6.1 Depth of Subexcavation

The depth of subexcavation of the topsoil and peat/organic soils, as encountered in the boreholes at the toe of the embankment slope and based on the approach discussed in Section 6.5, is summarized in Tables 5a and 5b following the text of this report for each of the embankment widening areas. As discussed in Section 6.5.3, the depth of subexcavation has been limited to 3 m.

6.6.2 Excavation and Subexcavation Procedures

Construction procedures for the widening of the existing Highway 401 embankment should implement the guidelines of OPSD 203.020, which require that the side slope of the existing Highway 401 embankment be temporarily excavated to a 1H:1V profile to allow subexcavation and replacement of peat/organic material from below the existing embankment fill. As per the discussions in Section 6.5.3, OPSD 203.020 should be modified such that the width of subexcavation should extend only to the widened toe of slope (see Drawing 5) and steeper, near-vertical excavation sideslopes should be used to accommodate property constraints. Following these guidelines may still result in some organic deposits remaining in place below the transition area between the existing and widened embankments; further discussion on this aspect is provided in Section 6.6.4 of this report. Subexcavation and removal of organics is proposed for Sections C through F to a maximum depth of

3 m. Based on the results of the subsurface investigation, peat and organics extend up to approximately 5.5 m below the original ground surface. Peat and organics at the toe of slope in Sections A and B (areas of sliver widening) will be left in place, and in some locations in Sections C and F.

The groundwater table/surface water level is at or near the original ground surface along most of the existing embankment. The subexcavation works could be carried out sub-aqueously below the groundwater/surface water level; alternatively, dewatering of the groundwater/surface water could be carried out prior to and during subexcavation and backfilling works. The use of subaqueous excavation will increase the factor of safety against instability and against excavation base heave as compared with unwatered conditions, and will be less expensive than unwatering/dewatering of the lengths of excavation associated with the embankment widening work. Although a greater level of compaction could be achieved on the backfill with the use of dewatering, it is considered that acceptable performance of the backfill material will be achieved provided that the recommendations provided herein are followed. As such, subaqueous excavation is recommended for this contract.

In order to maintain an adequate factor of safety against instability of the existing Highway 401 embankments, special excavation and subexcavation procedures will be required. An operational constraint has been developed for inclusion in the Contract Documents to address these items, as follows:

- Excavation of the existing embankment fill to a 1H:1V slope within the embankment widening footprint will have to be carried out in sections. For embankment fill heights exceeding 5 m and 7 m, the excavation length (as measured parallel to the highway direction) should not exceed 200 m and 100 m, respectively, for periods not exceeding 6 weeks.
- Where subexcavation depths are greater than 1 m below the original ground surface at the embankment toe, subexcavation of the peat and organic soils within the embankment widening footprint will have to be carried out in short sections perpendicular to the highway alignment, with the subexcavation length (as measured parallel to the highway direction) not more than 3 m at any time.
- Subexcavation and backfilling operations will have to be carried out simultaneously such that the subexcavation is not left open for more than 3 m in length at any given time.
- Subexcavation depths should be limited to 3 m.
- Subexcavation works 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. In this regard, any existing embankment fill or soils above the water table would be classified as Type 3 soil. Temporary excavation through the existing embankment fill shall be made, where required, with side slopes no steeper than 1H:1V from the crest of the existing highway embankment to the toe of slope.
- The width of subexcavation should extend only to the widened toe of slope (see Drawing 5) and steeper, near-vertical excavation side slopes should be used below the native ground surface to accommodate property constraints.
- Some distress to the existing highway embankment may occur during excavation and subexcavation; provisions for traffic control measures shall be included to maintain the safe operation of Highway 401 during excavation and backfilling operations. It is recommended that visual monitoring be completed each

day for any sections of highway embankment adjacent to active subexcavation/backfilling works. This visual monitoring could be completed as part of the Contract Administrator assignment. Provided that excavation depths and sizes are limited as outlined above, it is unlikely that temporary lane closures will be required, however this should be revisited once excavation works have begun and the performance of the embankment side slopes during excavation is observed.

An operational constraint entitled "Swamp Excavation" for the staged subexcavation and subaqueous replacement of the peat and organics as described above, is appended to the report. As part of the Contract Administrator's terms of reference, a Foundations Specialist should be retained by the CA to oversee the subexcavation and replacement operations.

6.6.3 Backfill of Subexcavated Areas

Subexcavation of up to approximately 3 m below the original ground surface will be required in Sections C through F. The groundwater table/surface water level is at or near the original ground surface at all of the swamp/wet ground crossing sites addressed in this report. As discussed in Section 6.6.2, it is recommended that the subexcavation backfilling works will be carried out below the groundwater/surface water level.

Because of the wet conditions, it is recommended that the subexcavation areas be backfilled using Granular "B" Type II fill. This fill material contains limited fine soil particles, and will not tend to segregate during placement; in addition, because this fill material is from a crushed source, the angular particles will perform better than non-crushed (i.e., rounded) soil particles during compaction of subsequent lifts of embankment fill above the water table.

Alternatively, end-dumped rock fill could be used for backfill of the subexcavated areas. There is potential for loss of fine soil particles from the native soils below into voids within the rock fill. A minimum 0.3 m thick "transition layer" of Granular "A" or Granular "B" Type II fill could be placed at the base of the subexcavation as a transition between the native soils and the rock fill. If this transition layer is not placed, allowance should be made for an average of about 0.5 m of rock loss due to punching of the rock fill into the underlying cohesive soils. Long-term settlements which might result from the slow migration of fines from the native cohesive soils into the coarse rock fill are not expected to affect the overall performance of the embankment.

Because of the significant difference in unit weight between Granular "B" or rock fill and peat/organic soils being replaced, the placement of these materials will apply additional load to the underlying fine-grained soils beneath the subexcavated areas and will result in additional consolidation settlement of the underlying silty clay soils over and above that of the embankment fill placed for the widening. These loads have been taken into account in the results of the analyses presented in Section 6.4, based on replacement of peat with rock fill.

6.6.4 Preloading to Mitigate Settlements Resulting from "Trapped Organics"

As discussed in Section 6.1.2, we understand that construction practices when the existing embankment was built included removal of organic deposits within the footprint of the excavation, which was defined by the line which extended down from the edge of granulars at a 1 horizontal to 1 vertical slope. Similarly, as discussed above in Section 6.6.2, the subexcavation and replacement of peat and organics deposits at the toe of the existing slope for the proposed widening will also begin at a line drawn a 1H:1V profile. If the new and old subexcavation limits do not match or overlap, there is potential for a "trapped" wedge of organics beneath the embankment to exist. The presence of such a zone of organics may be able to be observed during subexcavation of the organics by monitoring the conditions at the face of the excavation nearest to the embankment.

It is not advisable to oversteepen the existing side slopes of the embankment beyond the recommended 1H:1V side slopes. As such, if organic deposits are present, these deposits will remain trapped beneath the toe (and side slopes) of the widened embankment. Organic deposits are likely to be highly compressible, and as such, are expected to contribute to additional settlement of the embankment fills along the side slopes and toe are of the new widening. While such settlements are not expected to directly affect the total settlements predicted at the crest of the slope, differential settlement between the crest and toe of the slope could lead to oversteepening and ravelling of the side slopes and, in severe cases, loss of ground or softening of the shoulder at the crest of the slope.

To mitigate potential settlement of these highly compressible materials we recommend that, where "trapped" organics are observed, the embankment be constructed and allowed to sit at full height (except where stability issues require staged construction) with appropriate monitoring for a period of 6 months or longer following construction to allow for settlement of the underlying trapped organics. Given the permeability of highly organic soils, it is expected that most of the primary settlement (and some secondary settlement) of organic soils would take place within this time frame and that post-construction settlements at the toe would be sufficiently minimized to mitigate the potential for ravelling and loss of ground at the crest of slope.

6.7 Embankment Construction

The embankment fill for the widening areas should be placed and compacted in accordance with MTO's Special Provisions 206S03 and 105S10. Benching of the existing embankment side slopes should be carried out to "key in" the new fill materials for the widening, in accordance with OPSD 208.010. Commonly in embankment widening construction, the fill material cut from the existing embankment side slope for creation of these benches is re-used for the embankment widening below/adjacent to each bench area. Additional fill for construction of the embankment widening above the level of the original ground surface (i.e., above the groundwater level) could consist of clean earth fill, granular fill or rock fill.

From a geotechnical/foundations perspective, granular or fine rock fill is preferred for the construction of the embankment widening above the level of the original ground surface (i.e., above the groundwater level), as it will provide better compatibility with the existing embankment fill materials – both those fill materials remaining in-place in the existing embankment side slope, and any existing embankment fill that is re-used for the widening after being cut from the benches. At this site, we understand that rock fill generated from the widening of Highway 401 west of the wetland will most likely be used as embankment fill.

If rock fill is adopted for the embankment widening areas, the native soils beneath the rock fill should first be covered with a minimum 300 mm thick sand and gravel blanket (OPSS 1010 Granular "B" Type II or similar). It is assumed that the new rock fill for the widening generated from the adjacent limestone cuts will be similar in gradation to the fine rock fill within the existing embankment. As such, it is likely that the new fill can be placed adjacent to the existing embankment fill without the need for special grading or separation layers between the new and existing materials. If the new rock fill proves to be significantly coarser than the existing rock fill or the pavement structure, the filter compatibility of the two materials will need to be checked to avoid the potential for migration of soil particles into voids of adjacent layers. If the materials are sufficiently dissimilar, there is a potential for migration of finer particles which could result in settlement/sinkholes propagating to the ground surface and the surface of the rock fill layer will need to be carefully graded and "chinked" or a separation layer placed, before placing any granular fill for the pavement structure.

If earth fill or granular fill is used, placement of topsoil and seeding or pegged sod is recommended to reduce surface water erosion on the widened embankment side slopes.

6.7.1 Sliver Fills

The 2H:1V roundings create sliver fills in Sections A and B which will be challenging to construct using conventional methods. To accommodate the sliver fills, the gradation of the rock fill placed in Sections A and B will need to be limited to 200 mm minus and will need to be benched in to the existing slope using a modified OPSD 208.010. At the toe of slope beneath the placed fill, it is recommended that a biaxial geogrid be placed to improve the local stability, limit punching of the rock fill into the underlying organic deposits, and even out differential settlements along the toe of the slope. This geogrid would also form part of the temporary access road (Section 6.8).

In Section B, where the stability is marginal and where placement of additional fills at the toe proposed cross-section is not recommended, rock fill material should not be end-dumped (e.g. to avoid overspill at the toe), but instead placed using a long reach excavator or bucket placed from the toe of slope.

Alternatively, in areas where the geometry of the existing slope is such that additional fills are only required within the upper 4 m of the slope (e.g. Sta. 26+875 and 26+900 EB in Section B), consideration could be given to excavating a temporary bench from surface to allow for conventional placement of the material. With a slight reduction in the widening (possibly by median narrowing), this could also be achieved along most of the southern slopes of Section B (i.e., from Station 26+825 to 26+925), which are the most critical slopes from a stability perspective.

An NSSP for construction of sliver fills has been prepared for inclusion in the contract documents and is attached in Appendix B.

6.8 Temporary Access Road Construction

A 5 to 7 m wide temporary access road is planned on the north and south sides of the embankment in order to access the toe of slope at the east pier of the CNR overpass structure. Based on the 60 percent drawings provided by MRC, the temporary access roads will cut down through the existing embankment starting at Station 27+050 in Section C and proceed down the slope to the west at a grade of about 4 to 5 percent to Station 26+950 (start of Section B). West of Station 26+950 (in Sections B and A), the temporary construction road is to be constructed approximately 1 m above the existing ground surface at the toe of slope. In this area, the toe of slope is underlain by surficial organic deposits about 4 m, and as much as 5.5 m thick. These surficial deposits are underlain by an extensive silty clay deposit up to 21 m thick. The upper portion of the silty clay consistency ranges from very soft to soft. The water table is at or very near ground surface.

It is understood that the road will likely need to be capable of supporting loads imposed by concrete trucks, pile driving/caisson equipment, or possibly large cranes needed to construct the superstructure, although the exact loading and configuration of the load is unknown at this time.

The sub-excavation of organics for the construction of a temporary access road is not a viable option for a number of reasons described in the previous sections, including impact to the environment. Other options such as timber crib support for the access road may not be economical and will further impact the wetlands. The removal of timber may not provide a satisfactory solution since it will be slow, tedious work and would cause much more damage to the existing natural ground.

Using lightweight materials such as styrofoam or slag could also be considered for this project. The styrofoam lightweight fill requires special construction techniques because the material cannot be placed directly on the ground covered with water in the wetland. The buoyancy effect has to be counterbalanced with an adequate amount of earth cover. This styrofoam material is also very expensive due to its high supply cost. Use of lightweight slag in place of Granular B or rock with geotextile/geogrid is a viable alternative but obtaining slag material from far away sources to the project site is very expensive. If slag is used, its environmental impacts in this wetland have to be evaluated.

The most feasible method of support of the temporary access road over the sensitive wetlands is by the use of reinforced geogrid/geotextile combination with Granular B, Type II material. Depending on the equipment loading on the road, one or two layers of uniaxial grid and one layer of stiff biaxial geogrid should be sufficient to support and reinforce the fill for the access roads. Each layer of grid requires a minimum of 0.5 m spacing between each lift to function properly. It should be noted that this option will not prevent settlement but is designed to distribute the applied loads and prevent shear failure of the soft organic and silty clay layers.

The installation should be as follows:

- Trees and shrubs or other large vegetation greater than 25 mm in diameter that may interfere with the placement of geosynthetics should be close cut and cleared.
- Placement of a layer of bi-axial geogrid to act as reinforcement and separation
- Placement of an initial layer of Granular B, Type II with a minimum thickness of 300 mm.
- Placement of at least one layer of uni-axial geogrid to act as reinforcement.
- Placement of additional layers of uni-axial geogrid, as necessary, and Granular B, Type II to achieve a final above grade thickness of 1.0 m.
- Placement of at least 150 mm of Granular A to provide a suitable driving surface.
- Placement of additional Granular A material, as required, to maintain trafficability and grades during construction, particularly in consideration of the presence of the underlying compressible peat deposits.

The anticipated settlement should be about half the fill thickness. The strength and the type of geogrid and geotextile should be designed by the manufacturer once the loading conditions and type of fill are known. The combination of the geotextile and geogrid should prevent any contamination of the underlying soils and facilitate the removal of the fill afterwards, if required. The environmental impact is also minimized.

It is understood that, to provide turtle habitat, the temporary access road will be left in place.

The design of the temporary access road is dependent on the nature of the heavy equipment that will use it. As such, the design of the temporary access road should be developed by the contractor/specialist supplier, in accordance with the general guidelines provided in the attached NSSP for the "Construction of Geogrid Reinforced Temporary Access Road" (Appendix B), and submitted to MTO for review and approval.

7.0 CLOSURE

This report was prepared by Ms. Erin S. O'Neill, P. Eng., under the direction of the Project Manager, Mr. Michael Snow, P. Eng. Mr. Fintan J. Heffernan, P. Eng., Golder's Designated MTO Contact for this project, conducted a technical and independent quality control review of the report.

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Table 1
Summary of Embankment Widening Sections
Highway 401 – Kingston, Cataraqui Embankments
G.W.P. 78-99-00

Section Designation	Approx. Highway 401 Station (STA)	Existing Embankment Height, Side Slopes	Proposed Width of Widening[3]	Relevant Boreholes	Simplified Stratigraphy at Toe of Slope
Section A	STA 26+720 to 26+825 WBL and EBL	Emb. Height: 8.5 – 11 m Side Slopes: 1.25-1.3H:1V	a) 0.5 to 4.0 m b) 0 to 6.0 m	W1, W2, W3 E2, E3, E4 B4, B9, B10	4 to 5 m of peat and organics (WB), 2 to 4 m of peat and organics (EB), over generally firm to very stiff or hard silty clay.
Section B	STA 26+825 to 26+925 WBL and EBL	Emb. Height: 6.5 – 8.5 m Side Slopes: 1.25-1.3H:1V	a) 3 to 4 m b) 0 to 1.5 m	W4, W5 E5, E6 S1	4 to 6 m of peat and organic clay or marl, and 2 to 3 m very soft to soft silty clay over stiff to very stiff silty clay.
Section C	STA 26+925 to 27+075 WBL STA 26+925 to 27+025 EBL	Emb. Height: 4.8 – 6.5 m Side Slopes: 1.25-1.3H:1V	a) 3 to 4 m b) 1.5 to 3 m	W6, W7, W8 E7, E8 S2	2 to 4 m of peat and organics overlying generally firm to hard silty clay.
Section D	STA 27+075 to 27+175 WBL STA 27+025 to 27+175 EBL	Emb. Height: 4 – 4.8 m Side Slopes: 1-1.25H:1V	a) 2.8 to 4 m b) 1.8 to 3 m	W9, W10 E9, E10, E11 S3	1 to 3 m of peat and 2 to 3 m of soft to firm silty clay over stiff silty clay.
Section E	STA 27+175 to 27+500 WBL and EBL	Emb. Height: 3 – 4 m Side Slopes: 1.25-4H:1V	a) 2 to 6 m b) 2 to 3.5 m	W11, W12, W13, W14,W15 E12, E13, E14, E15, E16, E17, E18, S3	0 to 2 m of silty clay fill, 1 to 3 m of peat or organics over generally stiff to hard silty clay.
Section F	STA 28+200 to 28+450 WBL and EBL	Emb. Height: 5.8 – 8.5 m Side Slopes: 1.25-4H:1V Widening: 1 to 4.5 m	a) 1 to 4.5 m b) 1 to 4.5 m	E19, E20, E21, E22, E23, E23B, E24, S4	0 to 1 m of fill, 0 to 4 m of peat and organics overlying generally stiff to hard silty clay or clay.

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[3] Width of widening is measured horizontally from the existing sideslopes based on a) original 4H:1V roundings geometry and b) revised 3H:1V roundings in Sections C to F and 2H:1V roundings in Sections A and B

Table 2
Summary of Foundation Engineering Parameters used in Analyses
Highway 401 – Kingston
G.W.P. 78-99-00

Embankment Section (Approximate Station)	Stratigraphic Unit		Top Elevation (m)	Thickness (m)	γ' (kN/m^3)	ϕ' (°)	c' (kPa)	S_u (kPa)	σ'_p (kPa)	e_0	C_c	C_r	c_v (cm^2/sec)
Highway 401 EBL/WBL STA 26+720 to 26+825 (Section A)	Below existing embankment	Embankment Fill	84.2	12.3	21	40							
		Silty Clay	71.9	7.3	18.8			42-100	168-400	0.94	0.34	0.05	6 x 10^{-3}
	At toe of slope	Peat	75	2.7 - 4.6	10.5	40	1		Replaced with embankment fill				
		Silty Clay	72.4	1.9	18.8			30	120	0.94	0.34	0.05	2 x 10^{-3}
		Silty Clay	70.5	3.5	18.8			40	160	0.94	0.34	0.05	2 x 10^{-3}
		Silty Clay	67	2.9	18.8			80	320	0.94	0.34	0.05	2 x 10^{-3}
Highway 401 EBL/WBL STA 26+825 to 26+925 (Section B)	Below existing embankment	Embankment Fill	83.2-82.2		21	40							
		Peat	68.0	1.0	11.2	40	1		NC	5.27	2.79	n/a	n/a
		Silty Clay	67	12.9	18.3			100	400	0.94	0.34	0.05	6 x 10^{-3}
	At toe of slope	Peat	75.5	4.0-5.5	10.7	48	1		Replaced with embankment fill				
		Organic clayey silt/marl	73.4-71.3	0.9-2.1	13	27							
		Silty Clay to clay	71.3-70.0	2.5 - 4	18.3			35-6	25	0.94	0.34	0.05	5 x 10^{-3}
		Silty Clay	68-67.3	1 - 2	18.3			6-100	25-400	0.94	0.34	0.05	5 x 10^{-3}
		Silty Clay to clay	67.6-66.5		18.3			80-100	400	0.94	0.34	0.05	5 x 10^{-3}

Embankment Section (Approximate Station)	Stratigraphic Unit		Top Elevation (m)	Thickness (m)	γ' (kN/m^3)	ϕ' (°)	c' (kPa)	S_u (kPa)	σ'_p (kPa)	e_0	C_c	C_r	c_v (cm^2/sec)
Highway 401 STA 26+925 to 27+075 (WBL) STA26+925 to 27+025 (EBL) (Section C)	Below existing embankment	Embankment Fill	81.5	8.0	21	40							
		Peat	73.5	0.6	11.2	40	1		NC	5.27	2.79	n/a	n/a
		Organic Silty Clay	72.9	0.7	16	27		72	288	3.0	2.75	0.275	n/a
		Silty Clay	72.2	6.3	18.9			80-95	320-380	0.94	0.34	0.05	5×10^{-3}
	At toe of slope	Peat	75.5	4.3	9.6/ 10.2	40	1		Replaced with embankment fill				
		Silty Clay	71.2	0.9	18.9			35		0.94	0.34	0.05	2×10^{-3}
		Silty Clay	70.3	4.4	18.9			70		0.94	0.34	0.05	2×10^{-3}
Highway 401 STA 27+075 to 27+175 (WBL) STA 27+025 to 27+175 (EBL) (Section D)	Below existing embankment	Embankment Fill	80.0	6.5	21	40							
		Peat	73.5	1.0	11.2	40	1		NC	5.27	2.79	n/a	n/a
		Silty Clay	72.5	1.5	18.4				450	0.94	0.34	0.05	4×10^{-3}
		Silty Clay	71.0	6.4	18.4			95	380	0.94	0.34	0.05	4×10^{-3}
	At toe of slope	Peat	75.0	2.5	9.6	40	1		Replaced with embankment fill				
		Silty Clay, slightly organic	72.5	1.5	16.4	40							
		Silty Clay	71.0	0.4	18.4			6-10	25	0.94	0.34	0.05	4×10^{-3}
		Silty Clay	69.6	2.0	18.4			10-95	25-380	0.94	0.34	0.05	4×10^{-3}
		Silty Clay	67.6	12	18.4			95	380	0.94	0.34	0.05	4×10^{-3}

Embankment Section (Approximate Station)	Stratigraphic Unit		Top Elevation (m)	Thickness (m)	γ' (kN/m^3)	ϕ' (°)	c' (kPa)	S_u (kPa)	σ'_p (kPa)	e_0	C_c	C_r	c_v (cm^2/sec)
Highway 401 EBL/WBL STA 27+175 to 27+500 (Section E)	Below existing embankment	Embankment Fill	79.5	7.4	21	40							
		Peat	73.5	1.0	11.2				NC	5.27	2.79	n/a	n/a
		Silty Clay	72.5	1.5	19.3			95	450	0.94	0.34	0.05	4×10^{-3}
		Silty Clay	71.0	6.4	19.3			95	380	0.94	0.34	0.05	4×10^{-3}
	At toe of slope	Peat	75.5	2.4	9.6	40	1		Replaced with embankment fill				
		Organic Silty Clay	73.1	1.0	12	27		54	216	0.94	0.34	0.05	2×10^{-3}
		Clayey Silt	72.1	7.5	19.3			100	400	0.94	0.34	0.05	2×10^{-3}
Highway 401 EBL/WBL STA 28+200 to 28+450 (Section F)	Below existing embankment	Embankment Fill	81.4	6.9	21	40							
		Peat	74.5	0.3	10.2	40	1		NC	5.27	2.79	n/a	n/a
		Organic Clay	74.2	0.5	16.4	27		95	NC	3.0	2.75	n/a	n/a
		Silty Clay	73.7	5.7	18.6			95	380	0.94	0.34	0.05	5×10^{-3}
	At toe of slope	Peat	75.1	1.5	10.2	40	1		Replaced with embankment fill				
		Organic Clay	73.6	2.8	16.4	27							
		Silty Clay	70.9	0.6	18.6			15-100	60 - 400	0.94	0.34	0.05	4×10^{-3}
		Silty Clay	70.3	2.6	18.6			100	400	0.94	0.34	0.05	4×10^{-3}

Table 3
Stability/Settlement Mitigation Options
Highway 401 – Kingston Cataraqui Embankments
G.W.P. 78-99-00

Stability / Settlement Mitigation Option	Applicability / Feasibility	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Full Sub-excavation of Compressible Organic Soils	Applicable where compressible organic deposits are located at the toe of the slope. Feasible where deposits do not extend beyond about 9 m below ground surface.	▪ Reduced total settlements of the embankment side slopes and toe. ▪ Reduced differential settlements between the crest of the widening and the toe of slope, which could result in distortion and potential progressive shallow failures of the embankment side slopes. ▪ Reduced potential for settlements of travelled portions of the highway with future widenings, by eliminating "trapped" organics below the embankment. ▪ Improved long-term embankment stability.	▪ Generation of large volumes of excavation spoil requiring disposal/management. ▪ Some increase in settlement of existing embankment due to additional loads at toe resulting from replacement of light organic soils with heavier embankment fills. ▪ The need for a larger corridor of land acquisition. ▪ Greater quantities of rock fill required.	Additional costs associated with sub-excavation, disposal and replacement of compressible organic deposits. Additional temporary shoring and protection costs where depths exceed X m.	▪ Risk of instability of existing embankment slopes without appropriate temporary protection measures in place, particularly where deep cuts are required adjacent to high embankments. ▪ Staged excavation in strips of limited width may be required.
Partial Sub-excavation of Soft Cohesive Soils	Applicable where soft soils at the toe of the slope result in unstable slopes or excessive differential settlement between the crest and toe. Feasible where deposits do not extend beyond about 9 m below ground surface.	▪ Improved long-term stability of widened embankment. ▪ Reduced requirement for stabilizing toe berms, minimizing need for additional right-of-way. ▪ Reduced total settlements of the embankment side slopes and toe. ▪ Reduced potential for settlements of travelled portions of the highway with future widenings, by eliminating "trapped" organics below the embankment.	▪ Increased delay in construction associated with excavating the soft cohesive soils. ▪ Increased quantity of rock fill required to replace subexcavated cohesive soils. ▪ Increased generation of excess excavation spoil.	Costs associated with additional sub-excavation, disposal and replacement of soft cohesive soils. Additional temporary shoring and protection costs where depths exceed 4 m.	▪ Risk of instability of existing embankment slopes without appropriate temporary protection measures in place, particularly where deep cuts are required adjacent to high embankments. ▪ Potential requirement for temporary lane closures or speed reductions.

Stability / Settlement Mitigation Option	Applicability / Feasibility	Advantages	Disadvantages	Relative Costs	Risks / Consequences
No removal of peat (or partial removal) at toe of slope	Applicable where geometric modifications to slope mean no new fills being placed on virgin peat deposits. Feasible only there is time to construct embankment in slow stages.	▪ Reduced construction costs associated with effort to subexcavate peat. ▪ Slight reduction of rock fill quantities below grade if filling is controlled and no significant failures occur during construction. ▪ In areas where only partial removal is carried out, no need for temporary shoring during subexcavation.	▪ Non-standard construction. ▪ Embankment must be constructed in many stages over time to avoid risk of instability of the widened embankment during construction. ▪ Compressible peat deposits left in place beneath embankment which will continue to settle over time. ▪ Compressible organics will become trapped under the widened embankment sideslopes and will be difficult to remove in the future.	Increased costs associated with lengthened project schedule and additional monitoring during construction. Cost reductions trucking and disposal of peat, and potentially in total rock fill placed. Increased long-term costs for maintenance of side slopes.	▪ Risk of instability of widened embankment during construction, unless constructed in many stages. ▪ Delays to construction depending on ground response. ▪ Ongoing long term settlement of organic deposits and potential for future performance issues.
Stabilizing Toe Berms	Applicable where soft soils at the toe of the slope cannot be removed and thus result in unstable slopes. Feasible where sufficient right-of-way is available to permit placement of toe berms.	▪ Improved long-term stability of widened embankment. ▪ Improved temporary stability of existing embankment (excavation depths less than with excavation and removal of soft cohesive soils at depth. ▪ Reduced post-construction settlements of travelled portions of highway for future widening (from preloading effect).	▪ Additional right-of-way may be required to allow for placement of toe berm. ▪ Increased generation of excess excavation spoil in toe berm is placed as an extended zone of subexcavation and replacement below existing grade. ▪ Increased quantity of rock fill required for toe berms. ▪ Small increase in settlements at the toe and side slopes of embankment due to added load of berm. ▪ Construction is delayed to allow for primary consolidation to be completed and possibly for staged construction (if required).	Additional costs associated with sub-excavation, disposal and replacement with rock fill of additional compressible organic deposits where toe berm is constructed as an extended zone of full sub-excavation.	▪ Toe berm may act as an additional driving load and further destabilize slope.

Stability / Settlement Mitigation Option	Applicability / Feasibility	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Preloading	Applicable where predicted settlement at crest of slope exceeds tolerances. Feasible where rate of consolidation is fast enough to allow for further consolidation within a reasonable time period.	▪ Reduced magnitude of long-term, post-construction settlements by promoting such settlements to occur under embankment fill loads in advance of final grading of the embankment. ▪ Improved strength of underlying cohesive soils (at toe), resulting in improved short and long-term stability of widened embankment. ▪ Provides a means of identifying and mitigating the additional settlements resulting from the potential presence of a "trapped" wedge of organics beneath the embankment.	▪ Construction is delayed to allow for primary consolidation to be completed and possibly for staged construction (if required). ▪ An instrumentation and monitoring program would be required to assess when end of primary consolidation is reached. ▪ Regrading will be required to account for settlement prior to construction of the final pavement structure.	Additional costs associated with delays to construction and remobilization to regrade before final pavement structure. Additional costs associated with instrumentation and monitoring.	▪ Preload time could be reduced by instrumenting embankment and monitoring actual rate of settlement.
Surcharging	Applicable where predicted settlement at crest of slope exceeds tolerances and where rate of consolidation too slow to achieve the desired reductions within an acceptable timeframe. Feasible where there is sufficient room to permit placement of surcharge (at crest) and where embankment is short enough that additional load is felt by compressible deposits at depth.	▪ Reduced magnitude of long-term, post-construction settlements by promoting such settlements to occur under embankment fill loads in advance of final grading of the embankment and at a faster rate than with preloading alone. ▪ Decreased delay time for construction over preloading alone. ▪ Reduced width of stabilizing toe berm (if required) if toe berm is surcharged because of increased strength of surcharged cohesive soils, and associated reductions in excavation spoil, quantity of rock fill, and time to construct toe berm.	▪ Construction is delayed, albeit less than for preloading alone, to allow for primary consolidation to occur. ▪ Longer construction time if staged construction is required. ▪ Larger quantity of rock fill if toe berms are required for stability, as compared with preloading alone. ▪ An instrumentation and monitoring program would be required to assess when end of primary consolidation is reached. ▪ Increased handling of rock fill (or Granular 'B') to remove the surcharge.	Additional costs associated with additional fill placement and handling. Additional costs associated with instrumentation and monitoring.	▪ Reduced risk of slope instability if soft cohesive soils near toe of slope are surcharged and strengths increased.

Stability / Settlement Mitigation Option	Applicability / Feasibility	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Lightweight Fill	Applicable where reduction in loading would improve slope stability or mitigate settlements in excess of allowable limits. Feasible only for placement in limited areas above the water table and below frost levels for EPS-type fills (e.g., near existing culvers).	▪ Improved long-term stability of widened embankment. ▪ Reduced settlements at crest and toe of slope. ▪ No significant delay in construction; and elimination of the need for stabilizing toe berms.	▪ Significant cost for procurement. ▪ EPS type fills can only be placed above the water table and below frost levels. ▪ Slag type fills may not be suitable for placement in environmentally sensitive watersheds.	Significant additional expense of lightweight fill (depending on the volume required).	
Ground Improvement	Applicable where more traditional means of slope stabilization are not feasible. Feasibility of individual methods of ground improvement depend on the soils to be improved, the slope geometry, and the required strength gain.	▪ Improved long-term stability of existing and widened embankment. ▪ Improved temporary stability of existing embankment by reducing (or eliminating) the depth of subexcavation and replacement at the toe. ▪ Reduced magnitude of long-term, post-construction settlements by shedding loads to stiffer members (where column-supported ground improvement is used). ▪ Minimal impact on construction schedule, as work could be completed in advance of the larger scale embankment widening operations.	▪ Additional investigation, testing and design will be required to develop the most cost-effective design. ▪ A specialty contractor will need to be retained to carry out ground improvement works. ▪ A separate instrumentation and monitoring, and testing program would be required to verify the results of the ground improvement.	▪ Depending on the selected method of ground improvement, this option may be more costly than temporary shoring for partial subexcavation of deep cohesive soils.	▪ Minimal risk to existing embankment during improvement and construction, compared to deep excavations adjacent to high embankment slopes (e.g., partial subexcavation).

Stability / Settlement Mitigation Option	Applicability / Feasibility	Advantages	Disadvantages	Relative Costs	Risks / Consequences
Modified Geometry	Applicable where avoiding placement of fills at toe can eliminate or reduce stability or settlement concerns. Feasible only if modifications to the road platform design can be accommodated.	▪ Reduced loading on side slopes and toe of slope results in reduced settlement and brings the factor of safety of widened embankments closer to existing conditions. ▪ Where slope geometry is modified such that no (or limited) additional fill is placed at the toe of slope, subexcavation of organics at toe can be eliminated.	▪ Increased potential for instability, erosion, or reduction in shoulder width of near-surface fills where roundings are steepened. ▪ Creation of sliver widening which are more challenging to construct. ▪ Additional settlement and potential for oversteepening of toe of slope where organics are left in place.	▪ Potential for increased long-term maintenance costs associated with shoulder maintenance.	▪ Minimizes risk to existing embankment during widening by reducing new fills on sideslopes and eliminating deep excavations adjacent to high embankment slopes.

Table 4
Summary of Preferred Foundation Mitigation Options
Highway 401 Kingston – Cataraqui Embankments
G.W.P 78-99-00

Highway Section and Approximate Station	Existing Embankment Height and Side Slopes	Proposed Embankment Fill, Side Slope and Width of Widening	Added fill height A) at edge of FPS B) mid-slope C) peat removal at toe	Assumed Stability / Settlement Mitigation Option for Settlement Calculations	Estimated Settlement (δ) During Construction	Estimated- Post-Construction Settlement (δ) with Full Subexcavation of Organics at Toe	Selected Settlement / Stability Mitigation Option (as per Section 6.5)
Section A 26+720 to 26+825	8.5 – 11 m 1.25H:1V	Rock Fill Proposed 1.25H:1V Widening: 1 to 5 m	A) 0.8 to 1.6 m B) 2.5 to 3.8 m C) 1.8 to 4.6 m	Full subexcavation of organics at toe (OPSD 203.020) + 2-4 month preload where trapped organics are identified	At Edge of FPS $\delta_{CLAY\ (PR)}$ = 35 mm $\delta_{CLAY\ (REC)}$ = 10 mm $\delta_{FILL\ (IMM)}$ = 10 mm At toe of slope δ_{TOE} = 180 mm	At Edge of FPS $\delta_{CLAY\ (SEC)}$ = 25 mm At toe of slope δ_{TOE} = 80 mm	Modified Slope Geometry Sliver fills with geogrid-reinforced temporary access road at toe of slope. No organics removals 6 month preload
Section B 26+825 to 26+925	6.5 – 8.5 m 1.25-1.5H:1V	Rock Fill Proposed 1.25H:1V Widening: 3 to 4 m	A) 0.6 to 1.2 m B) 3.2 C) 4.3 to 5.5 m	Ground improvement of weak clays and full subexcavation or improvement of the organics at toe (OPSD 203.020) + 2-4 month preload where trapped organics are identified	At Edge of FPS $\delta_{ORG\ (PR)}$ = 25 mm $\delta_{CLAY\ (REC)}$ = 30 mm $\delta_{FILL\ (IMM)}$ = 10 mm At toe of slope δ_{TOE} = 420 mm	At Edge of FPS $\delta_{ORG\ (SEC)}$ = 50 mm $\delta_{CLAY\ (SEC)}$ = 40 mm At toe of slope δ_{TOE} = 130 mm	Modified Slope Geometry Sliver fills with geogrid-reinforced temporary access road at toe of slope. No organics removals 6 month preload
Section C 26+925 to 27+075 (WBL) 26+925 to 27+025 (EBL)	4.8 – 6.5 m 1.25-2H:1V	Rock Fill Proposed 1.25H:1V Widening: 3 to 5 m	A) 0.6 to 1.2 m B) 2.6 to 3.2 m C) 1.8 to 4.1 m	Full subexcavation of organics at toe (OPSD 203.020) + 2-4 month preload where trapped organics are identified	At Edge of FPS $\delta_{ORG\ (PR)}$ = 40 mm $\delta_{ORG\ (SEC)}$ = 10 mm $\delta_{CLAY(REC)}$ = 35 mm $\delta_{FILL\ (IMM)}$ = 10 mm At toe of slope δ_{TOE} = 200 mm	At Edge of FPS $\delta_{ORG\ (SEC)}$ = 75 mm $\delta_{CLAY\ (SEC)}$ = 35 mm At toe of slope δ_{TOE} = 65 mm	Modified OPSD 203.020 Staged Excavation in strips of limited length, width and depth 6 month preload
Section D 27+075 to 27+175 (WBL) 27+025 to 27+175 (EBL)	4 – 4.8 m 1.25-1.75H:1V	Rock Fill Proposed 2H:1V Proposed 1.25H:1V Widening: 3 to 5 m	A) 0.3 to 0.5 m B) 2.0 to 2.6 m C) 2.3 to 4.0 m	Full subexcavation of organics at toe (OPSD 203.020) + 2-4 month preload where trapped organics are identified	At Edge of FPS $\delta_{ORG\ (PR)}$ = 60 mm $\delta_{ORG\ (SEC)}$ = 10 mm $\delta_{CLAY(REC)}$ = 30 mm $\delta_{FILL\ (IMM)}$ = 10 mm At toe of slope δ_{TOE} = 275 mm	At Edge of FPS $\delta_{ORG\ (SEC)}$ = 50 mm $\delta_{CLAY\ (SEC)}$ = 30 mm At toe of slope δ_{TOE} = 85 mm	OPSD 203.020 Staged Excavation in strips of limited length, width and depth 6 month preload

Highway Section and Approximate Station	Existing Embankment Height and Side Slopes	Proposed Embankment Fill, Side Slope and Width of Widening	Added fill height A) at edge of FPS B) mid-slope C) peat removal at toe	Assumed Stability / Settlement Mitigation Option for Settlement Calculations	Estimated Settlement (δ) During Construction	Estimated- Post-Construction Settlement (δ) with Full Subexcavation of Organics at Toe	Selected Settlement / Stability Mitigation Option (as per Section 6.5)
Section E 27+175 to 27+500	3 – 4 m 1.25-4H:1V	Rock Fill Proposed 4H:1V Proposed 2H:1V Proposed 1.25H:1V Widening: 3 to 4 m	A) 0 to 0.5 m B) 0.2 to 2.2 m C) 0 to 2.4 m	Full subexcavation of organics at toe (OPSD 203.020) + 2-4 month preload where trapped organics are identified	At Edge of FPS $\delta_{ORG\ (PR)}$ = 40 mm $\delta_{ORG\ (SEC)}$ = 5 mm $\delta_{CLAY (REC)}$ = 15 mm $\delta_{FILL\ (IMM)}$ = 10 mm At toe of slope δ_{TOE} = 80 mm	At Edge of FPS $\delta_{ORG\ (SEC)}$ = 50 mm $\delta_{CLAY\ (SEC)}$ = 15 mm At toe of slope δ_{TOE} = 30 mm	OPSD 203.020 Staged Excavation in strips of limited length, width and depth 6 month preload
Section F 28+200 to 28+450	5.8 – 8.5 m 1.25-4H:1V	Rock Fill Proposed 4H:1V Proposed 1.25H:1V Widening: 1 to 4.5 m	A) 0 to 0.9 m B) 1.0 to 3.5 m C) 0 to 4.3 m	Full subexcavation of organics at toe (OPSD 203.020) + 2-4 month preload where trapped organics are identified	At Edge of FPS $\delta_{ORG\ (PR)}$ = 40 mm $\delta_{ORG\ (SEC)}$ = 5 mm $\delta_{CLAY\ (REC)}$ = 15 mm $\delta_{FILL\ (IMM)}$ = 10 mm At toe of slope δ_{TOE} = 85 mm	At Edge of FPS $\delta_{ORG\ (SEC)}$ = 35 mm $\delta_{CLAY\ (SEC)}$ = 10 mm At toe of slope δ_{TOE} = 30 mm	OPSD 203.020 Staged Excavation in strips of limited length, width and depth. 6 month preload

Table 5a
Summary of Subexcavation Requirements
WESTBOUND LANES – NORTH SIDE OF EMBANKMENT
Highway 401 Kingston Cataraqui Embankments
G.W.P. 78-99-00

Embankment Section (Approximate Station)	Borehole	Approximate Hwy 401 Station	Approximate Offset of Borehole from Hwy 401 Centreline	Ground Surface Elevation (m)	Depth of Subexcavation (m)	Base Elevation of Subexcavation (m)
Highway 401 WBL STA 26+720 to 26+825 (Section A)	W1	26+725	36.0	75.8	*	*
	W2	26+750	34.9	75.8	*	*
	W3	26+800	34.1	76.0	*	*
Highway 401 WBL STA 26+825 to 26+925 (Section B)	W4	26+850	31.0	76.5	*	*
	W5	26+900	31.1	75.6	*	*
Highway 401 WBL STA 26+925 to 27+075 (Section C)	W6	26+950	29.8	75.5	3.0	72.5
	W7	27+000	28.3	75.4	3.0	72.4
	W8	27+050	26.3	75.5	1.8	73.7
Highway 401 WBL STA 27+075 to 27+175 (Section D)	W9	27+100	26.8	75.4	2.3	73.1
	W10	27+150	24.5	75.6	2.4	73.2
Highway 401 WBL STA 27+175 to 27+500 (Section E)	W11	27+200	25.1	75.6	1.7	73.9
	W12	27+250	24.5	75.5	2.2	73.3
	W13	27+300	23.2	76.6	0.0	n/a
	W14	27+350	22.2	77.3	0.0	n/a
	W15	27+400	21.9	77.2	0.0	n/a

Notes:

[1] Ice/Water was present above the top of the peat at the time of drilling.

[2] Includes thickness of fill encountered above the peat/organic deposits.

* No subexcavation or removals of peat, constructed as sliver widening with temporary access road at toe of existing slope.

Table 5b
Summary of Subexcavation Requirements
EASTBOUND LANES – SOUTH SIDE OF EMBANKMENT
Highway 401 Kingston Cataraqui Embankments
G.W.P. 78-99-00

Embankment Section (Approximate Station)	Borehole	Approximate Hwy 401 Station	Approximate Offset of Borehole from Hwy 401 Centreline	Ground Surface Elevation (m)	Depth of Subexcavation (m)	Base Elevation of Subexcavation (m)
Highway 401 EBL STA 26+720 to 26+825 (Section A)	E2	26+720	38.7	75.5	*	*
	E3	26+750	37.9	75.1	*	*
	E4	26+800	36.9	75.3	*	*
Highway 401 EBL/STA 26+825 to 26+925 (Section B)	E5	26+850	32.6	76.0	*	*
	E6	26+900	33.0	75.5	*	*
Highway 401 EBL/WBL STA 26+925 to 27+025 (Section C)	E7	26+950	29.5	75.7	3.0[2]	72.7
	E8	27+000	28.5	75.1	2.1	73.0
Highway 401 EBL/WBL STA 27+025 to 27+175 (Section D)	E9	27+050	33.3	75.3	2.5	72.8
	E10	27+100	29.1	75.5	3.0	72.5
	E11	27+150	26.3	75.1	3.0	72.1
Highway 401 EBL/WBL STA 27+175 to 27+500 (Section E)	E12	27+200	26.3	75.4	2.4	73.0
	E13	27+250	25.5	75.2	1.8	73.4
	E14	27+300	28.0	75.1	2.0	73.1
	E15	27+350	29.0	75.0	1.7[1]	73.3
	E16	27+400	27.9	75.0	1.8	73.2
	E17	27+450	25.0	75.0	1.2[1]	73.8
	E18	27+500	36.0	75.4	0	n/a
Highway 401 EBL STA 28+200 to 28+430 (Section F)	E19	28+200	39.1	75.8	0.0	n/a
	E20	28+250	29.8	75.1	3.0	72.1
	E21	28+300	32.3	75.0	1.8[1]	73.2
	E22	28+350	31.3	77.1	0	n/a
	E23	28+400	36.1	76.6	0	n/a
	E24	28+440	38.3	75.1	0	n/a

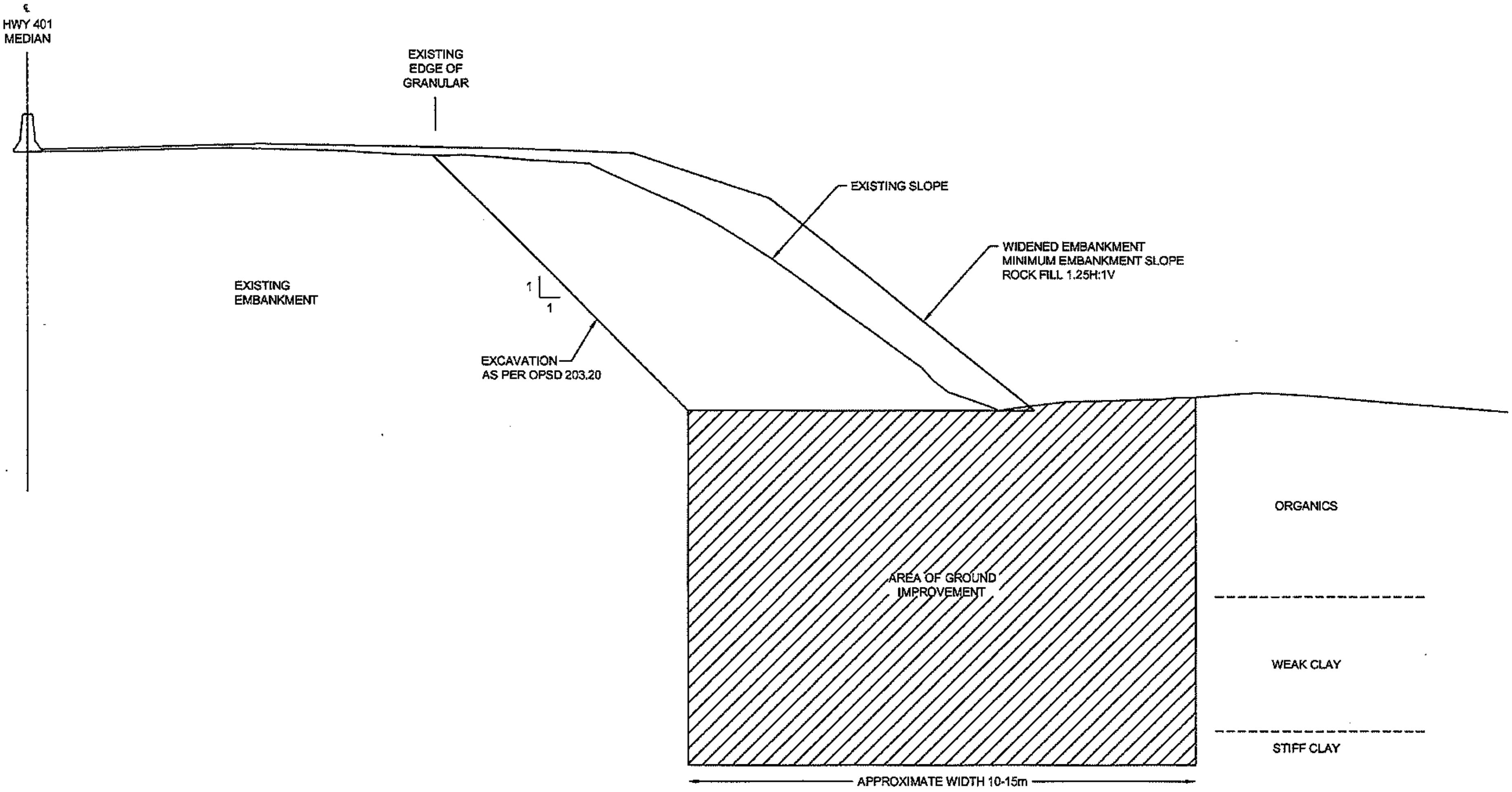
Notes:

[1] Ice/Water was present above the top of the peat at the time of drilling.

[2] Includes thickness of fill encountered above the peat/organic deposits.

* No subexcavation or removals of peat. Constructed as sliver widening, temporary access road constructed at toe of existing slope.

PLOT DATE: October 7, 2010
FILENAME: N:\Active\2008\1111\08-1111-0044 MRC Hwy 401\ACAD\Phase 2500\0811110044-2500-04.dwg



NOT TO SCALE

HIGHWAY 401 EMBANKMENT WIDENING

SCHEMATIC GROUND IMPROVEMENT SECTION

Golder Associates
Ottawa, Ontario, Canada

PROJECT No. 08-1111-0044 DATE 6 OCT. 10 CAD J.M.

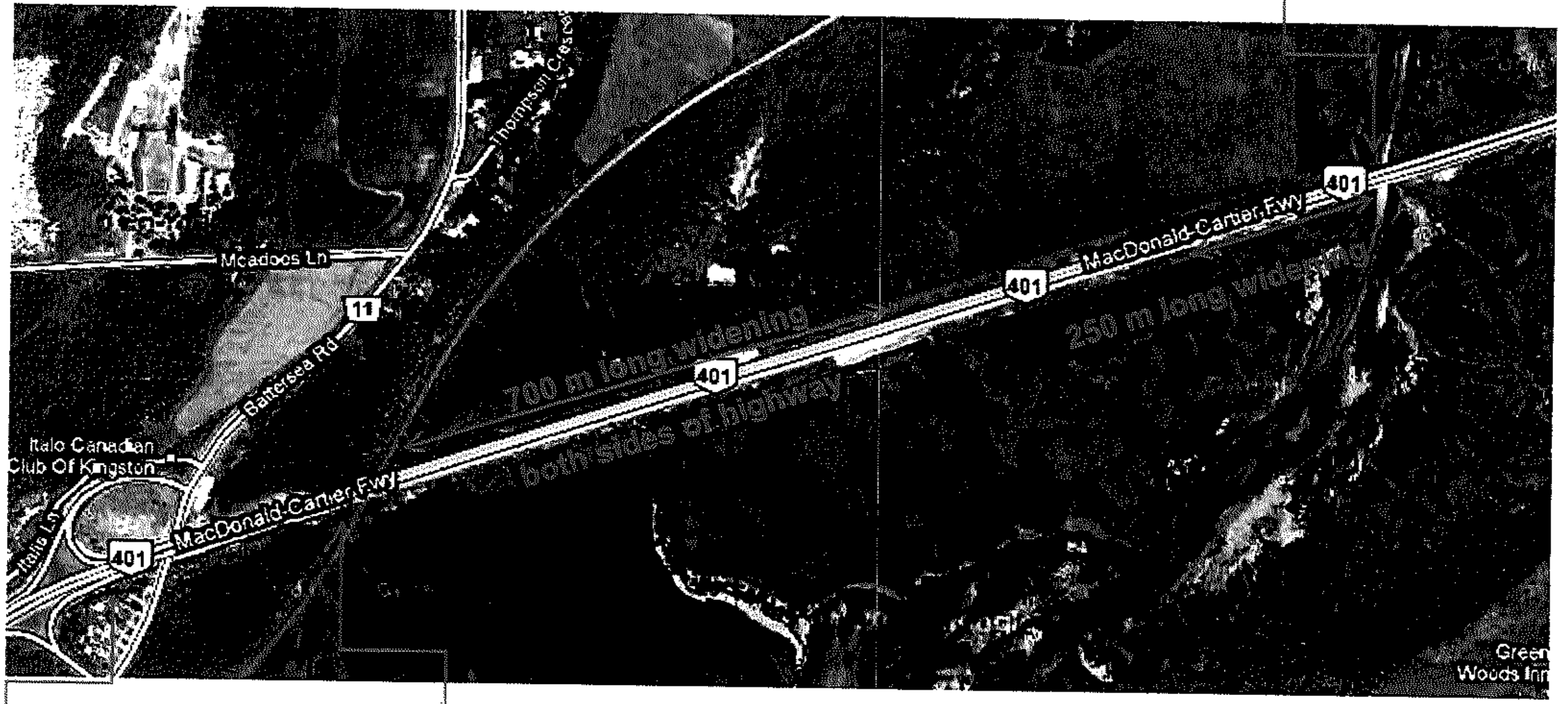
DRAWING 4

Foundations Investigation and Design

Highway 401 Embankment Widening Cataraqui Wetlands, Kingston, Ontario

Golder Associates

Site Location



Cataraqui River

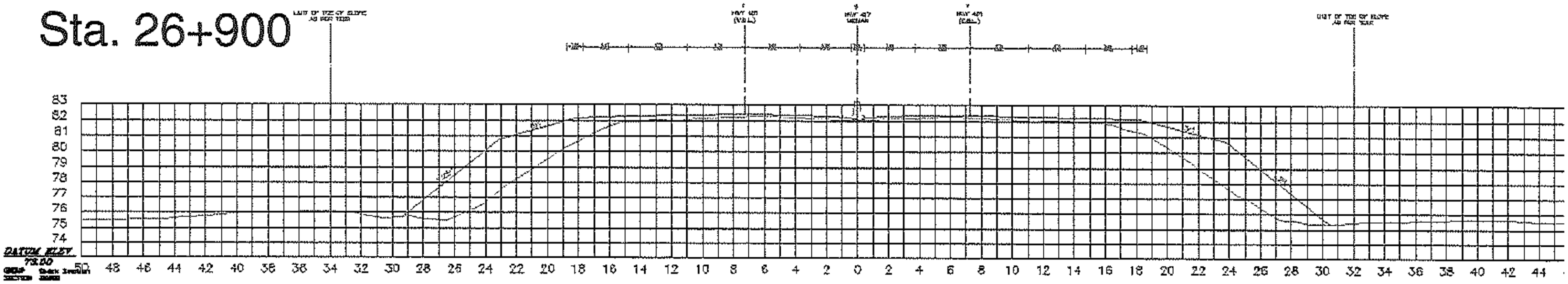
N ↑

Montreal Street Interchange

CNR Overpass

Proposed Embankment Widening

Sta. 26+900



Existing Embankment:

- rock fill, side slopes: typically 1.25H:1V to 1.5H:1V
- height: 11 m at CNR , < 3 m in middle, 8 m near Cataraqui River

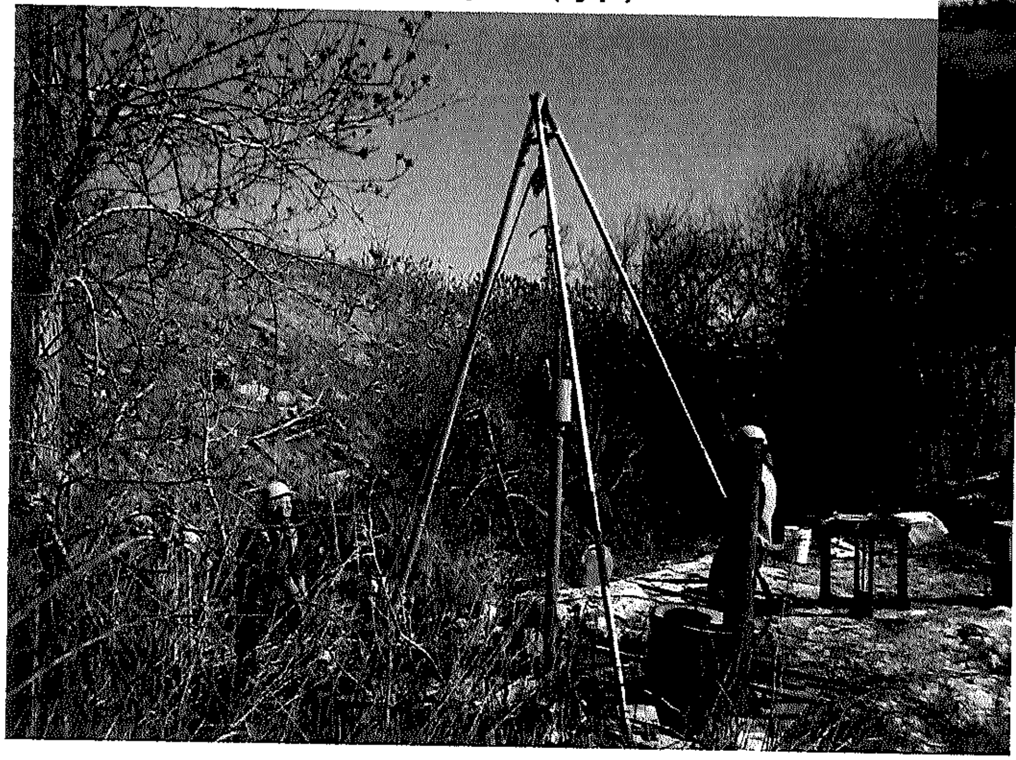
New Embankment:

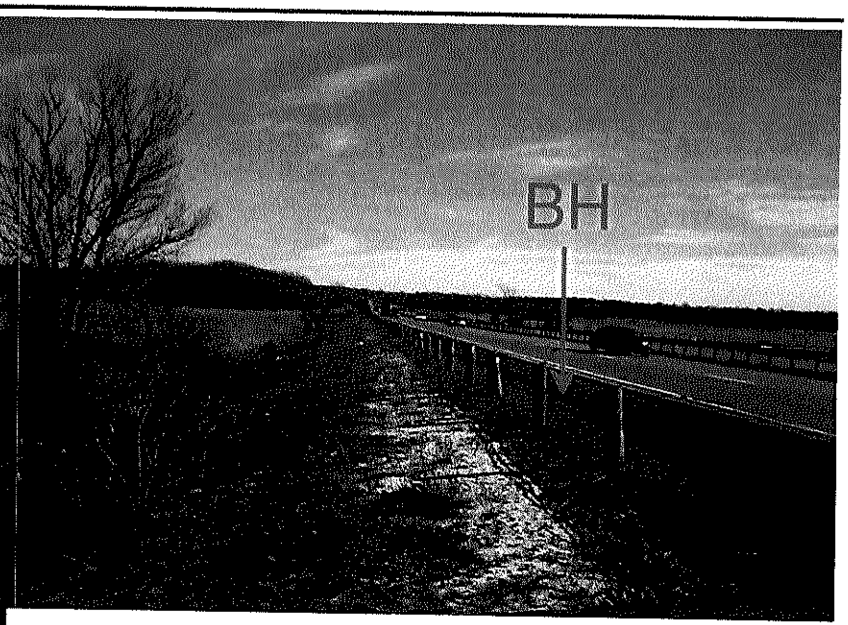
- widening Hwy 401 from 4 to 6 lanes (~ 5 m laterally)
- preferred embankment fill: Rock Fill (availability)
- side slopes 1.25H:1V, with 3H:1V rounding

Field Investigation

Toe of Slope Boreholes

- put down 4 m from toe of existing embankment at ~50 m spacing
- 38 BHs, 6-10 m depth (typ) or refusal

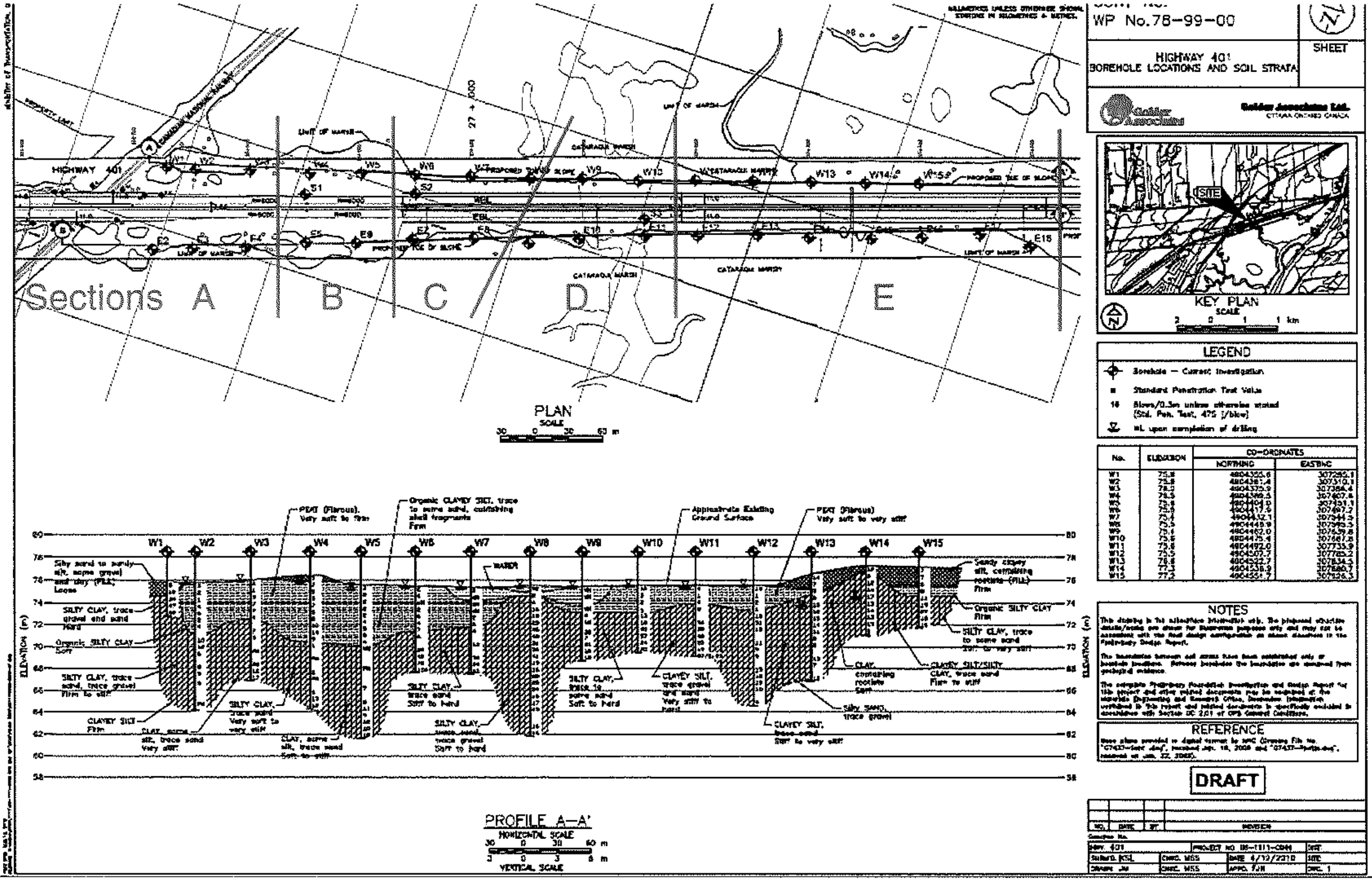




Crest of Slope Boreholes

- boreholes put down at shoulder
- 4 BHs, 9-15 m depth
- fill beneath original ground?
- organics beneath embankment?

Borehole Locations and Stratigraphic Section



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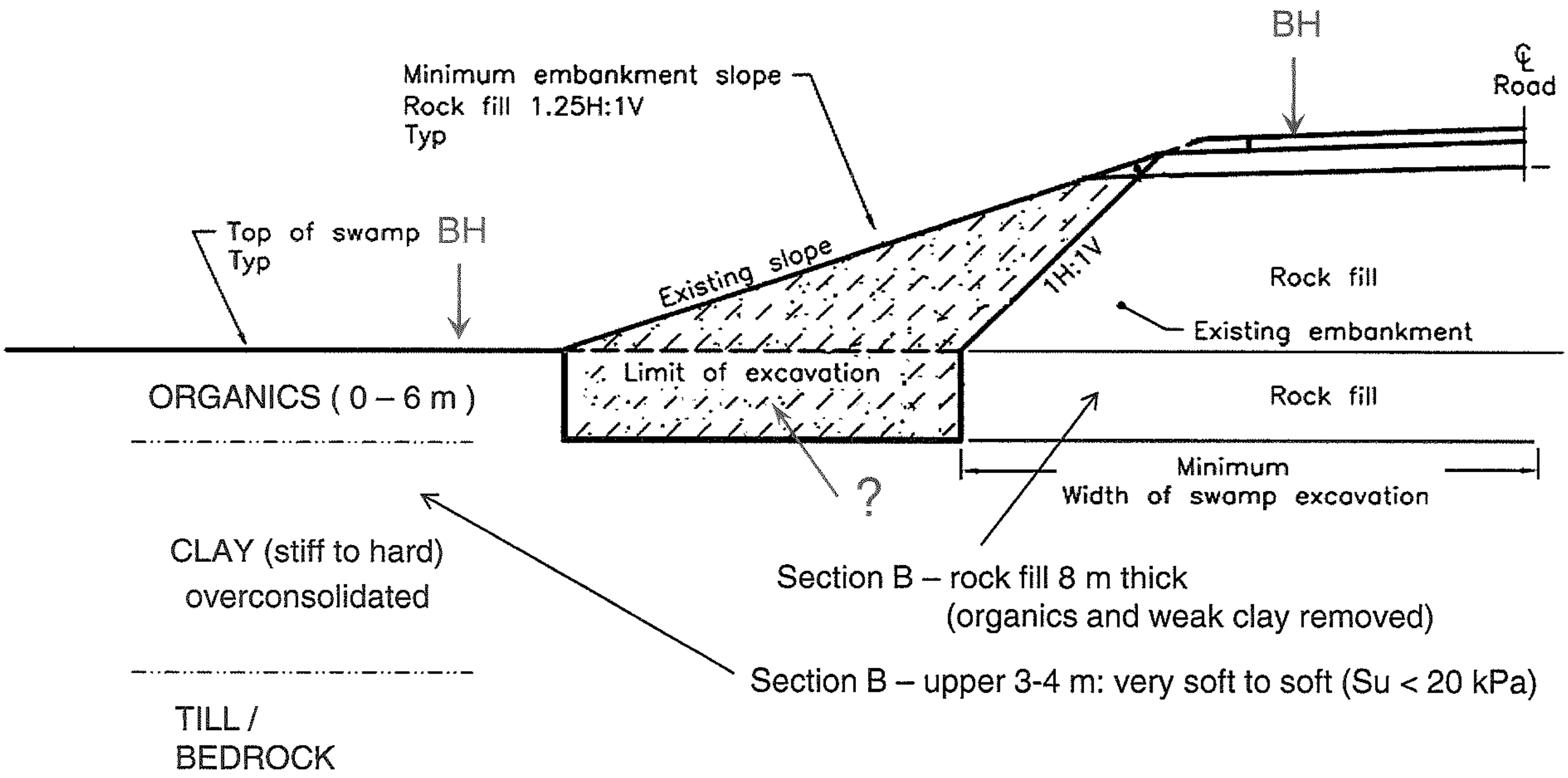
Existing Embankment Construction



ORGANICS (0 – 6 m)

CLAY (stiff to hard) overconsolidated

TILL / BEDROCK

Section B – rock fill 8 m thick (organics and weak clay removed)

Section B – upper 3-4 m: very soft to soft (Su < 20 kPa)

Subsurface Conditions

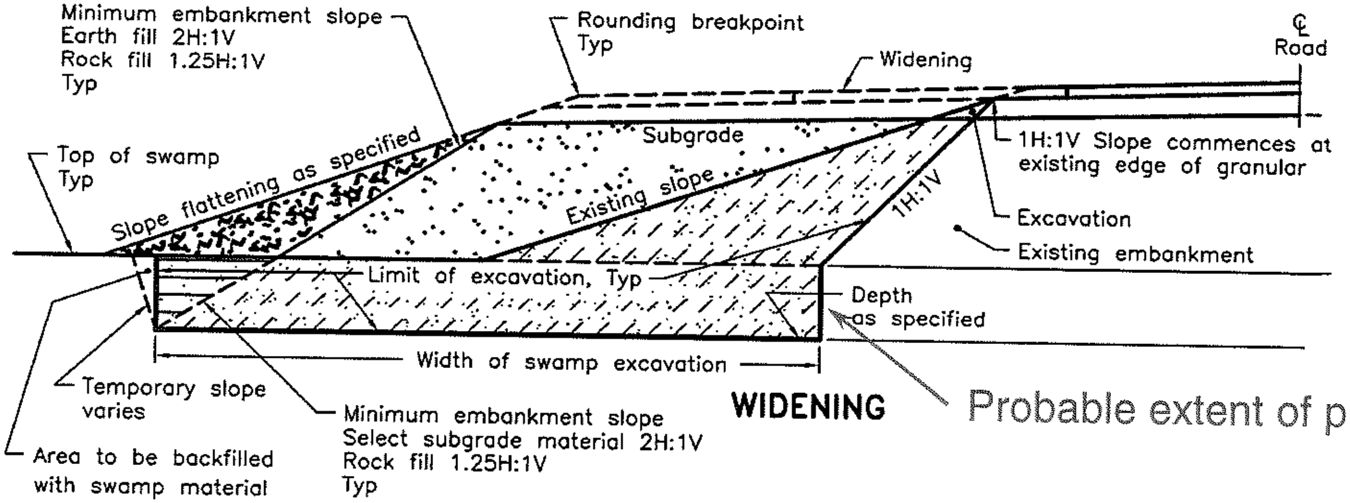
Toe of slope (Native ground conditions):

- peat and organic deposits
    - thickness varies across site (0 - 6 m) but thickest in Section B
    - fibrous, very soft to stiff, water contents 220 to 685 % (typ.)
- clay
    - firm to very stiff or hard (overconsolidated) along most of alignment
    - in Section B, upper 3 to 4 m: very soft to soft (Su < 20 kPa)
- till, bedrock, auger refusal
    - thin veneer (0.2 m) of till over limestone or sandstone bedrock
    - inferred bedrock depth ~ 14-22 m near bridge, shallower to the east

Existing Embankment:

- primarily fine rock fill, contains obstructions (cobbles and boulders);
    - fill up to 2 below o.g. in Sections C,D,F (subexcavation of peat)
    - fill 8 m below o.g. in Section B (subexcavation of peat and weak clay)
- native soils
    - up to 1 m of organics over very stiff clay

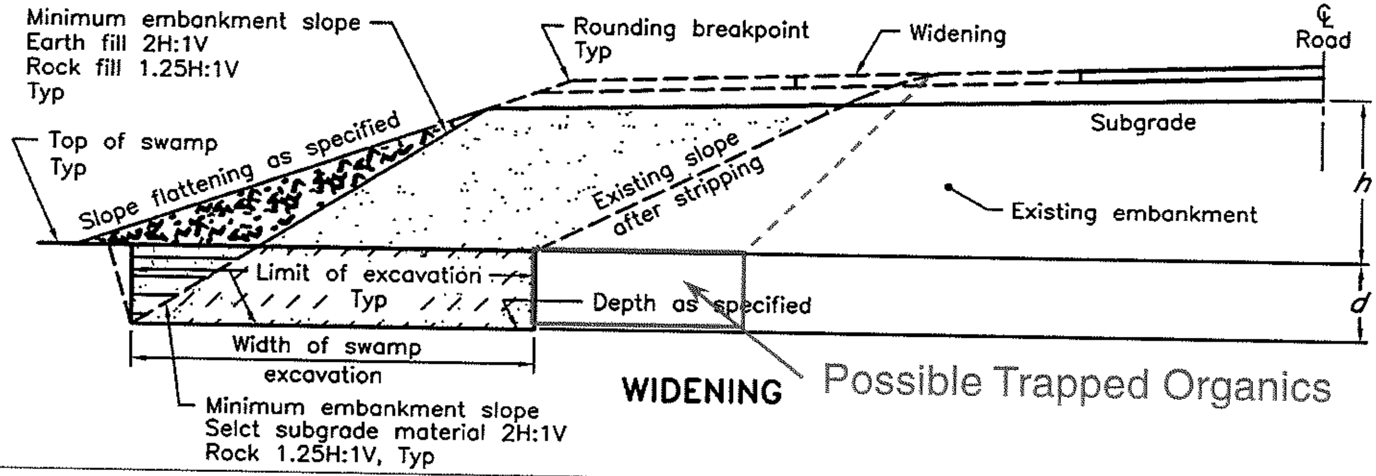
Typical MTO Practice
Widening Embankments over Swamps



OPSD 203.020

Slopes cut to 1H:1V

Probable extent of previous removals



OPSD 203.030

Existing slopes maintained

Possible Trapped Organics

Settlement Analysis Results

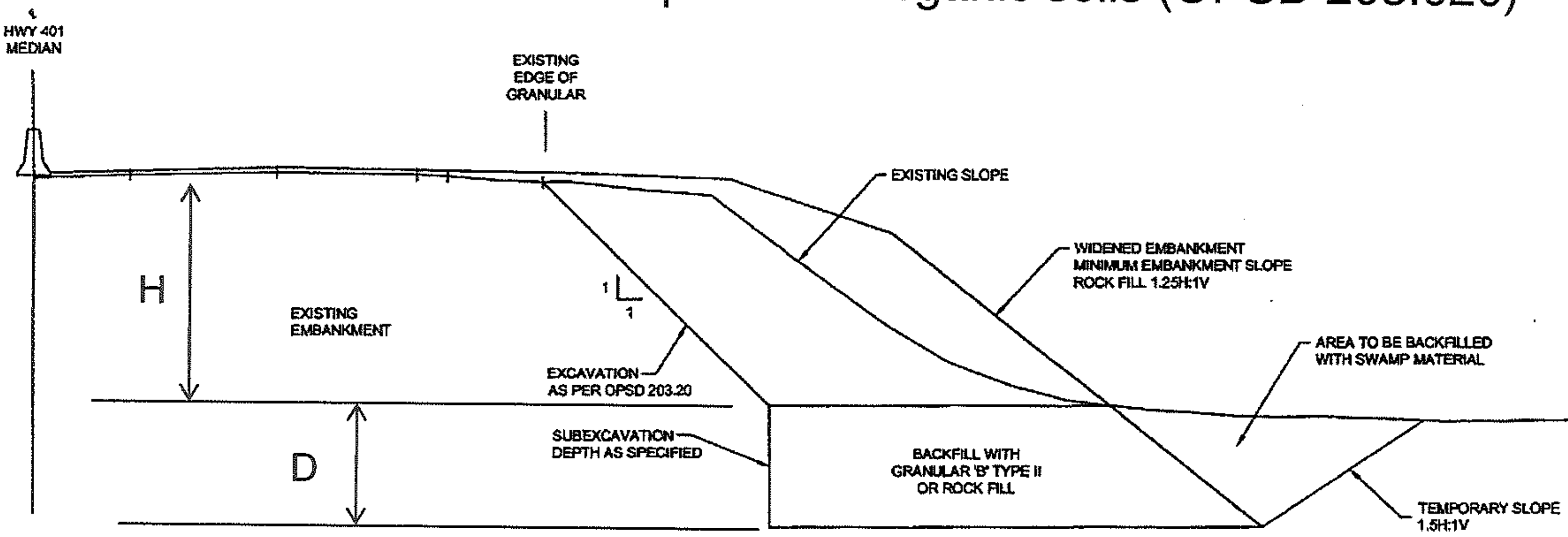
Highway Section and Approximate Station	Distance from CN Bridge Abutment	Estimated Settlement during construction	Estimated Post-Construction Settlement with Full Subexcavation of Organics at Toe after 10 years	Post Constuction Settlement Criteria	Meets Settlement Criteria at Edge of FPS
Section A 26+720 to 26+825	0 – 30 m* 30-70 m 70 -170 m	δ_{FPS} = 55 mm δ_{TOE} = 180 mm	δ_{FPS} = 25 mm δ_{TOE} = 80 mm	10 - 25 mm* 25 - 50 mm 50 - 100 mm	yes
Section B 26+825 to 26+925	70-170 m (NW corner) > 170 m	δ_{FPS} = 65 mm δ_{TOE} = 420 mm	δ_{FPS} = 90 mm δ_{TOE} = 130 mm	50 - 100 mm 100 - 200 mm	yes
Section C 26+925 to 27+050	> 170 m	δ_{FPS} = 95 mm δ_{TOE} = 200 mm	δ_{FPS} = 110 mm δ_{TOE} = 65 mm	100 - 200 mm	yes
Section D 27+050 to 27+175	> 170 m	δ_{FPS} = 110 mm δ_{TOE} = 275 mm	δ_{FPS} = 80 mm δ_{TOE} = 85 mm	100 - 200 mm	yes
Section E 27+175 to 27+500	> 170 m	δ_{FPS} = 70 mm δ_{TOE} = 80 mm	δ_{FPS}= 65 mm δ_{TOE} = 30 mm	100 - 200 mm	yes
Section F 28+200 to 28+450	30-70 m 70 -170 m	δ_{FPS} = 70 mm δ_{TOE} = 85 mm	$\delta_{(FPS}$= 45 mm δ_{TOE} = 30 mm	25 - 50 mm 50-100 mm	yes

Stability Analysis Results

Highway Section and Approximate Station	Meets Global Stability Criteria	Subexcavation Procedures
Section A 26+720 to 26+825	yes	OPSD 203.020 staged excavation in strips of limited width temporary protection system required where organics > 4m thick
Section B 26+825 to 26+925	**NO**	**NEED STABILITY MITIGATION**
Section C 26+925 to 27+050	yes	OPSD 203.020 staged excavation in strips of limited width
Section D 27+050 to 27+175	yes	OPSD 203.020 staged excavation in strips of limited width
Section E 27+175 to 27+500	yes	OPSD 203.020 staged excavation in strips of limited width
Section F 28+200 to 28+450	yes	OPSD 203.020 staged excavation in strips of limited width temporary protection system required where organics > 4m thick

Subexcavation Procedures
Typical Section

Full sub-excavation of compressible organic soils (OPSD 203.020)



- sub-aqueous excavation, backfilled with rock fill
- H > 5 m: excavation of embankment to 1H:1V limited to 200 m long for 6 weeks
- H > 7 m: excavation of embankment to 1H:1V limited to 100 m long for 6 weeks
- D >1 m: staged excavation in short sections < 3 m wide
- D > 4 m: temporary protection system needed

Stability Mitigation Options Section B

Partial sub-excavation of soft cohesive soils under organics

- carried out during original embankment construction
- would require extensive temporary shoring to support 6.5 - 8.5 m embankment and 8.5 m deep cut

Modified geometry (flattened slopes, toe berms)

- not effective in improving global stability (adds more driving force)

Staged construction (with or with/out wick drains)

- not enough load to strengthen weak clays and improve stability

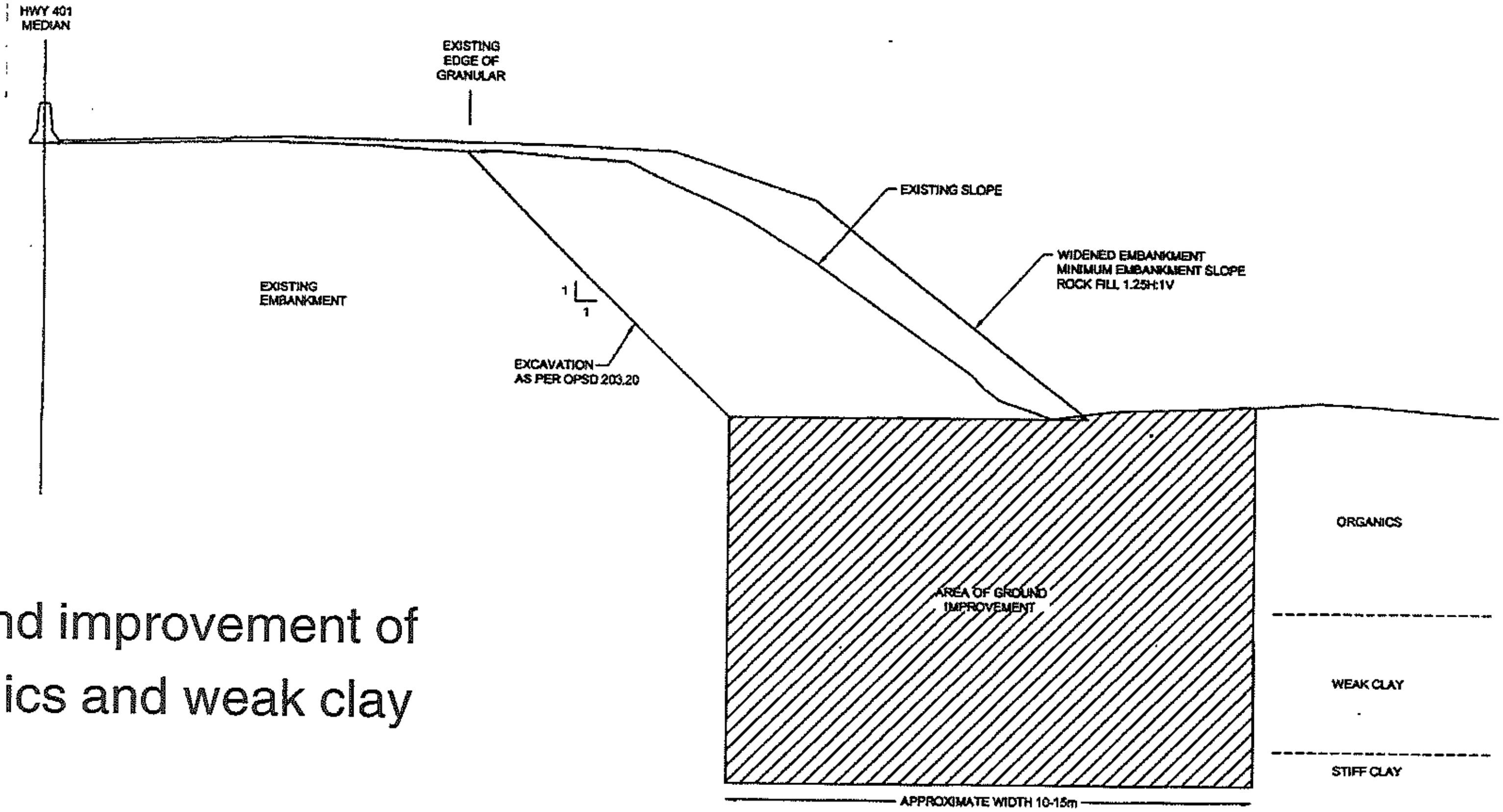
Lightweight Fill

- expensive, would still need extensive temporary shoring to excavate and replace organics

Ground Improvement

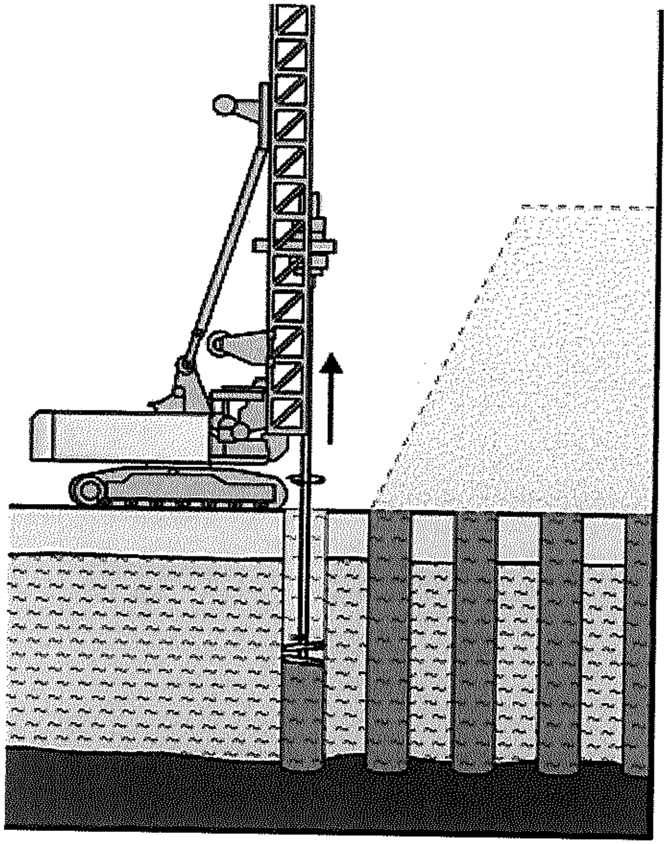
- preferred stability mitigation option
- limits risks to existing highway by eliminating below grade excavation

Typical Section
Section B



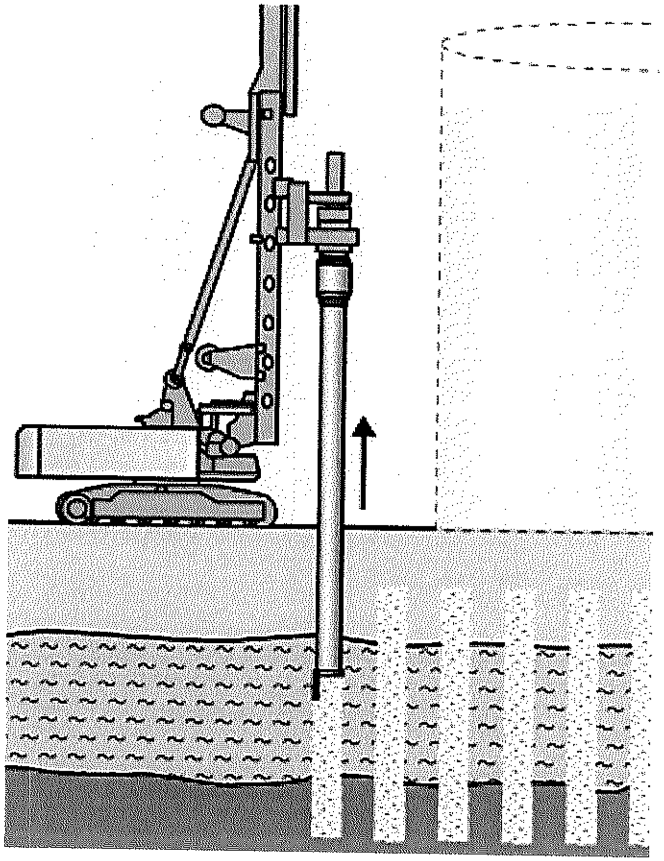
Ground improvement of organics and weak clay

Ground Improvement Techniques

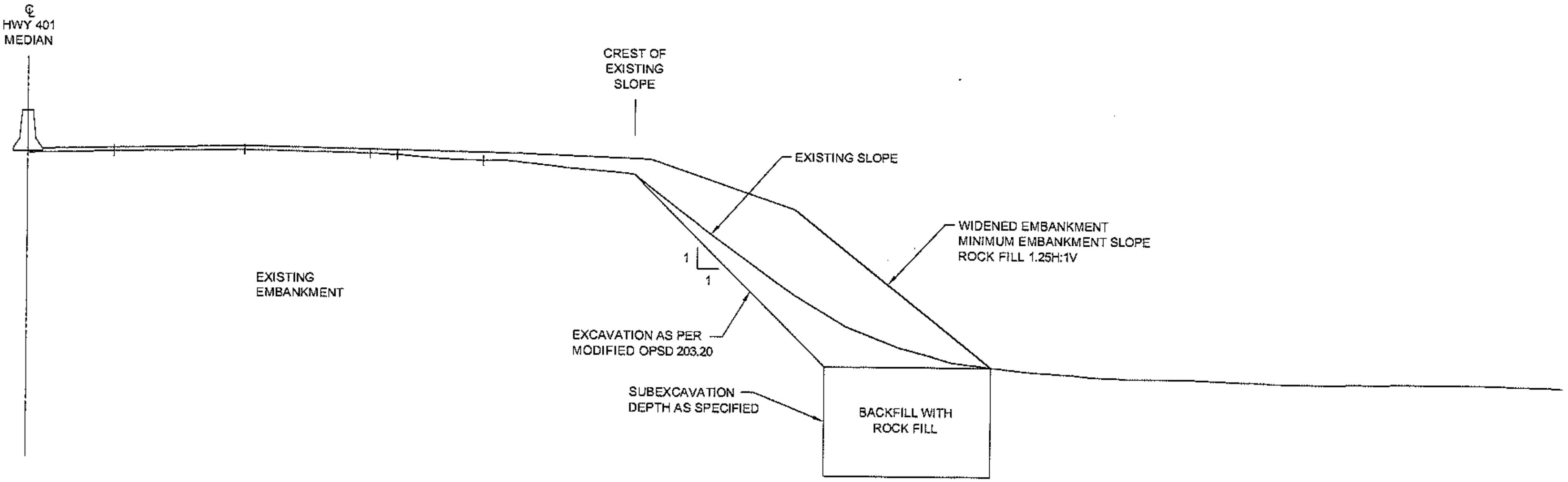


Deep soil mix columns or panels (barrettes or below grade wall)

Rigid inclusions



Next Steps



NOT TO SCALE

Golder Associates
Ottawa, Ontario, Canada

HIGHWAY 401 EMBANKMENT WIDENING

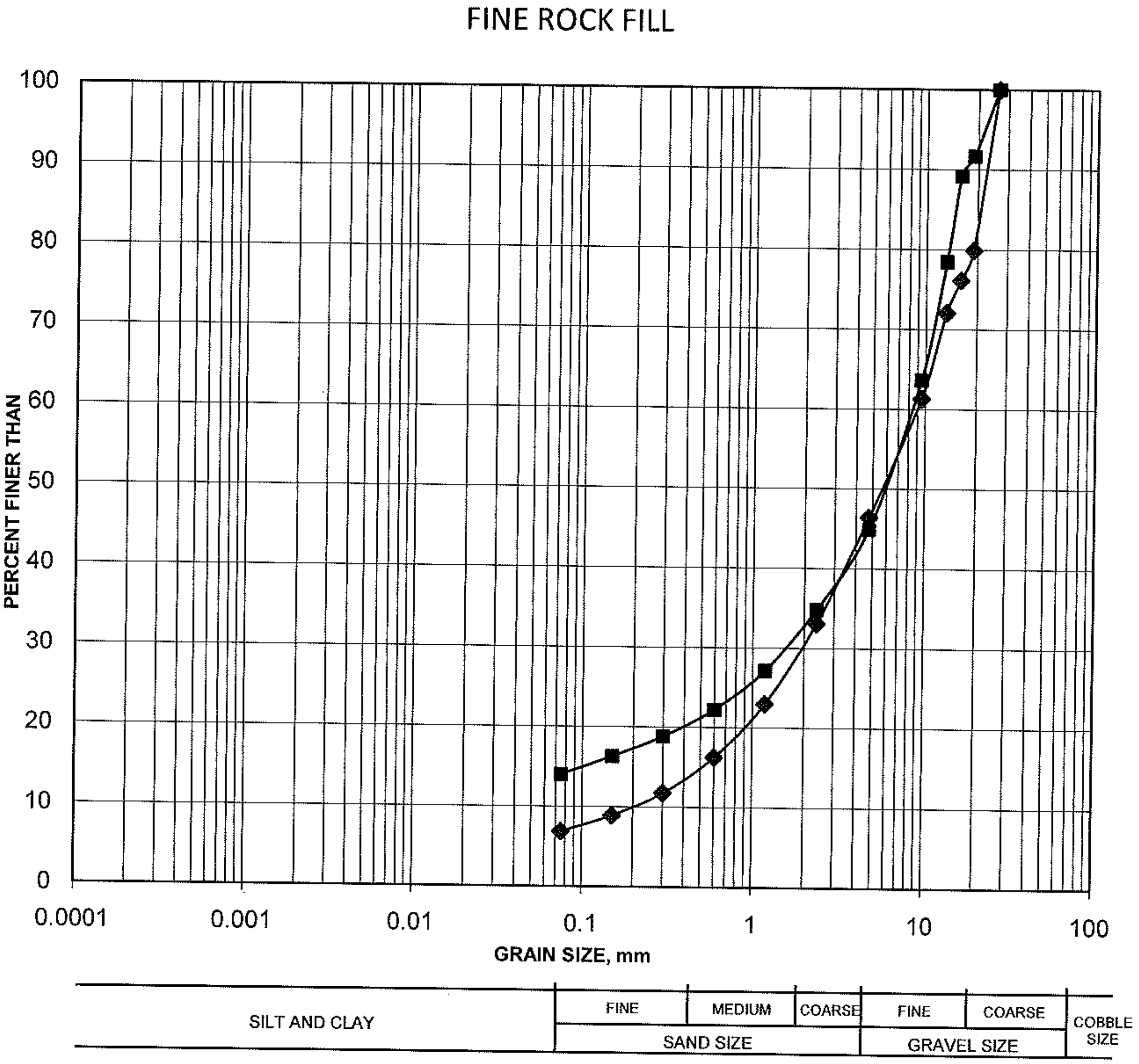
TYPICAL SECTION SUBEXCAVATION AND REPLACEMENT OF ORGANICS

PROJECT No. 08-1111-0044
DATE 27 MAR. 2012
CAD J.M.

DRAWING 5

GRAIN SIZE DISTRIBUTION

FIGURE 1

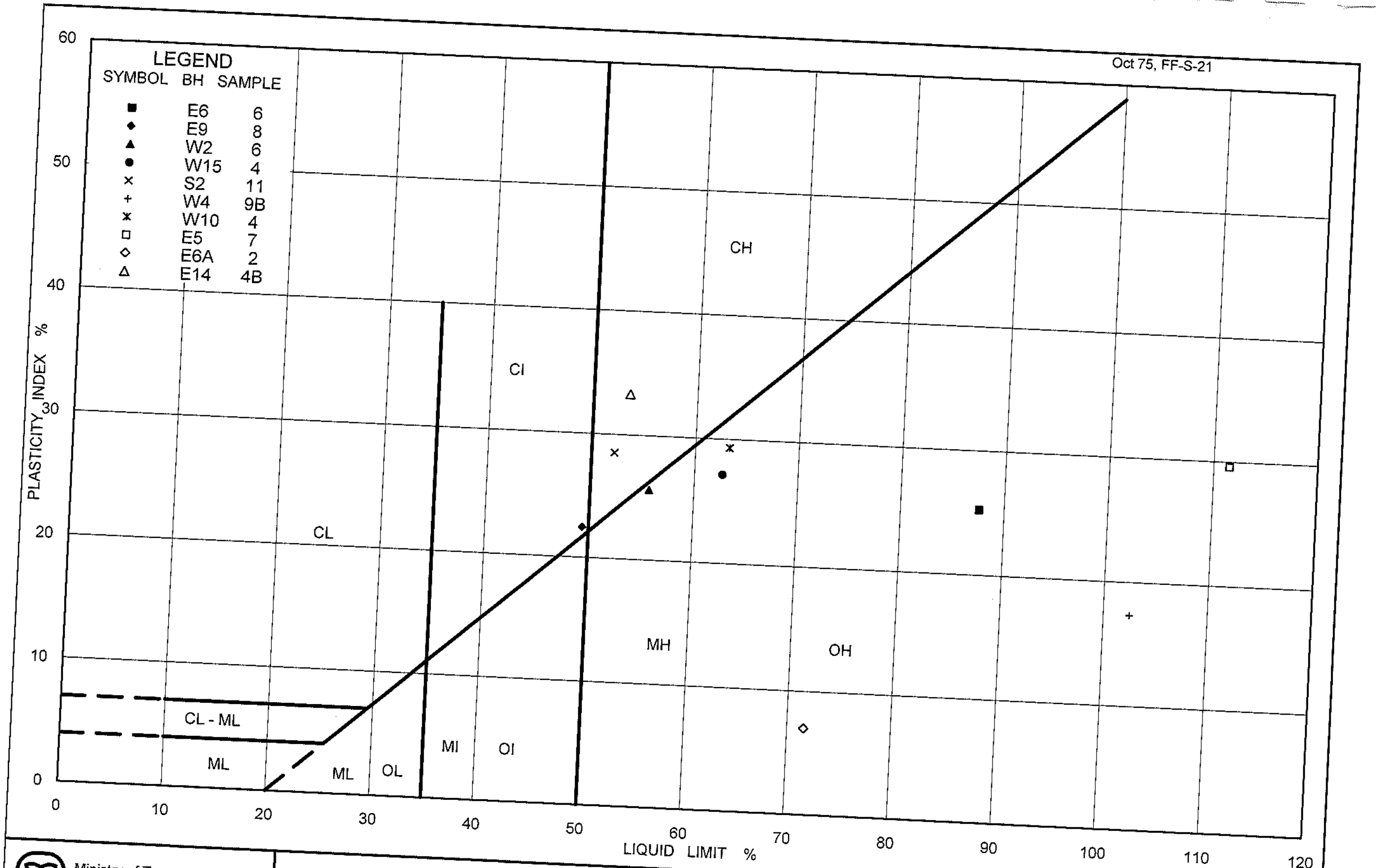


Borehole	Sample	Depth (m)
◆ S1	3	2.29-2.90
■ S1	14	10.67-11.28

Project: 08-1111-0044

Golder Associates

Created by:	CNM
Checked by:	ESO



Ministry of Transportation

Ontario

PLASTICITY CHART

ORGANIC DEPOSITS

FIG No. 2

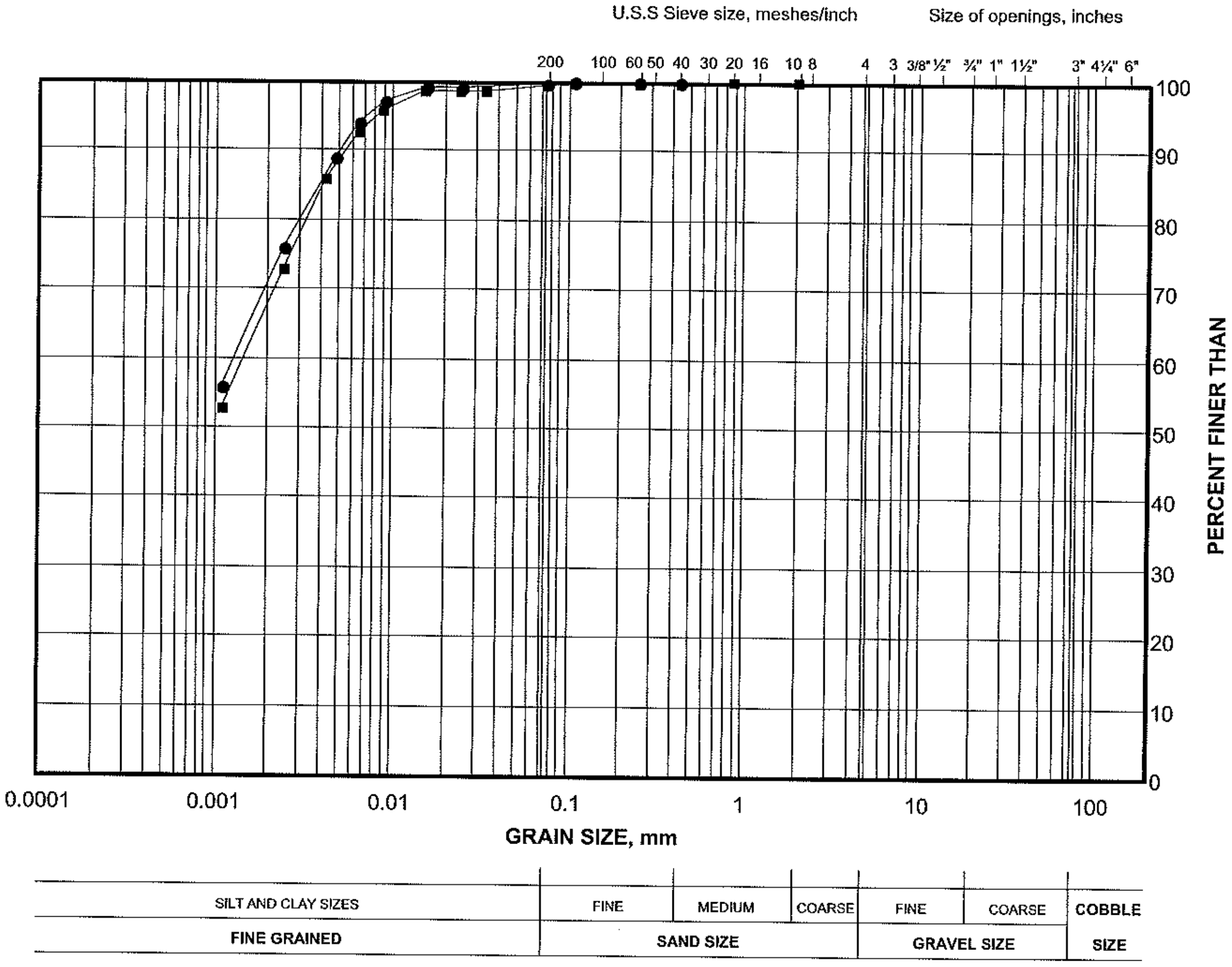
Project No. 08-1111-0044

Checked by: KSL

GRAIN SIZE DISTRIBUTION

Clay

FIGURE 8



LEGEND

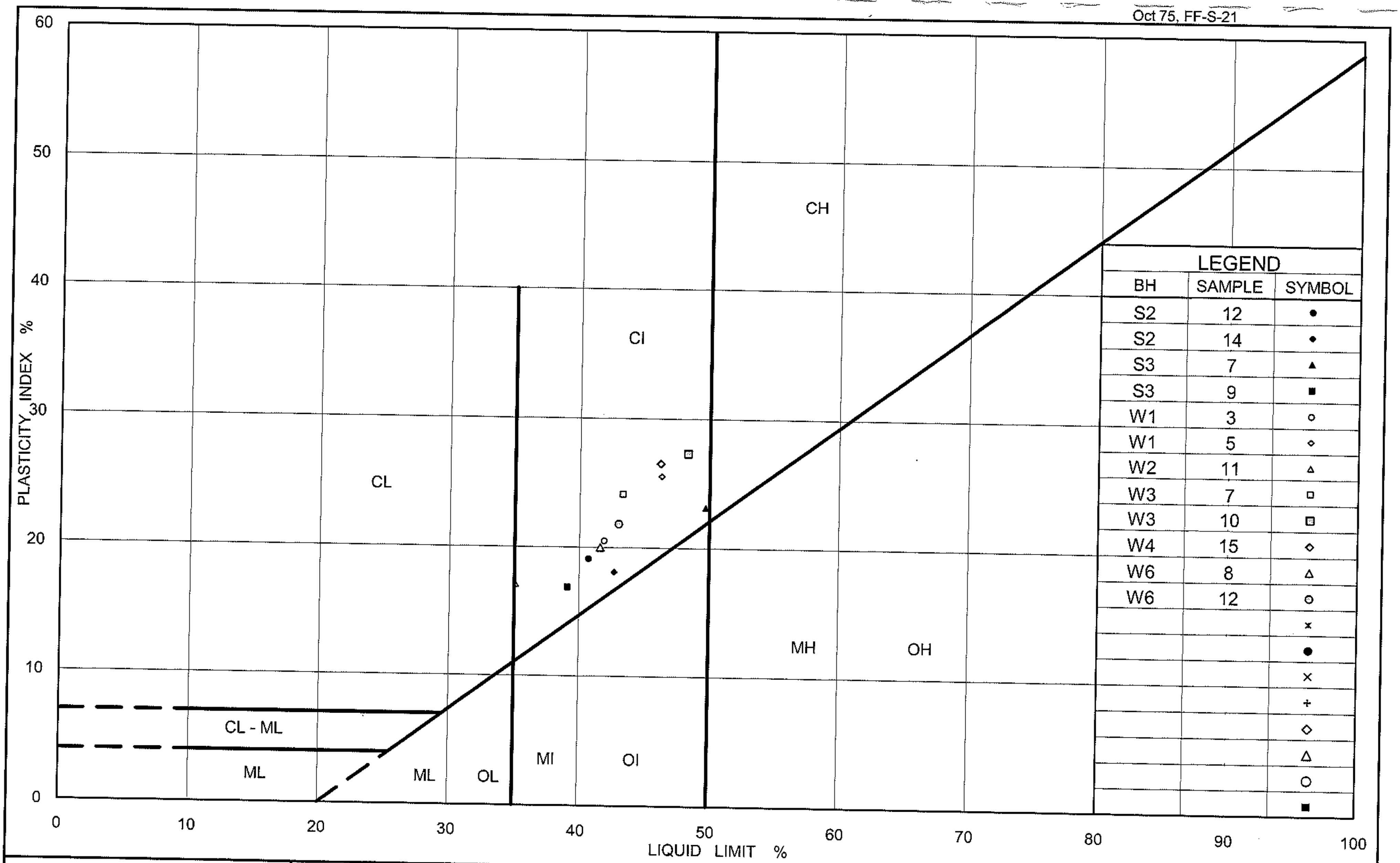
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	E5	13	66.5
■	W5	17	62.5

Project Number: 08-1111-0044

Checked By: KSL

Golder Associates

Date: 30-Mar-10



Ministry of Transportation

Ontario

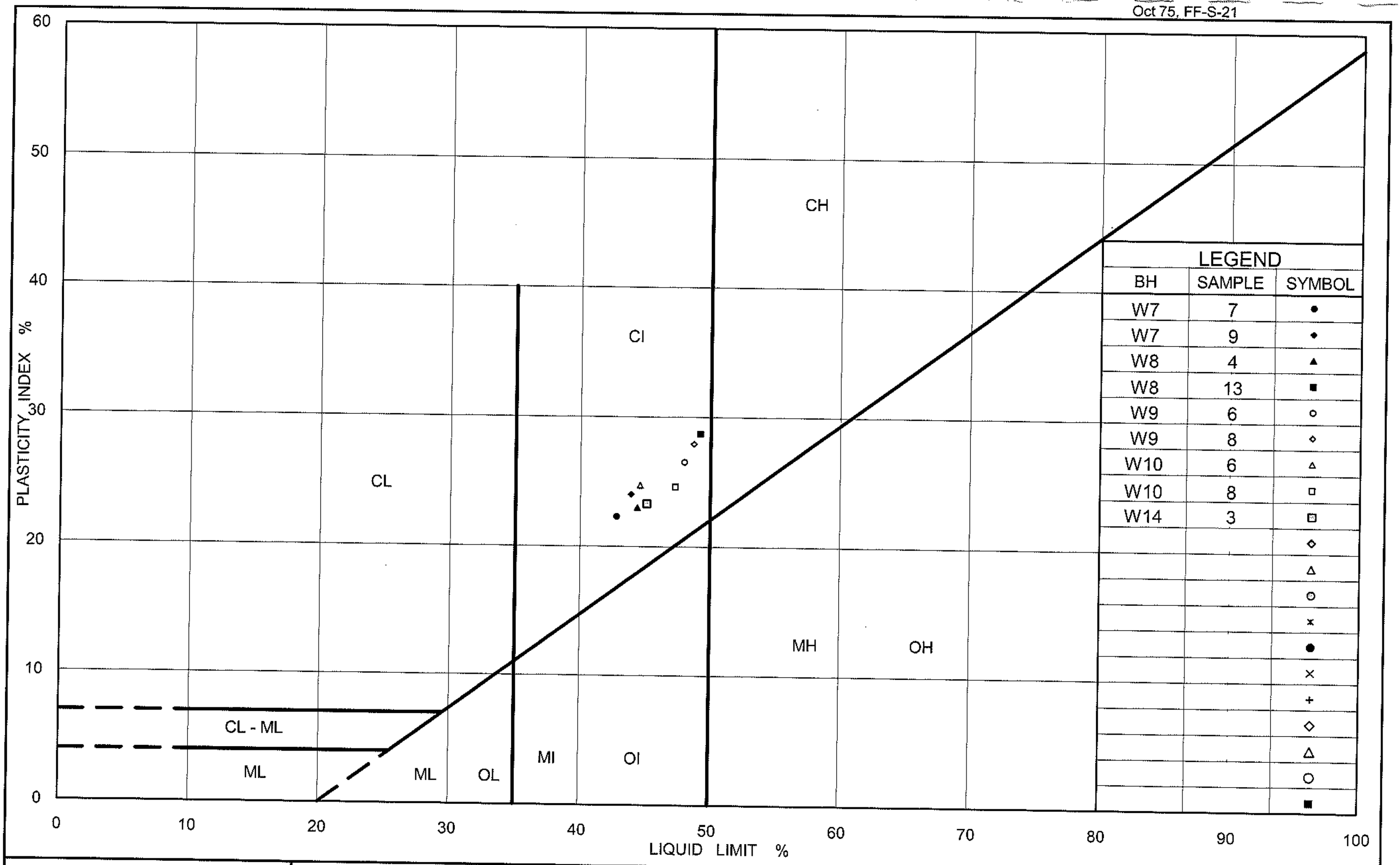
PLASTICITY CHART
Silty Clay

Figure No. 9A

Project No. 08-1111-0044

Checked By: KSL

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
W7	7	●
W7	9	◆
W8	4	▲
W8	13	■
W9	6	○
W9	8	◇
W10	6	△
W10	8	□
W14	3	⊡
		◇
		△
		⊙
		×
		●
		×
		+
		◇
		△
		○
		■

Ministry of Transportation

Ontario

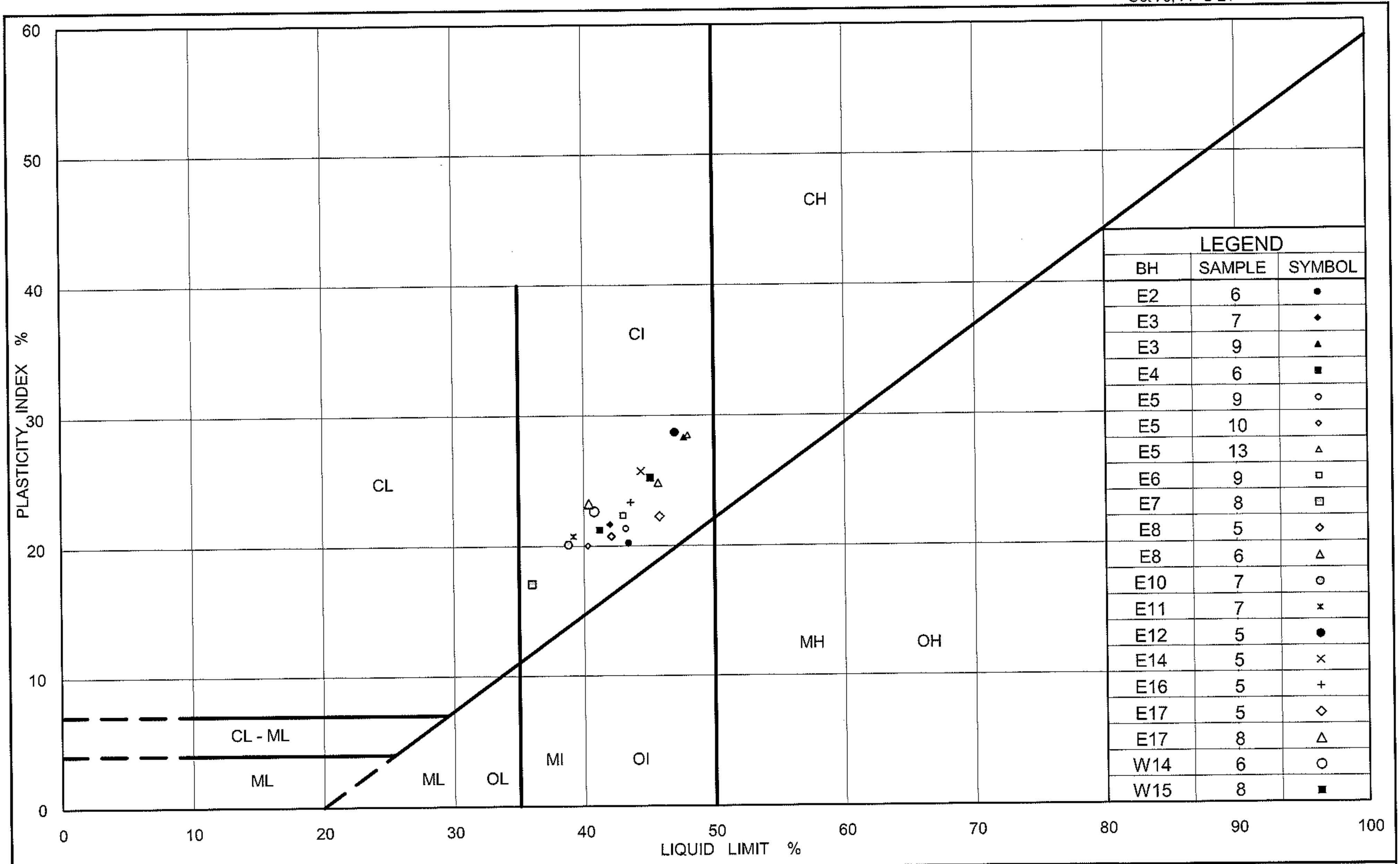
PLASTICITY CHART
Silty Clay

Figure No. 9B

Project No. 08-1111-0044

Checked By: KSL

Oct 75, FF-S-21



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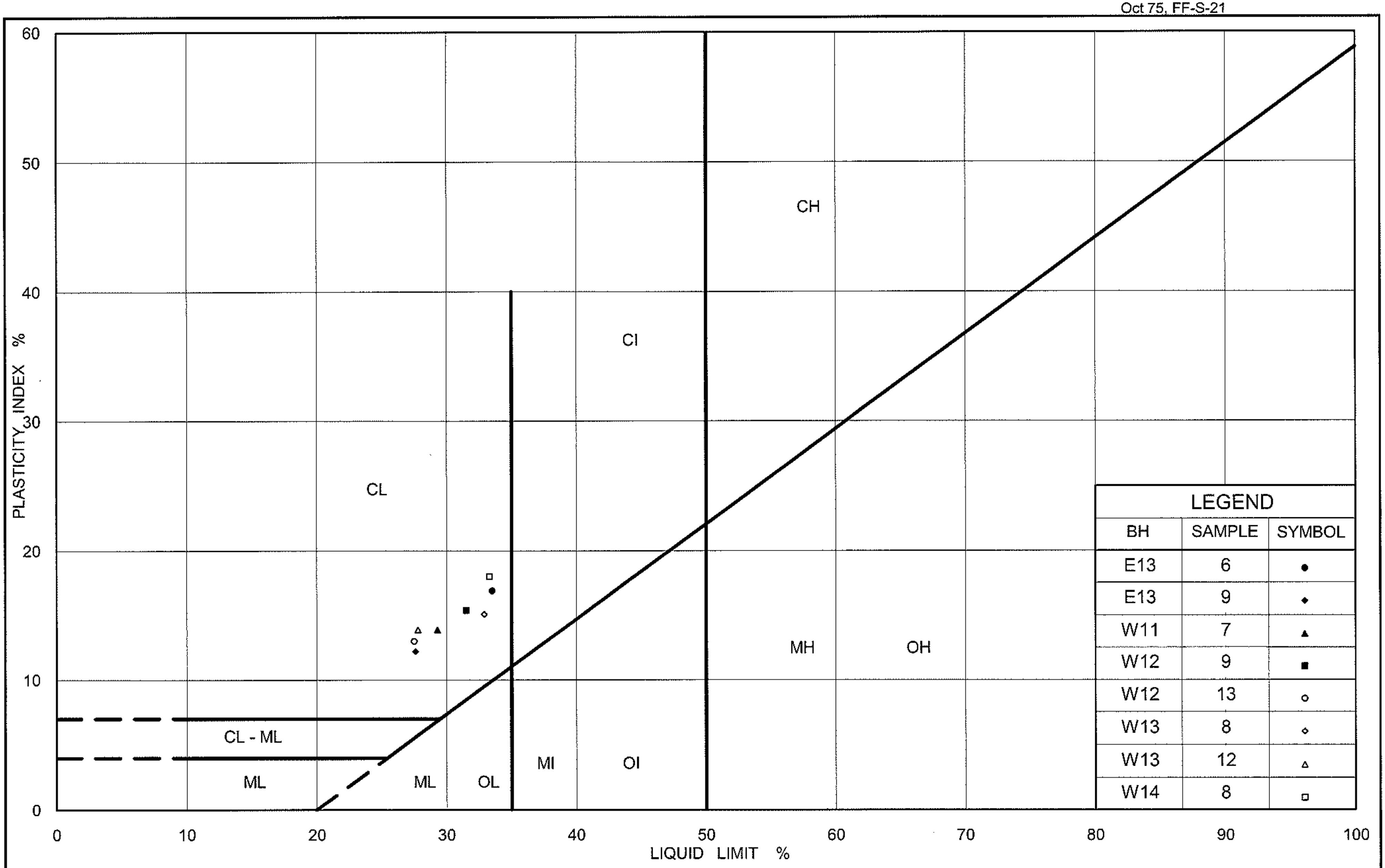
PLASTICITY CHART
Silty Clay

Figure No. 10

Project No. 08-1111-0044

Checked By: KSL

Oct 75, FF-S-21



BH	SAMPLE	SYMBOL
E13	6	●
E13	9	◆
W11	7	▲
W12	9	■
W12	13	○
W13	8	◇
W13	12	△
W14	8	□

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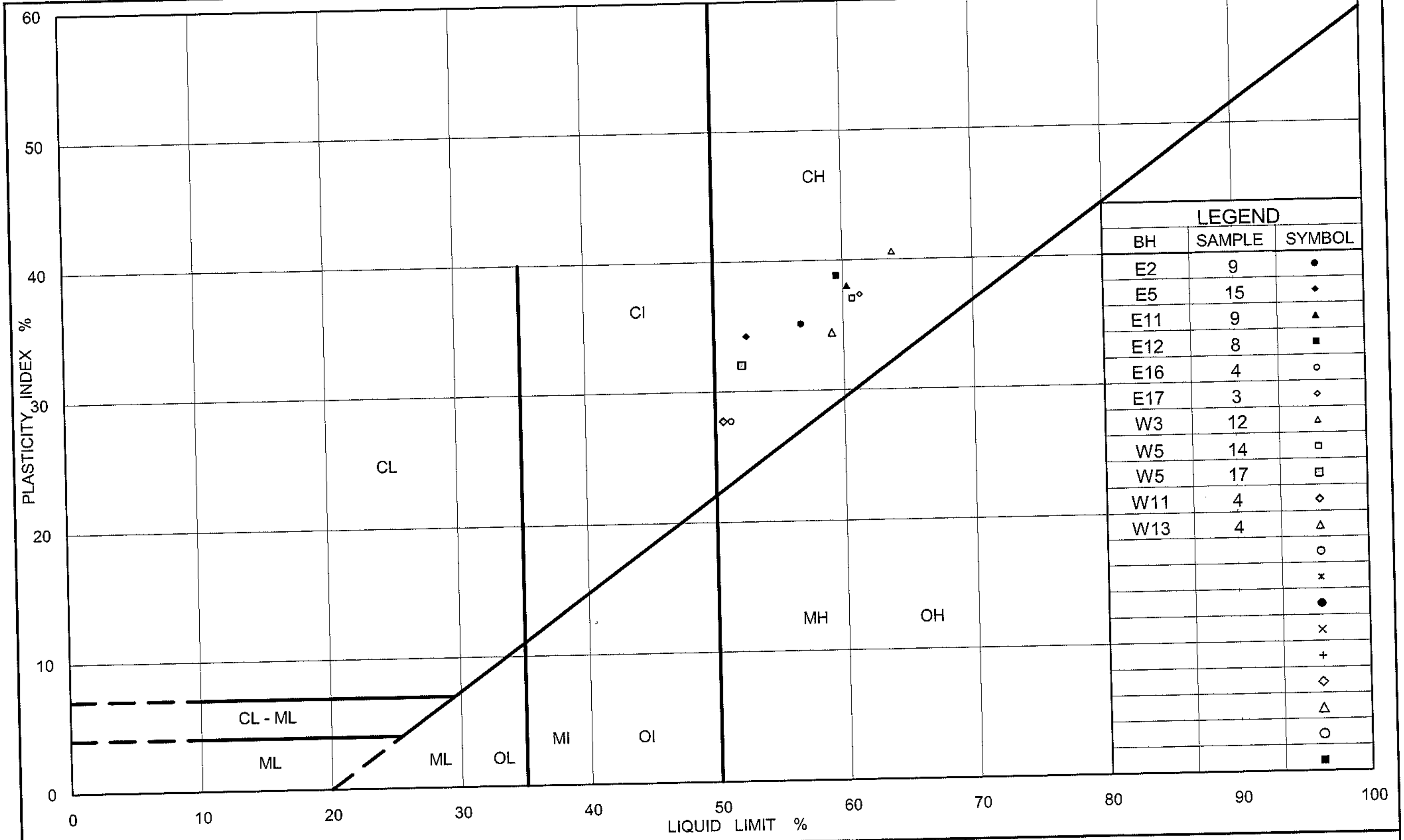
PLASTICITY CHART
Clayey Silt

Figure No. 11

Project No. 08-1111-0044

Checked By: KSL

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
E2	9	●
E5	15	◆
E11	9	▲
E12	8	■
E16	4	○
E17	3	◇
W3	12	△
W5	14	□
W5	17	⊡
W11	4	◇
W13	4	△
		○
		×
		●
		×
		+
		◇
		△
		○
		■

Ministry of
Transportation

Ontario

PLASTICITY CHART
Clay

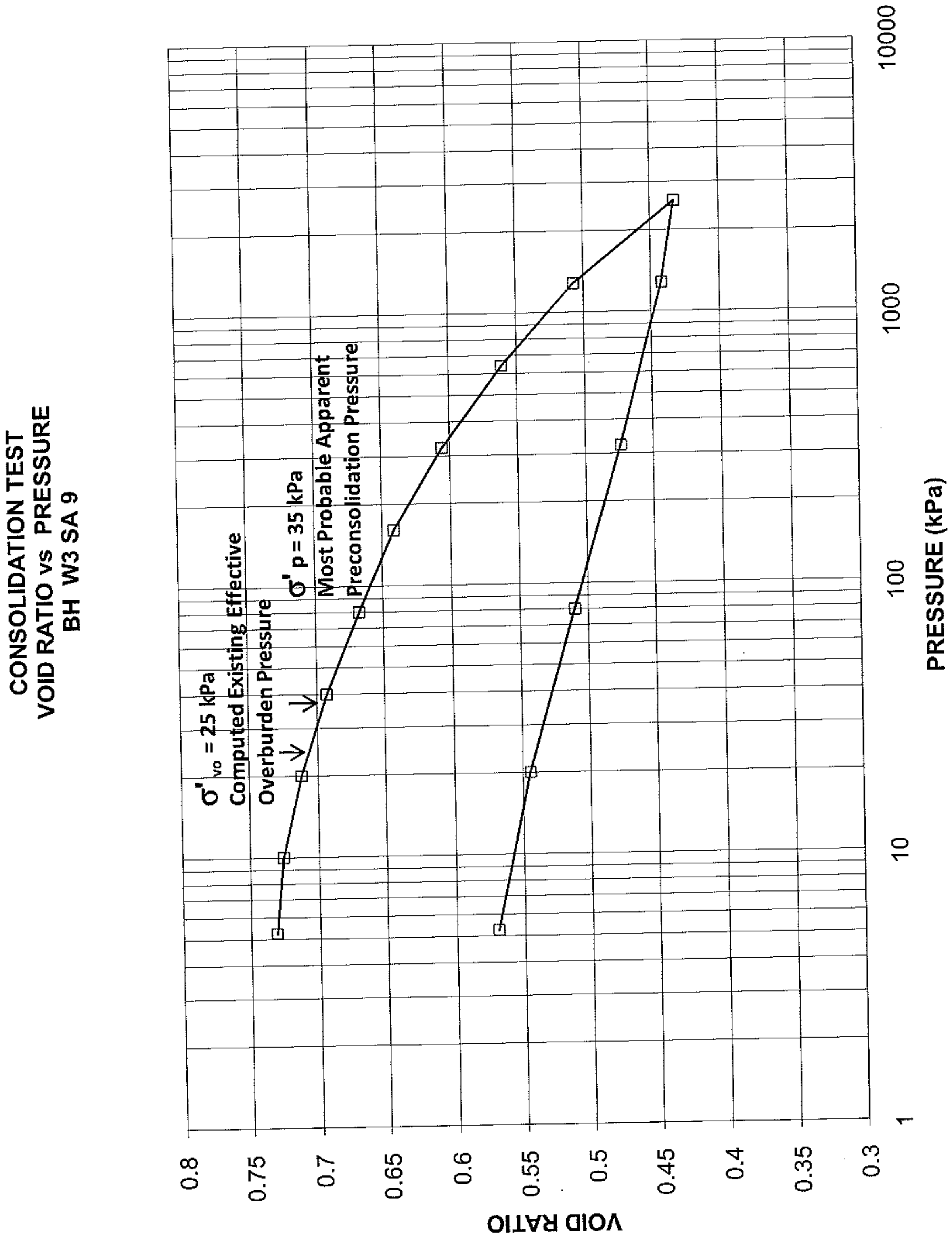
Figure No. 12

Project No. 08-1111-0044

Checked By: KSL

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE 13



Project No. 08-1111-0044

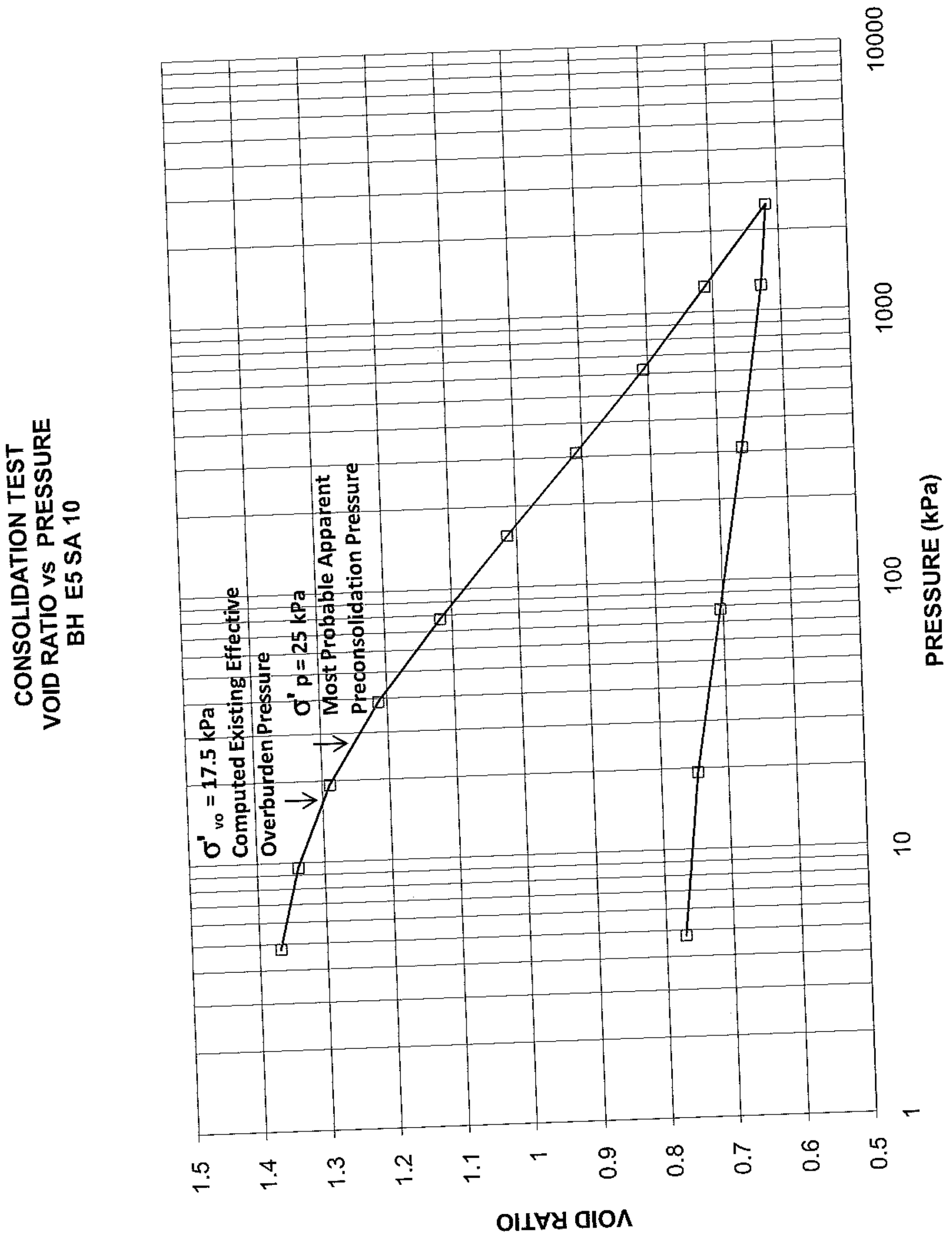
Prepared By: LFG

Golder Associates

Checked By: MM

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE 14



Project No. 08-1111-0044

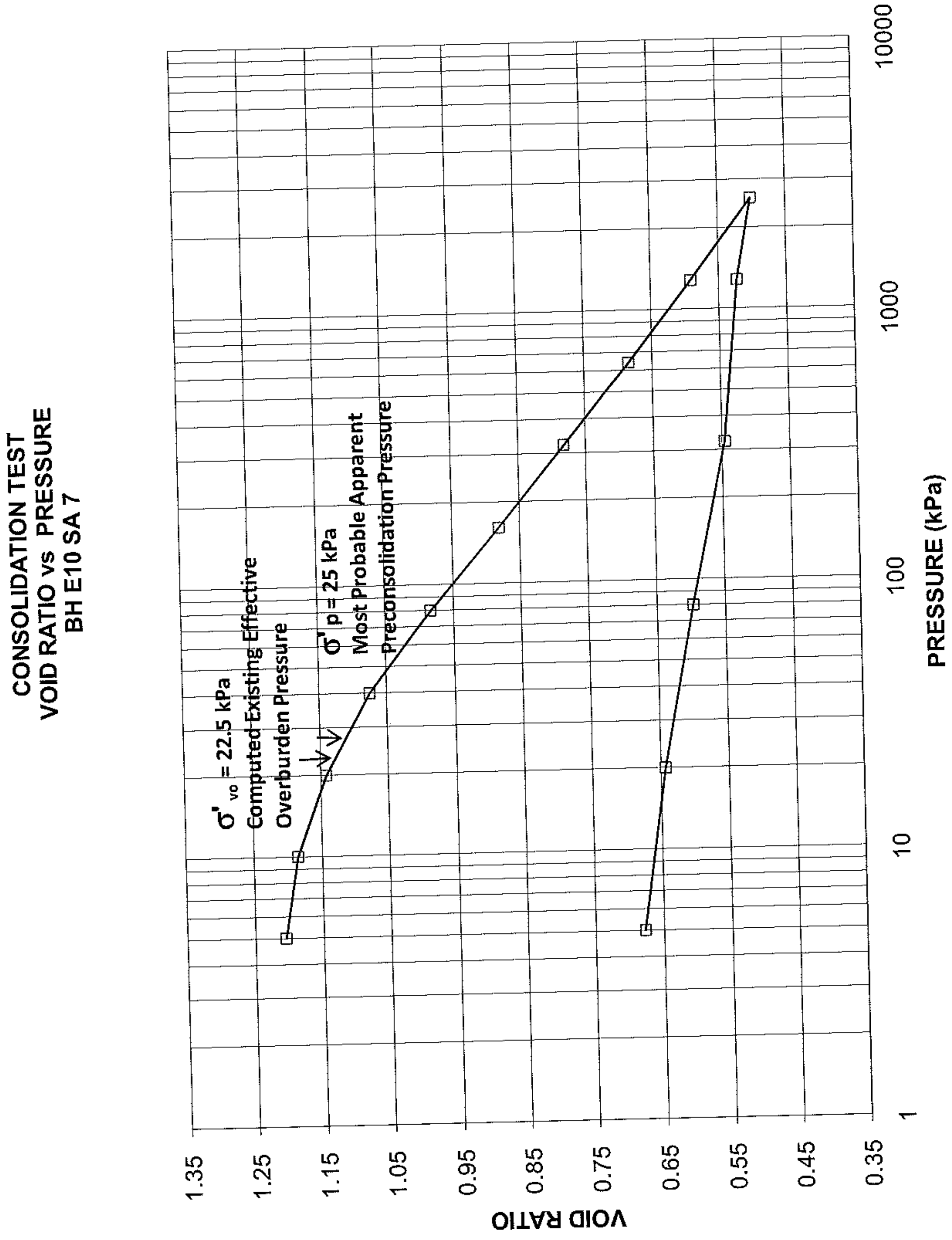
Prepared By: LFG

Golder Associates

Checked By: MM

CONSOLIDATION TEST
VOID RATIO VS. LOG PRESSURE

FIGURE 15



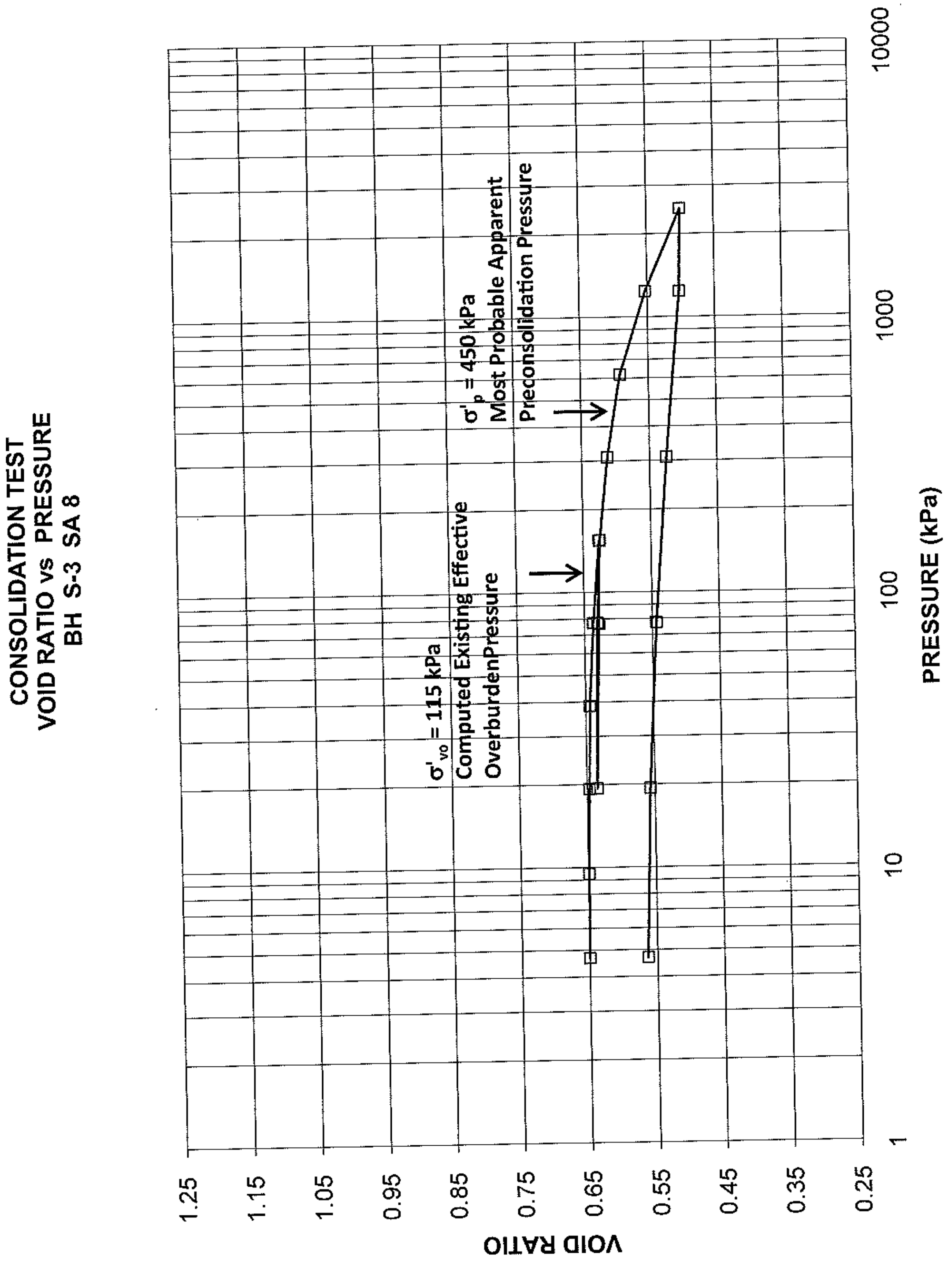
Project No. 08-1111-0044

Prepared By: LH

Golder Associates

Checked By: MM

CONSOLIDATION TEST VOID RATIO VS LOG PRESSURE	FIGURE 16



Project No. 08-1111-0044

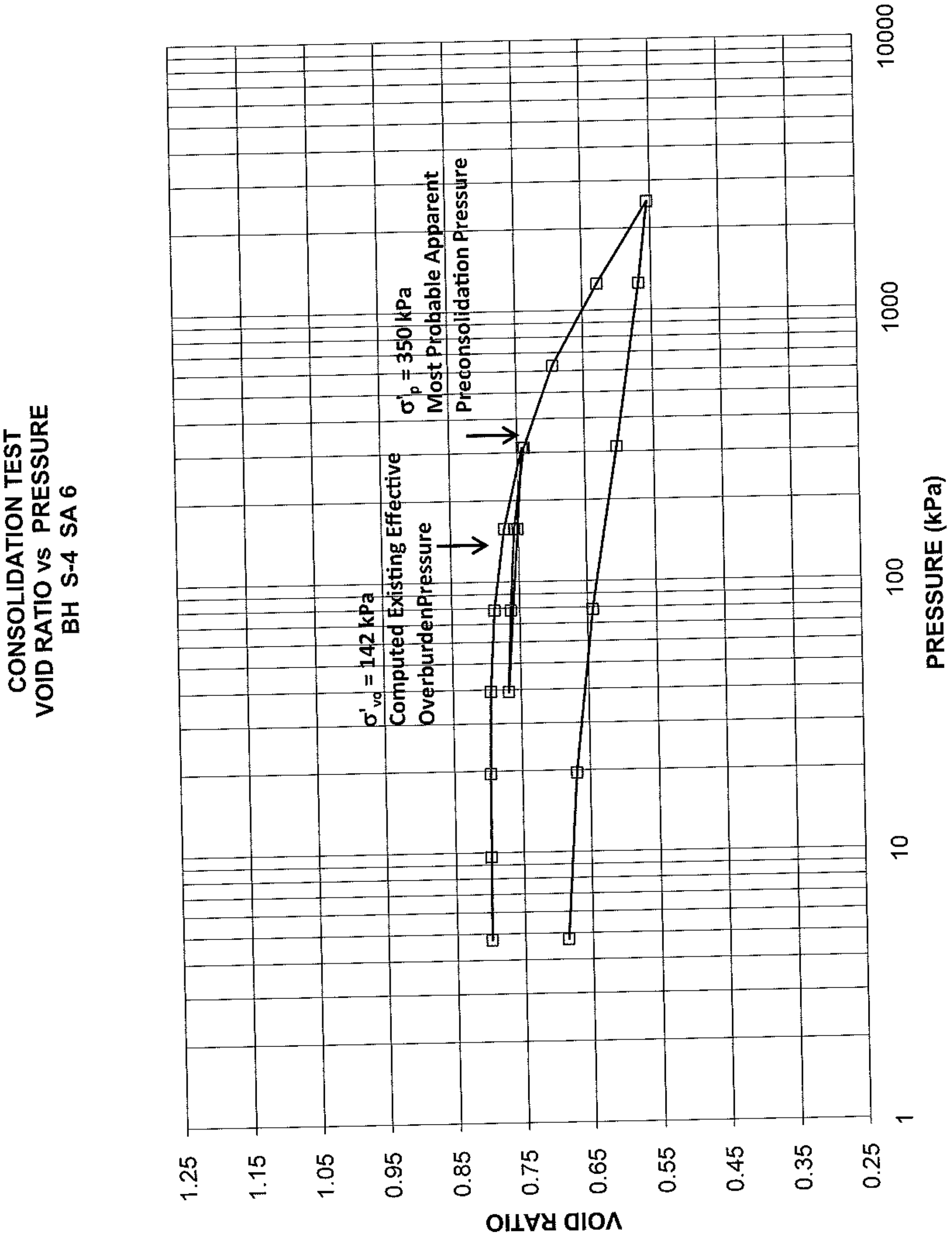
Prepared By: LFG

Golder Associates

Checked By: ESO

CONSOLIDATION TEST
VOID RATIO VS LOG PRESSURE

FIGURE 17



Project No. 08-1111-0044

Prepared By: LFG

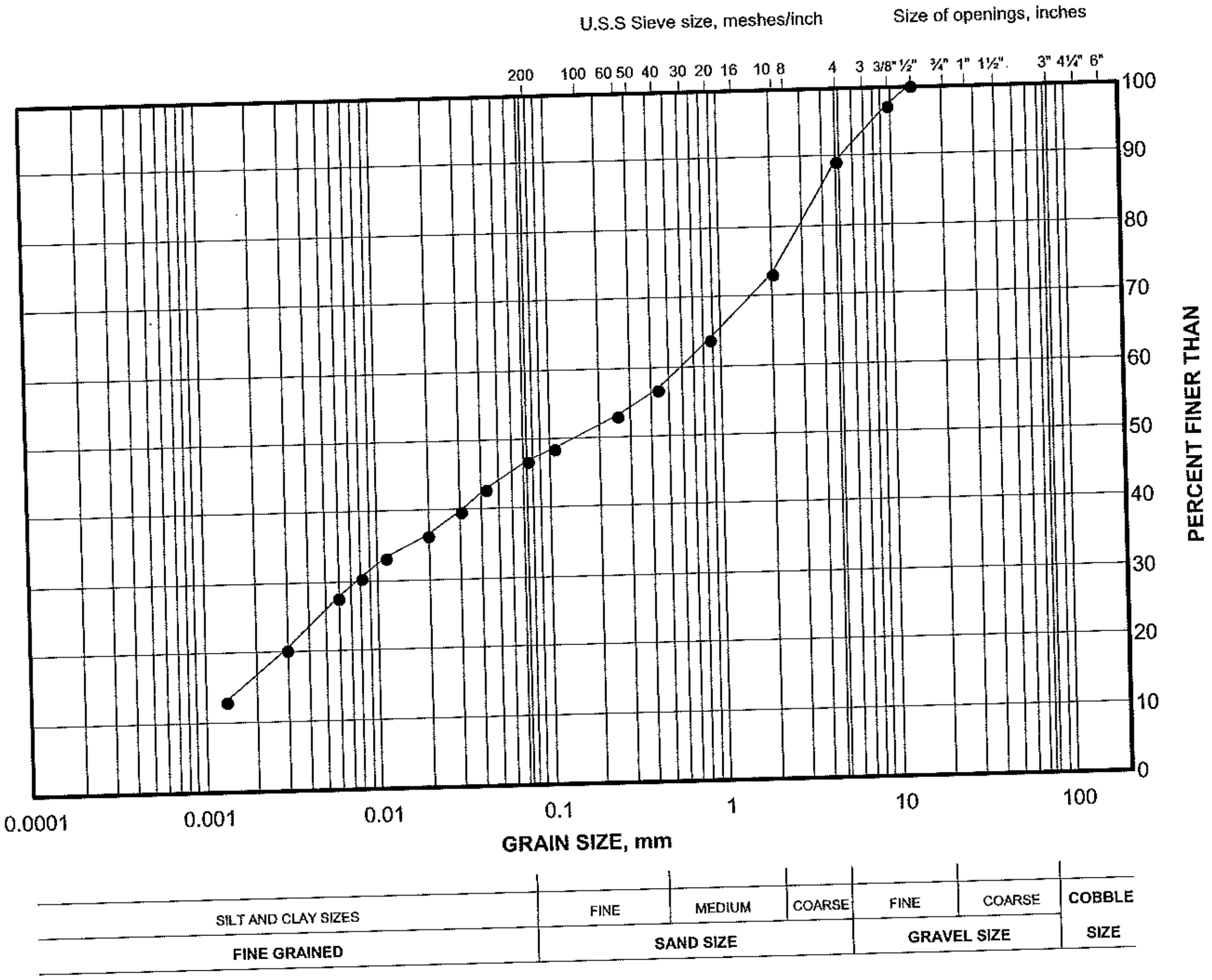
Golder Associates

Checked By: ESO

GRAIN SIZE DISTRIBUTION

Silty Sand

FIGURE 18



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	E17	12	68.2

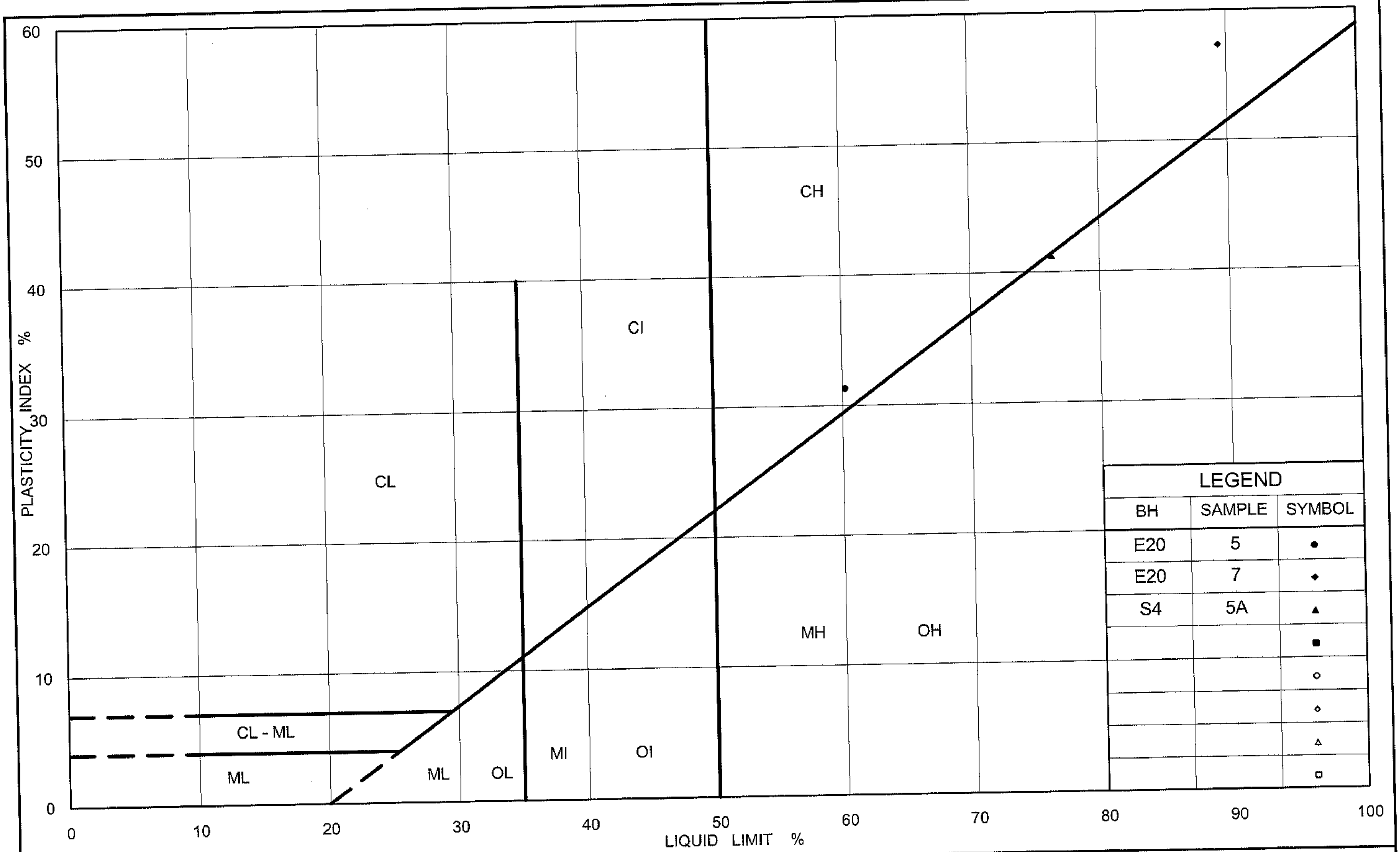
Project Number: 08-1111-0044

Checked By: KSL

Golder Associates

Date: 30-Mar-10

Oct 75, FF-S-21



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PLASTICITY CHART
Organic Clay

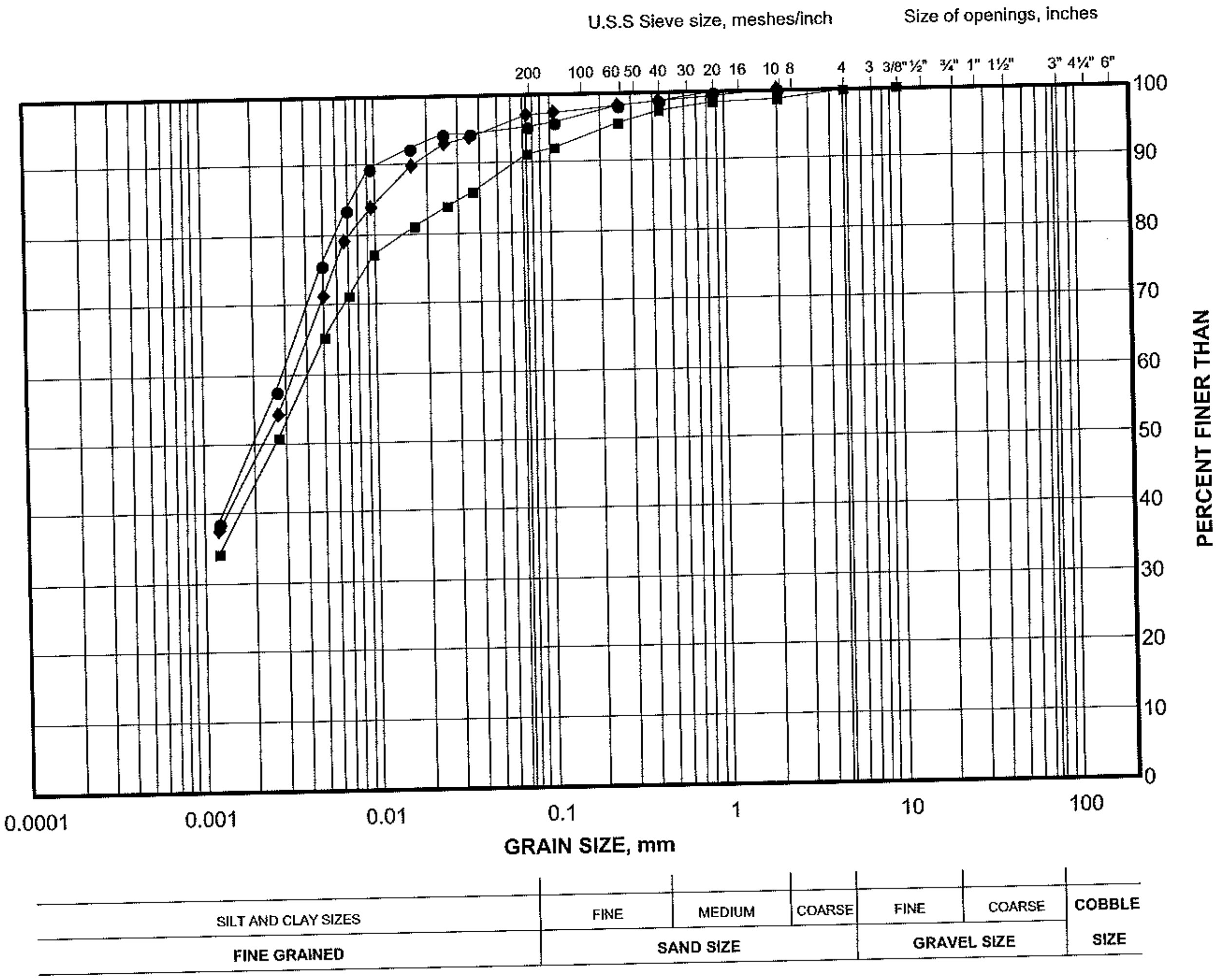
Figure No. 19

Project No. 08-1111-0044

Checked By: KSL

GRAIN SIZE DISTRIBUTION
Silty Clay

FIGURE 20



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	E22	10	71.3
■	E24	3	73.6
◆	E23	3	75.1

Project Numb er: 08-1111-0044

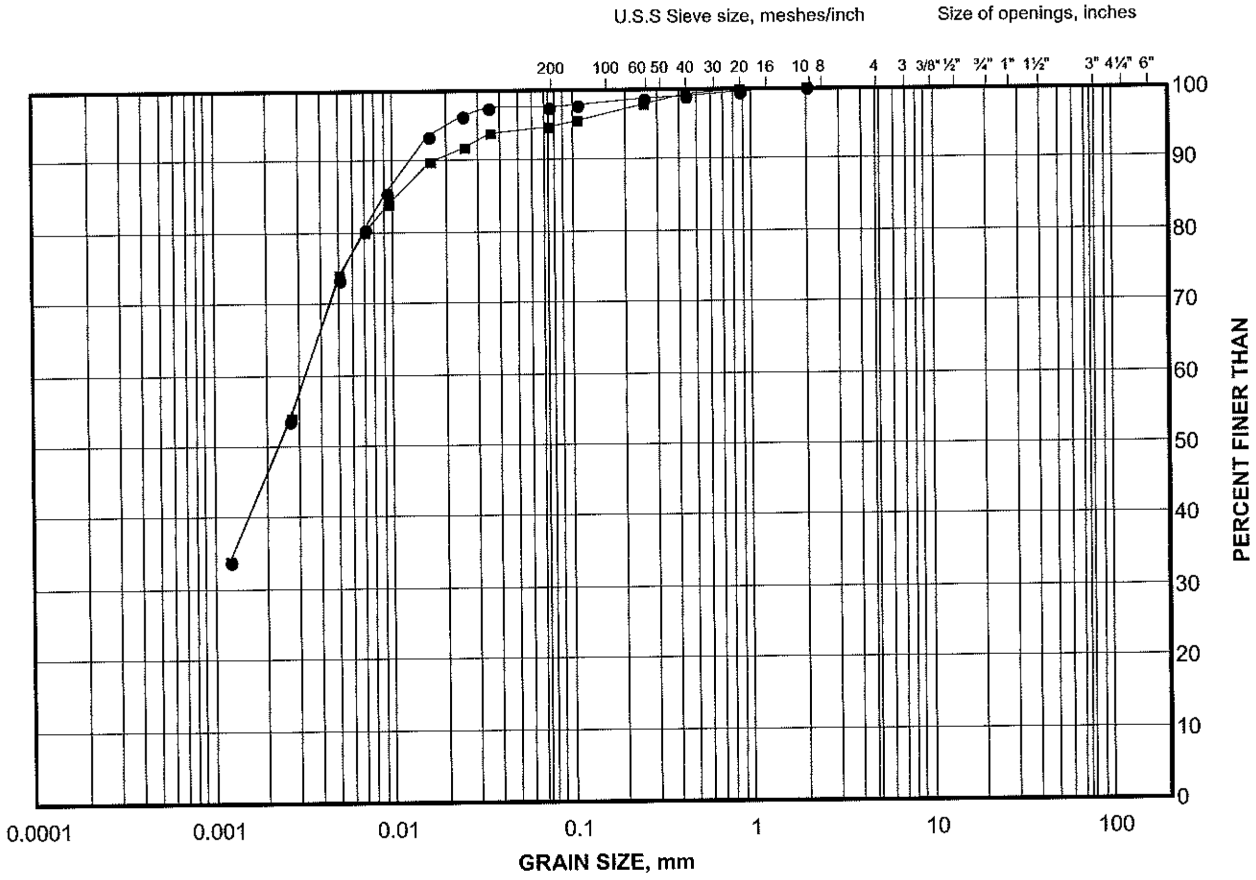
Checked By: KSL

Golder Associates

Date: 30-Mar-10

GRAIN SIZE DISTRIBUTION
Clayey Silt

FIGURE 21



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

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	E23B	3	75.2
■	E23B	5	74.0

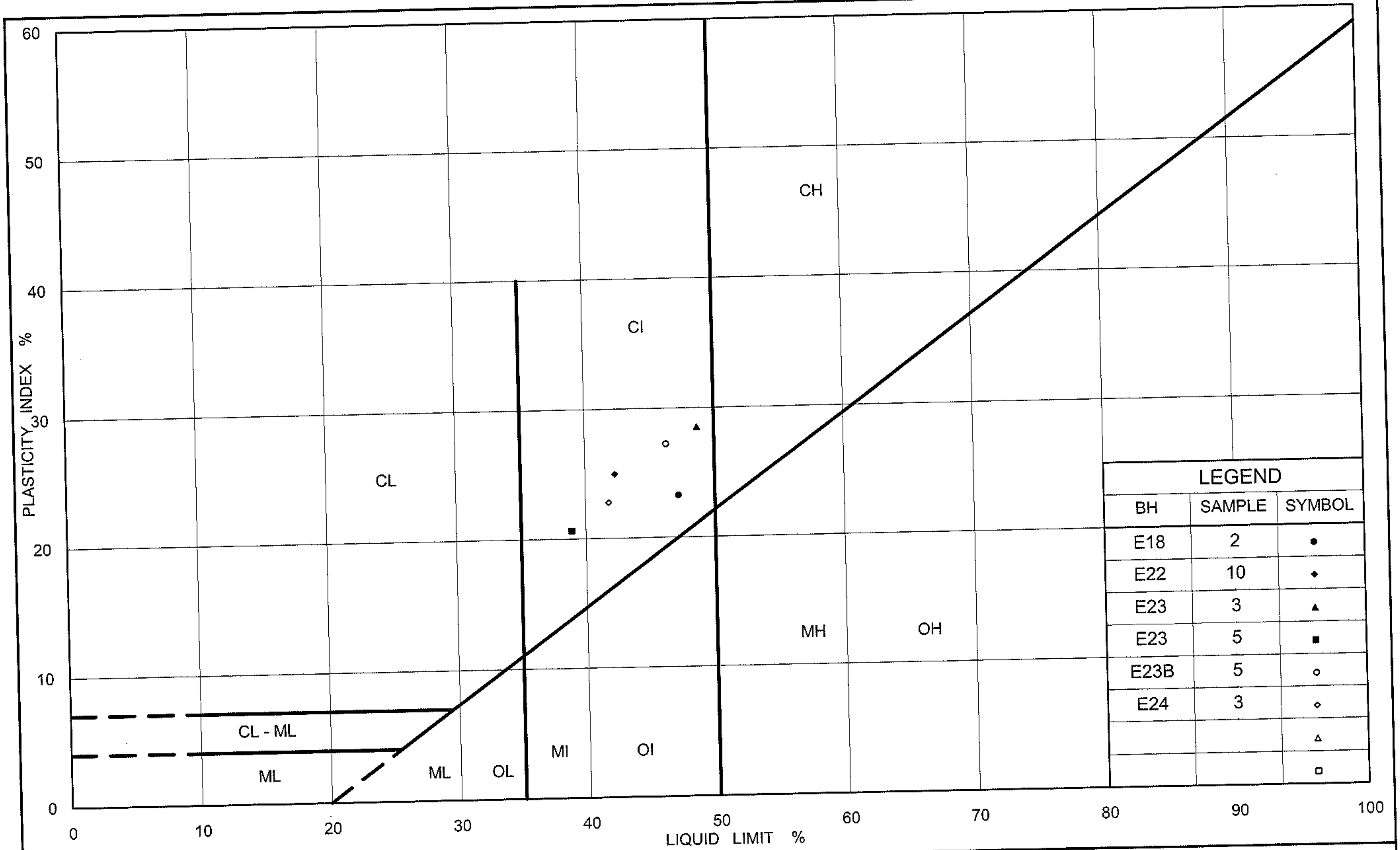
Project Number: 08-1111-0044

Checked By: KSL

Golder Associates

Date: 30-Mar-10

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
E18	2	●
E22	10	◆
E23	3	▲
E23	5	■
E23B	5	○
E24	3	◇
		△
		□

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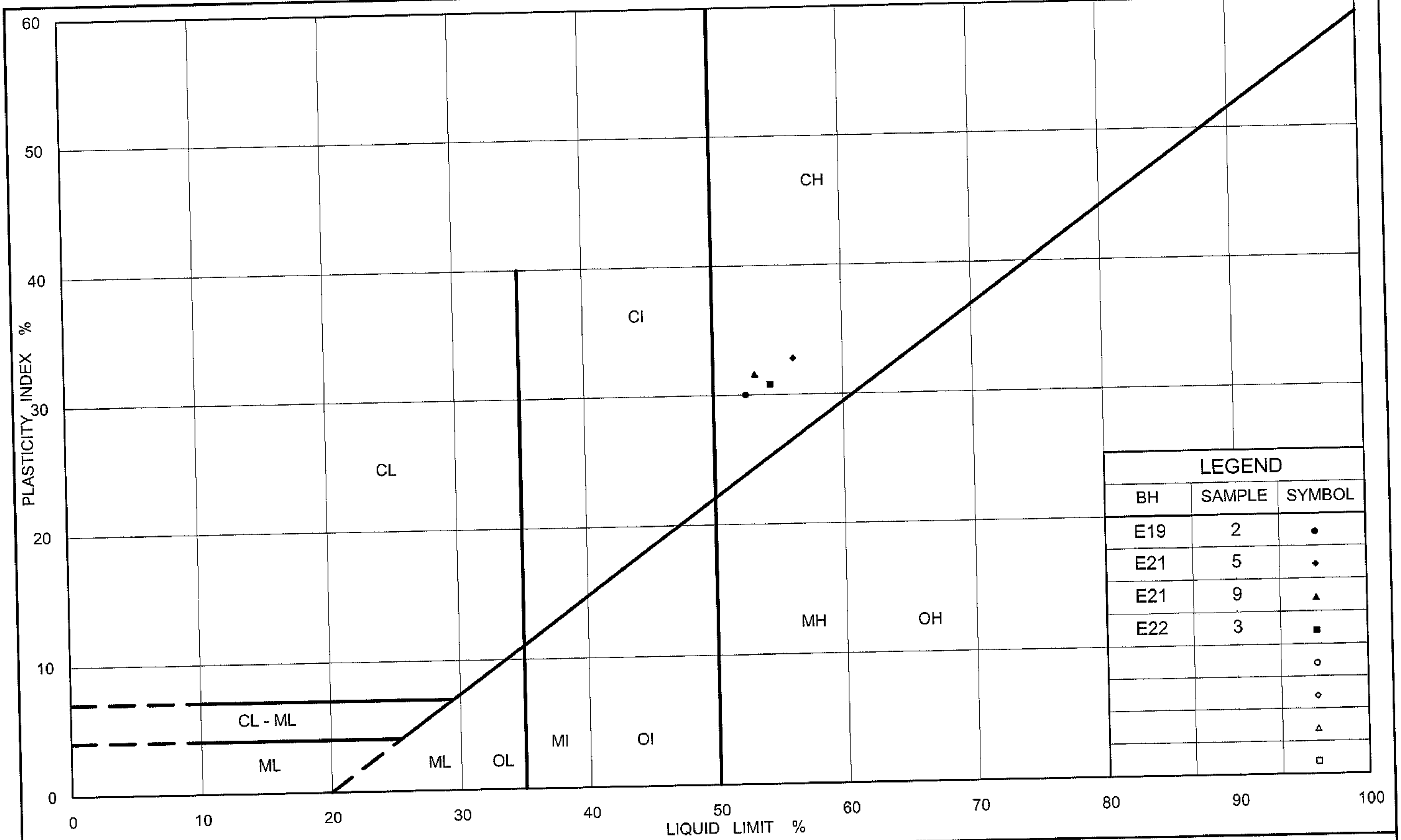
PLASTICITY CHART
Silty Clay

Figure No. 22

Project No. 08-1111-0044

Checked By: KSL

Oct 75, FF-S-21



BH	SAMPLE	SYMBOL
E19	2	●
E21	5	◆
E21	9	▲
E22	3	■
		○
		◇
		△
		□

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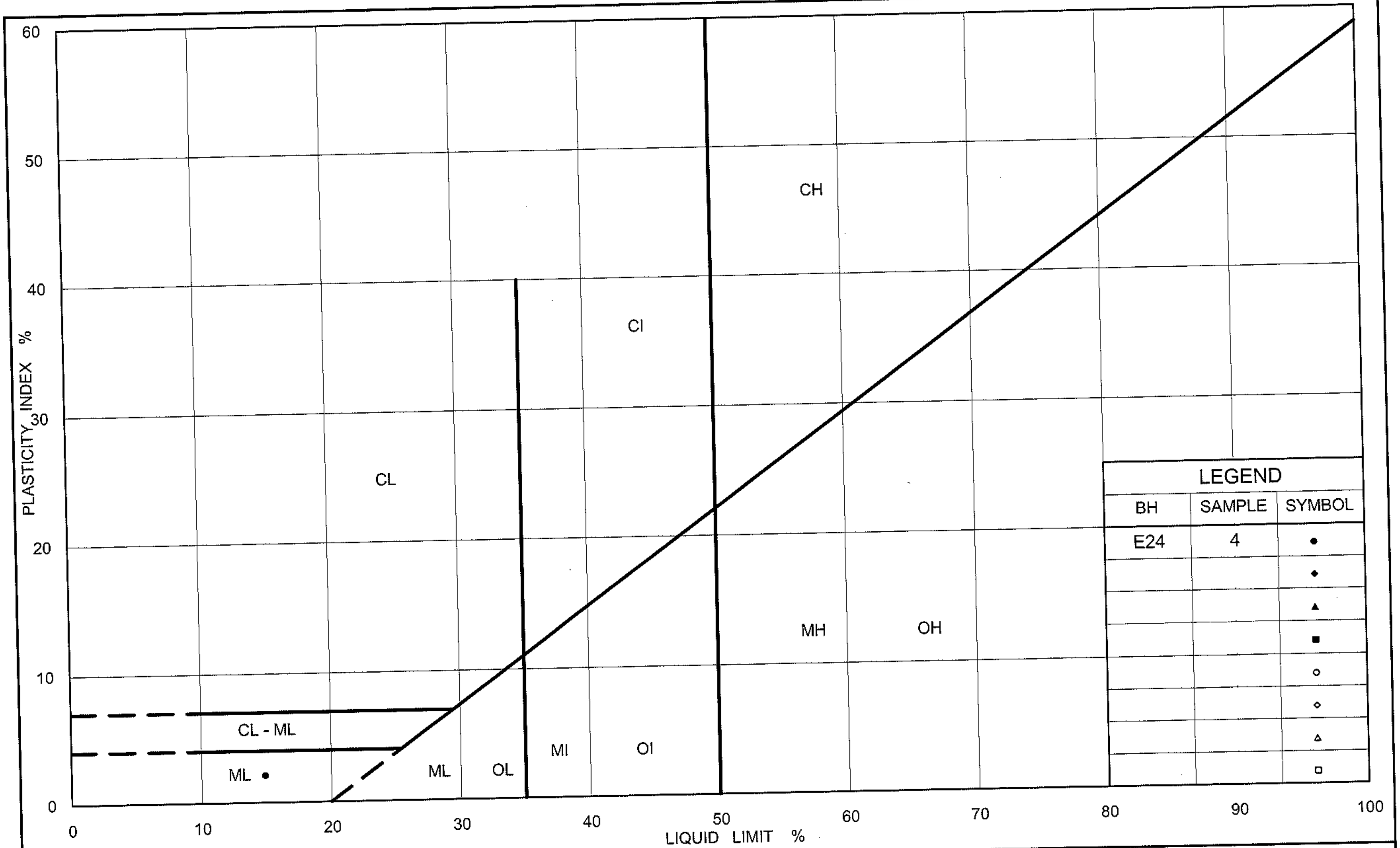
PLASTICITY CHART
Clay

Figure No. 23

Project No. 08-1111-0044

Checked By: KSL

Oct 75, FF-S-21



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PLASTICITY CHART
Silt

Figure No. 24

Project No. 08-1111-0044

Checked By:KSL

APPENDIX A

List of Abbreviations and Symbols
Record of Borehole Sheets
Record of Test Pits (Eastbound Toe of Slope)
Record of Test Pits (Westbound Toe of Slope)

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DO	Drive open
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample
DT	Dual Tube sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:
The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open Sampler for a distance of 300 mm (12 in.)
DD- Diamond Drilling

Dynamic Penetration Resistance; N_d:
The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive Uncased a 50 mm (2 in.) diameter, 60^0 cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

PH:	Sampler advanced by hydraulic pressure
PM:	Sampler advanced by manual pressure
WH:	Sampler advanced by static weight of hammer
WR:	Sampler advanced by weight of sampler and rod

Peizo-Cone Penetration Test (CPT):
An electronic cone penetrometer with a 60^0 conical tip and a projected end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded Electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Cohesionless Soils

Density Index (Relative Density)	N Blows/300 mm Or Blows/ft.
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

Consistency	C_u or S_u	
	Kpa	Psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	Over 200	Over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limited
w_l	liquid limit
C	consolidaiton (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	modified Proctor compaction test
SPC	standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane test (LV-laboratory vane test)
γ	unit weight

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

LIST OF SYMBOLS

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

I. GENERAL

π = 3.1416

ln x, natural logarithm of x

$\log_{10}$ x or log x, logarithm of x to base 10

g	Acceleration due to gravity
t	time
F	factor of safety
V	volume
W	weight

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\sigma'$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma''-u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1\sigma_2\sigma_3$	principal stresses (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1+\sigma_2+\sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight*)
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma'=\gamma-\gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R= p_s/p_w$) formerly (G_s)
e	void ratio
n	porosity
S	degree of saturation

* Density symbol is p. Unit weight symbol is γ where γ=pg(i.e. mass density x acceleration due to gravity)

(a) Index Properties (cont'd.)

w	water content
w_l	liquid limit
w_p	plastic limit
I_p	plasticity Index=(w_l-w_p)
w_s	shrinkage limit
I_L	liquidity index=$(w-w_p)/I_p$
I_c	consistency index=$(w_l-w)/I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index-$(e_{max}-e)/(e_{max}-e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (overconsolidated range)
C_s	swelling index
C_a	coefficient of secondary consolidation
m_v	coefficient of volume change
c_v	coefficient of consolidation
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation pressure
OCR	Overconsolidation ratio=σ'_p/σ'_{vo}

(d) Shear Strength

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

Notes: 1. $\tau=c'\sigma' \tan |'$

2. Shear strength=(Compressive strength)/2

LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

WEATHERING STATE

Fresh: no visible sign of weathering

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

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

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

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

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

BEDDING THICKNESS

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

JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	>3 m
Wide	1 – 3 m
Moderately close	0.3 – 1 m
Close	50 – 300 mm
Very close	<50 mm

GRAIN SIZE

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

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

O:\ Templates\Rock Description
Terminology

CORE CONDITION

Total Core Recovery

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

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

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

Description and Notes

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

Abbreviations

B –	Bedding	Ca-	Calcite
FO-	Foliation/Schistosity	P-	Polished
CL -	Cleavage	S-	Slickensided
SH -	Shear Plane/Zone	SM-	Smooth
VN-	Vein	R-	Ridged/Rough
F -	Fault	ST-	Stepped
CO-	Contact	PL-	Planar
J -	Joint	FL-	Flexured
FR-	Fracture	UE-	Uneven
MF -	Mechanical	W-	Wavy
A-	Angular	C-	Curved
BP-	Bedding Plane	H-	Hackly
BL-	Blast Induced	SL-	Sludge Coated
∥	Parallel To	TCA-	To Core Axis
	Perpendicular To	STR-	Stress Induced

Golder Associates

RECORD OF BOREHOLE No E2

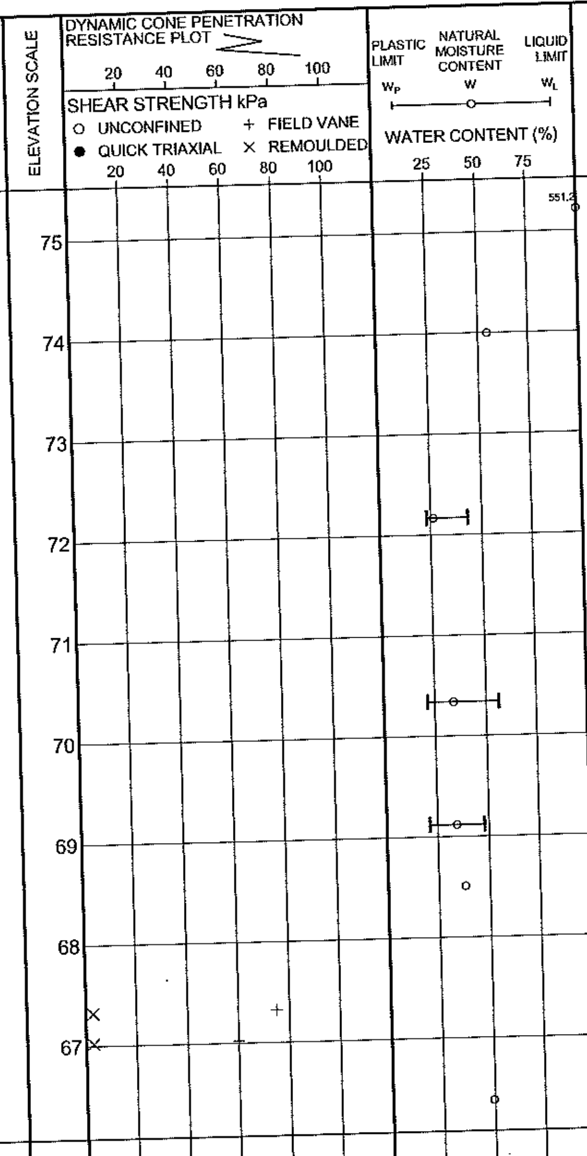
1 OF 1 **METRIC**

PROJECT 08-1111-0044	LOCATION N 4904279.4 ;E 307296.6	ORIGINATED BY	JEB
G.W.P. 78-99-01	BOREHOLE TYPE Portable Equipment, Continuous Sampling	COMPILED BY	AT
DIST HWY 401	DATE February 26, 2009	CHECKED BY	KSL
DATUM Geodetic			

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
75.5	ICE SURFACE					
0.0 75.2	ICE	1	SS	3		
	PEAT (Fibrous), trace sand, trace clay, containing rootlets Soft to firm Black Wet					
74.9 0.6	Organic CLAYEY SILT, trace sand, containing rootlets Firm Wet	2	SS	5	∇	OC = 9.2%
		3	SS	7		
73.7 1.8	SILTY CLAY, trace sand, containing peat and rootlets Very stiff to hard Grey Moist	4	SS	15		
		5	SS	65		
	Brown between depths of 3.1 m and 3.7 m	6	SS	171		
		7	SS	114		
		8	SS	60		
70.6 4.9	Limestone layer at a depth of 4.7 m CLAY, trace sand Hard Grey Moist	9	SS	31		
70.0 5.5	SILTY CLAY, trace gravel, trace sand, containing silt layers Very stiff Grey Moist	10	SS	17		
		11	SS	17		0 1 35 64
		12	SS	14		
		13	SS	9		
	Sand seams encountered at a depth of 9.1 m	14	SS	14		
66.1 9.5	END OF BOREHOLE					

NOTE:

1. Water level in open borehole at a depth of 0.9 m below ice surface (Elev. 74.6 m) upon completion of drilling.



UNIT WEIGHT γ kN/m³

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

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

Golder Associates

RECORD OF BOREHOLE No E3

1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ______ HWY 401
DATUM Geodetic

LOCATION N 4904291.4 ;E 307330.3
BOREHOLE TYPE Portable Equipment, Continuous Sampling
DATE March 3, 2009

ORIGINATED BY JEB
COMPILED BY AT
CHECKED BY KSL

SOIL PROFILE

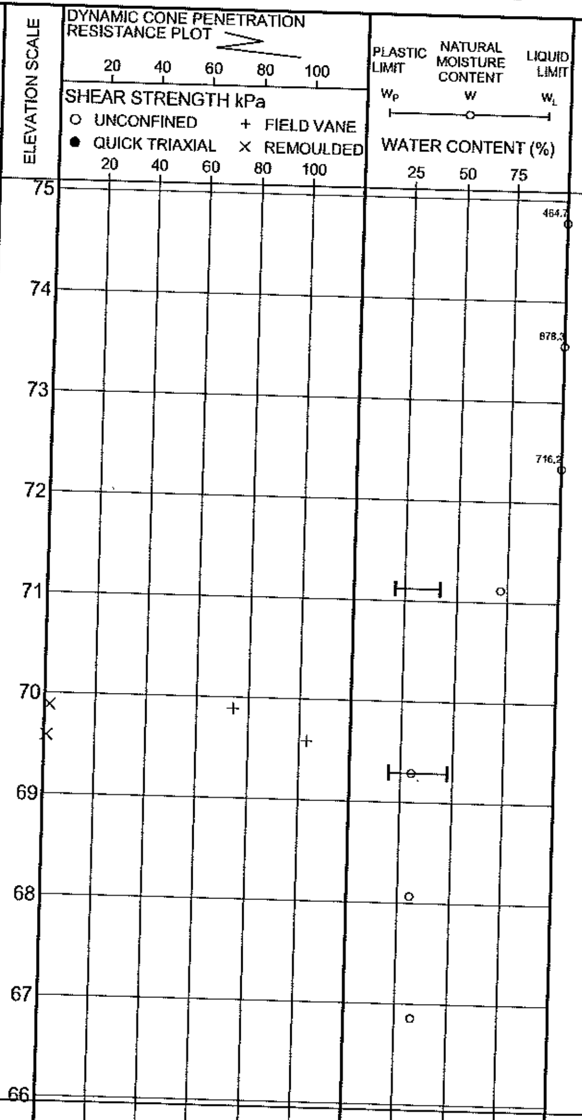
ELEV / DEPTH	DESCRIPTION
75.1	ICE SURFACE
0.0	ICE, containing organics
0.2	PEAT (Fibrous) Very soft to hard Dark brown to black Wet Becoming soft below a depth of 0.6 m
71.4	Trace clay at a depth of 3.4 m
3.7	SILTY CLAY, trace sand, containing shell fragments between depths of 3.7 m and 4.0 m Very soft to soft Grey
70.2	Moist
4.9	SILTY CLAY, trace sand Hard Grey Moist
65.9	
9.1	END OF BOREHOLE

NOTE:

1. Water level in open borehole at a depth of 0.3 m below ice surface (Elev. 74.8 m) upon completion of drilling.

SAMPLES

NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS
1	SS	33	∇
2	SS	3	
3	SS	2	
4	SS	3	
5	SS	4	
6	SS	2	
7	SS	WH	
8	SS	4	
9	SS	38	
10	SS	59	
11	SS	55	
12	SS	51	
13	SS	48	
14	SS	50	



UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

+ ^{3}, × ^{3}: Numbers refer to Sensitivity

○ $^{3\%}$ STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No E4

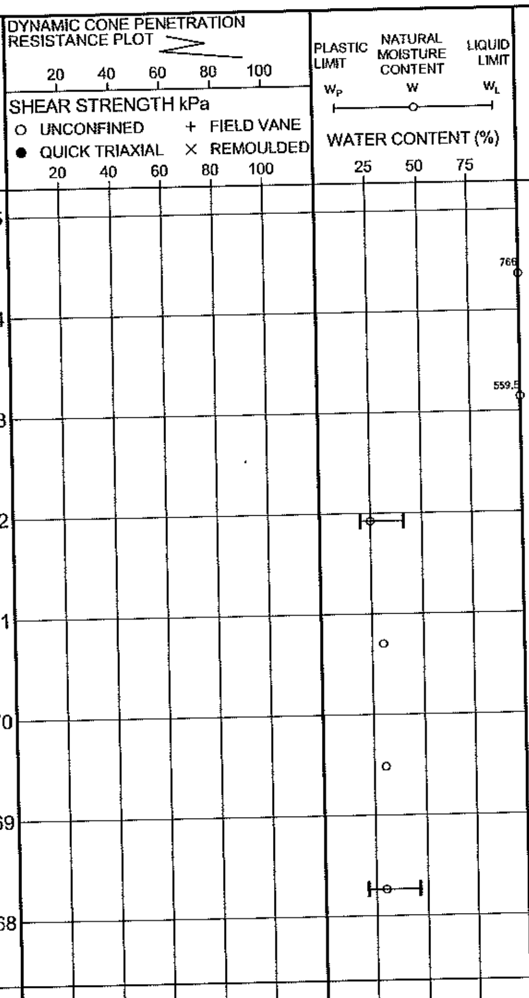
1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST HWY 401
DATUM Geodetic

LOCATION N 4904307.2 ;E 307374.7
BOREHOLE TYPE Portable Equipment, Continuous Sampling
DATE March 3 and 4, 2009

ORIGINATED BY JEB
COMPILED BY AT
CHECKED BY KSL

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES		
75.3	ICE SURFACE					
0.0 75.0	ICE, containing organics	1	SS	60	∇	75
0.3	PEAT (Fibrous) Hard Black Wet Becoming firm below a depth of 0.6 m	2	SS	5		
		3	SS	5		74
	Containing wood fragments between depths of 2.1 m and 2.7 m	4	SS	7		73
72.5		5	SS	6		
2.7	SILTY CLAY, trace gravel, trace sand Very stiff to hard Grey Moist	6	SS	38		72
		7	SS	32		71
		8	SS	20		
		9	SS	16		70
	Containing clayey silt seams between depths of 5.5 m and 6.1 m	10	SS	21		
		11	SS	20		69
		12	SS	27		68
		13	SS	26		
67.4 7.9	END OF BOREHOLE					



UNIT WEIGHT γ kN/m³

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

NOTE:

1. Water level in open borehole at a depth of 0.3 m below ice surface (Elev. 75.0 m) upon completion of drilling.

+ ^{3}, × ^{3}: Numbers refer to Sensitivity

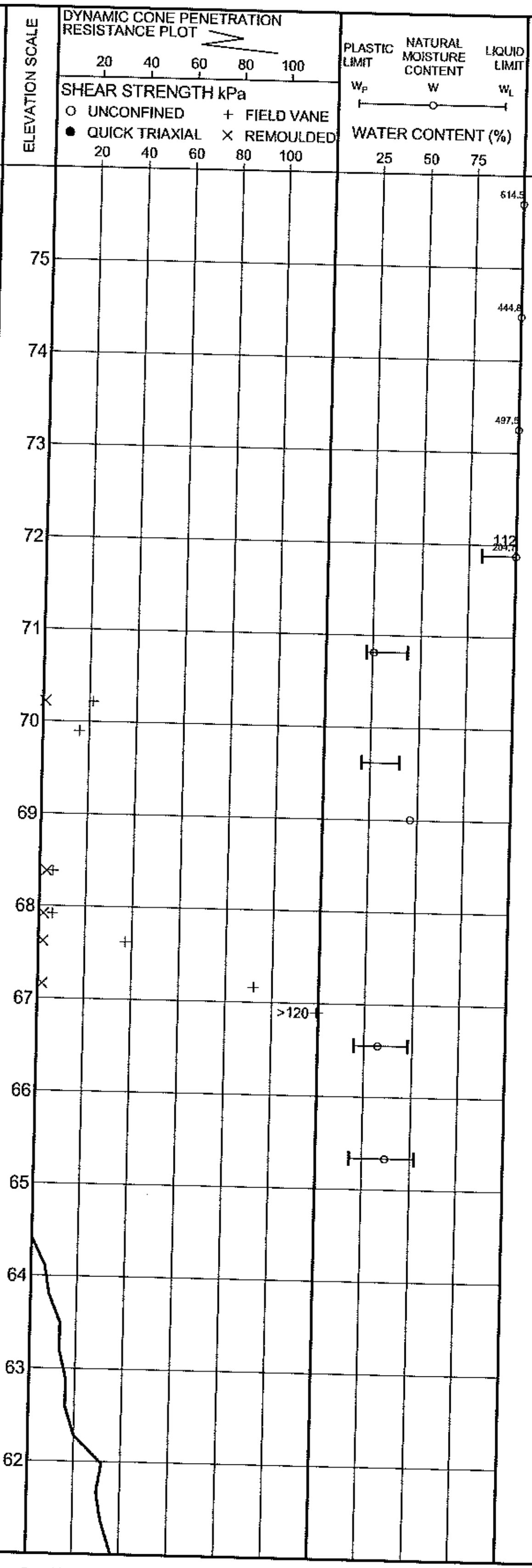
○ $^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No E5

1 OF 2 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ______ HWY 401
DATUM Geodetic
LOCATION N 4904328.0 ;E 307424.2
BOREHOLE TYPE Portable Equipment, Continuous Sampling, BW & NW Casing, Wash Boring and DCPT
DATE March 4 and 5, 2009
ORIGINATED BY DM
COMPILED BY AT
CHECKED BY KSL

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
76.0	GROUND SURFACE						
0.0	Sand, containing roots (FILL) Soft Black Wet	1	SS	3	∇		
75.4 0.6	PEAT (Fibrous), containing decomposed wood fragments Soft to firm Black Moist	2	SS	2			
		3	SS	3			
74.2 1.8	Silty PEAT (Fibrous) Firm Dark brown Wet	4	SS	5			
		5	SS	7			
		6	SS	6			
72.0 4.0	Organic CLAYEY SILT, some sand, containing shell fragments Very soft to firm Brown Wet	7	SS	7			OC = 14.6%
		8	SS	5			
70.8 5.2	SILTY CLAY, trace sand Soft to hard Grey Wet	9	SS	WH			0 1 41 58
		10	TO	PM		16.8	
		11	TO	PM			
66.9 9.1	CLAY, some silt, trace sand Stiff to very stiff Grey Moist	13	SS	20			0 0 29 71
		14	SS	18			
		15	SS	15			
		16	SS	11			
64.4 11.6	END OF BOREHOLE						



Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No E5

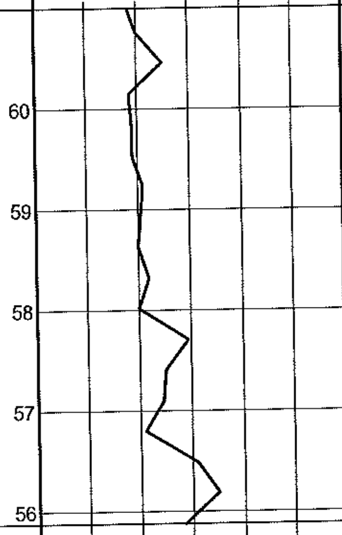
2 OF 2 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ______ HWY 401
DATUM Geodetic

LOCATION N 4904328.0 ;E 307424.2
BOREHOLE TYPE Portable Equipment, Continuous Sampling, BW & NW Casing, Wash Boring and DCPT
DATE March 4 and 5, 2009

ORIGINATED BY DM
COMPILED BY AT
CHECKED BY KSL

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ 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	GROUND WATER CONDITIONS	ELEVATION SCALE				
	— CONTINUED FROM PREVIOUS PAGE —										
55.9 20.1	END OF DCPT NOTE: 1. Water level in open borehole at a depth of 0.4 m below ground surface (Elev. 75.6 m) upon completion of drilling.										



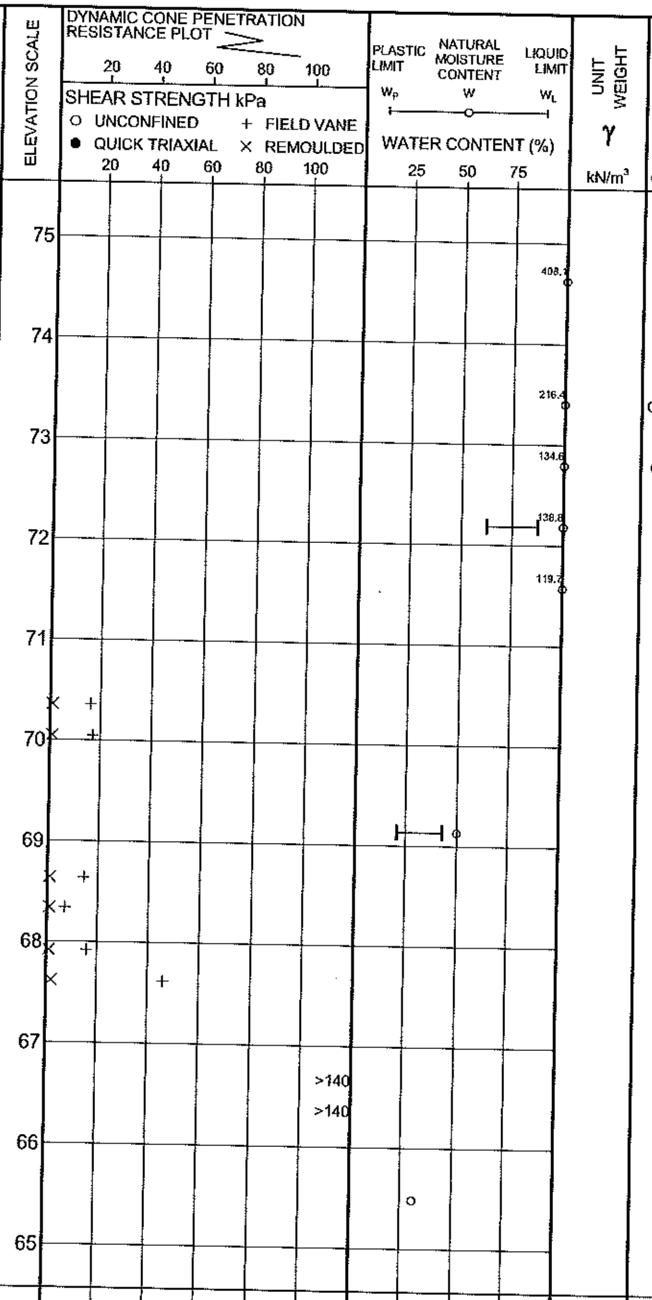
+^{3}, ×^{3}: Numbers refer to Sensitivity ○$^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No E6

1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST
HWY 401
DATUM Geodetic
LOCATION N 4904341.7 ;E 307466.8
BOREHOLE TYPE Portable Equipment, Continuous Sampling, BW Casing, Wash Boring
DATE March 2, 2009
ORIGINATED BY DM
COMPILED BY AT
CHECKED BY KSL

SOIL PROFILE ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
75.5	GROUND SURFACE					
0.0	PEAT (Fibrous), containing rootlets Very soft to soft Black Wet	1	SS	2	∇	
74.9 0.6	Silty PEAT (Fibrous) Very soft to soft Dark brown Wet	2	SS	1		
		3	SS	3		
73.4 2.1	Clayey silt, some sand, containing white shell fragments (MARL) Soft Light brown to brown Wet	4	SS	2		OC = 16.9%
		5	SS	3		OC = 7.3% 0 17 73 10
		6	SS	2		
		7	SS	2		
71.3 4.3	SILTY CLAY Very soft to soft Grey Wet	8	SS	WH		
		9	TO	PM		
67.6 7.9	CLAY, some silt, trace gravel, trace sand Stiff to hard Grey Wet	10	SS	12		
		11	SS	35		
		12	SS	39		
		13	SS	28		
64.6 11.0	END OF BOREHOLE					



NOTES:

1. Water level in open borehole at a depth of 0.4 m below ground surface (Elev. 75.1 m) upon completion of drilling.

2. An additional borehole was drilled 0.8 m South of Borehole E6A to obtain Shelby tube samples between depth of 2.4 m and 3.6 m. See Record of Borehole E6A for details.

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

Golder Associates

RECORD OF BOREHOLE No E6A

1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST HWY 401
DATUM Geodetic

LOCATION N 4904340.9 ;E 307467.0
BOREHOLE TYPE Portable Equipment, 51 mm Diameter Continuous Sampling, BW & NW Casing, Wash
DATE March 5, 2009

ORIGINATED BY DM
COMPILED BY AT
CHECKED BY KSL

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ 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	GROUND WATER CONDITIONS	ELEVATION SCALE				
75.6	GROUND SURFACE										
0.0	See Record of Borehole E6 for subsurface conditions						75 74				
73.2 2.4	Organic CLAYEY SILT, containing shell fragments, trace sand Soft Light brown Wet		1	TO	PM		73				
			2	TO	PM				138.8		OC = 5.7%
72.0 3.6	END OF BOREHOLE						72				

+³, ×³: Numbers refer to Sensitivity

○³% STRAIN AT FAILURE

Golder Associates

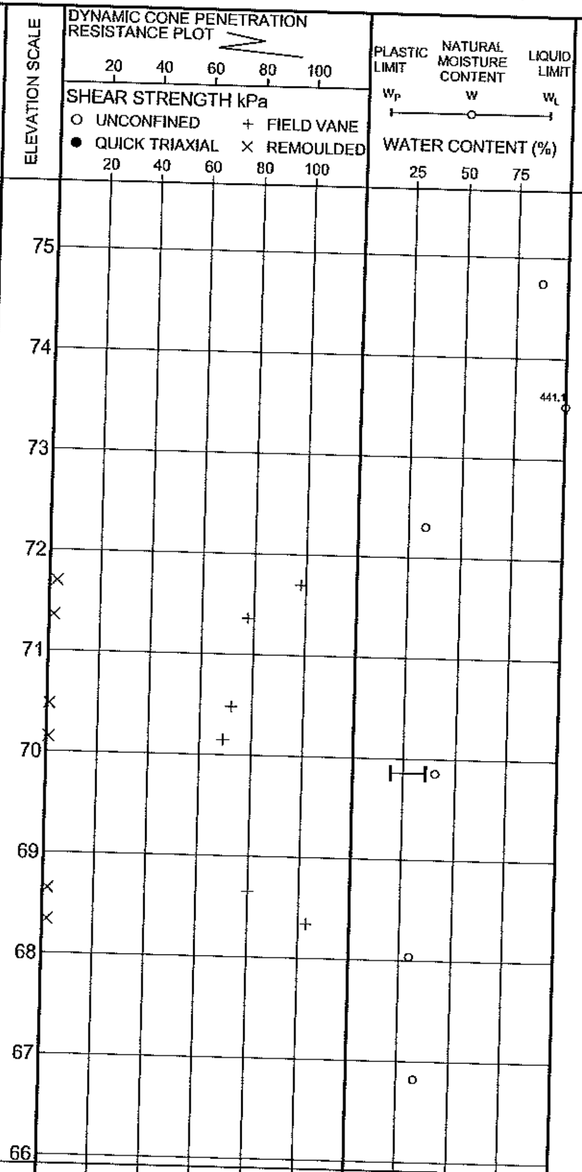
Foundation Design

RECORD OF BOREHOLE No E7

1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ______ HWY 401
DATUM Geodetic
LOCATION N 4904360.8 ;E 307513.4
BOREHOLE TYPE Portable Equipment, Continuous Sampling, BW Casing, Wash Boring
DATE February 26 and 27 2009
ORIGINATED BY DM
COMPILED BY AT
CHECKED BY KSL

ELEV / DEPTH	SOIL PROFILE — DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS
75.7	GROUND SURFACE					
75.4 0.0	Silty clay (FILL) Firm Brown to grey Moist	▨	1	SS	4	∇
0.3	PEAT (Fibrous), containing decomposed wood fragments Very soft to firm Wet		2	SS	1	
			3	SS	3	
			4	SS	4	
			5	SS	4	
72.7 3.0	SILTY CLAY, trace sand, containing rootlets to a depth of 4.3 m Soft to very stiff Grey Moist Becoming wet at a depth of 4.3 m		6	SS	7	
			7	SS	5	
			8	SS	2	
			9	TO	PM	
			10	SS	19	
			11	SS	27	
			12	SS	25	
			13	SS	23	
65.9 9.8	END OF BOREHOLE					



UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL

NOTE:

1. Water level in open borehole at a depth of 0.4 m below ground surface (Elev. 75.3 m) upon completion of drilling.

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

Golder Associates

RECORD OF BOREHOLE No E8

1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST HWY 401
DATUM Geodetic

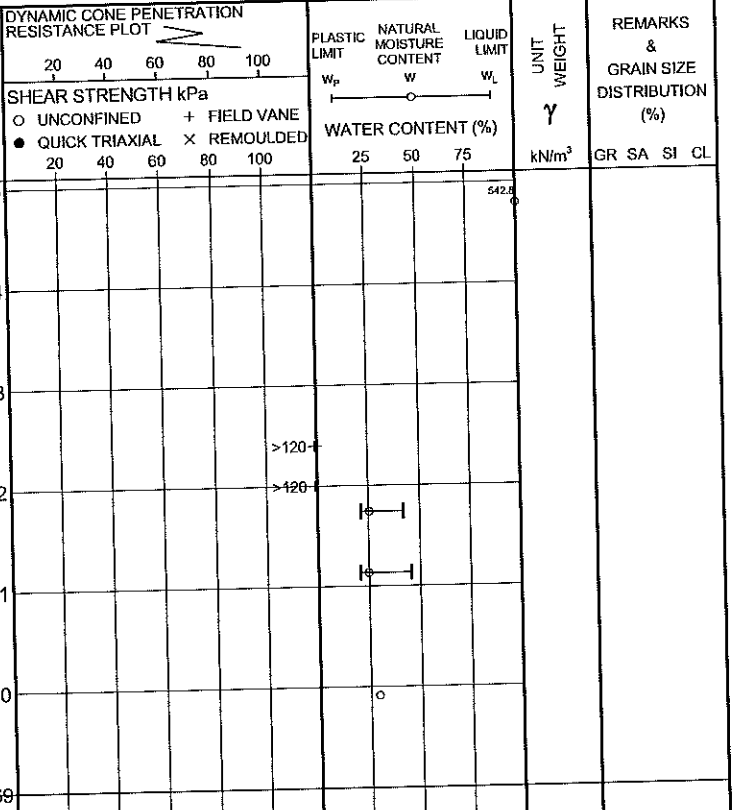
LOCATION N 4904378.9 ;E 307564.6
BOREHOLE TYPE Portable Equipment, Continuous Sampling
DATE February 26, 2009

ORIGINATED BY DM
COMPILED BY AT
CHECKED BY KSL

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
75.1	GROUND SURFACE					∇	75
0.0	PEAT (Fibrous), containing rootlets Very soft to soft Black to dark brown Moist		1	SS	4		
			2	SS	WH		74
			3	SS	WH		
73.0 2.1	SILTY CLAY, trace sand Firm to hard Greenish grey Moist		4	SS	4		73
							72
			5	SS	47		
			6	SS	43		71
			7	SS	21		
70.2 4.9	CLAYEY SILT, trace sand Very stiff Grey Wet		8	SS	20		70
			9	SS	26		
69.0 6.1	END OF BOREHOLE						69

NOTE:

1. Water level in open borehole at a depth of 0.1 m below ground surface (Elev. 75.0 m) upon completion of drilling.



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

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

RECORD OF BOREHOLE No E9

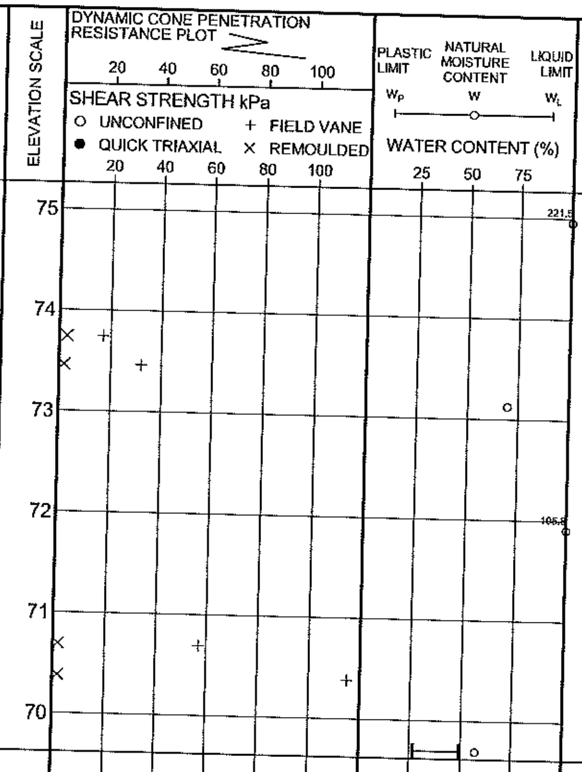
1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ______ HWY 401
DATUM Geodetic
LOCATION N 4904389.8 ;E 307612.9
BOREHOLE TYPE Portable Equipment, Continuous Sampling
DATE February 25, 2009
ORIGINATED BY KL
COMPILED BY AT
CHECKED BY KSL

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	 TYPE	 "N" VALUES	GROUND WATER CONDITIONS
75.3	GROUND SURFACE					
0.0	PEAT (Fibrous), containing rootlets Very stiff Brown to black		1	SS	17	∇
74.5 0.8	Organic SILTY CLAY, trace gravel, trace sand, containing wood fragments Soft to stiff Grey		2	SS	1	
			3	SS	4	
			4	SS	6	
			5	SS	7	
	Containing cobbles below a depth of 4.1 m		6	SS	8	
			7	SS	43	
69.6 5.6	END OF BOREHOLE SPOON REFUSAL		8	SS	00/0.15	

NOTE:

1. Water level in open borehole at a depth of 0.4 m below ground surface (Elev. 74.9 m) upon completion of drilling.



UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	OC = 9.9%

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

Golder Associates

RECORD OF BOREHOLE No E10

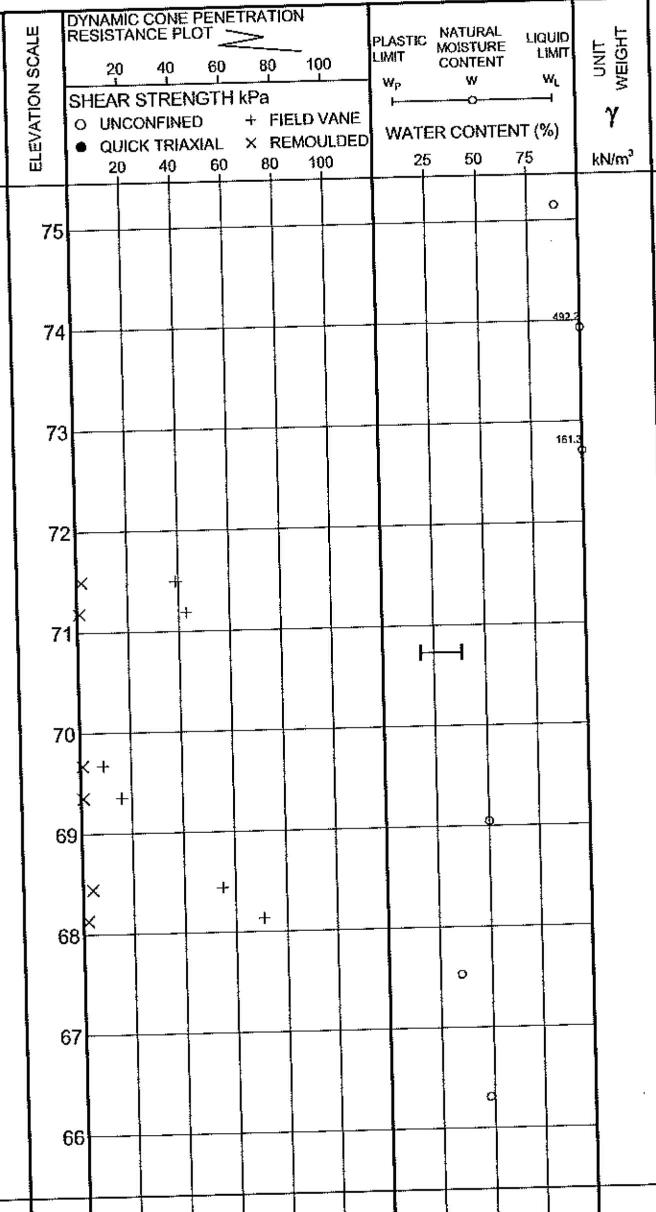
1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ____ HWY 401
DATUM Geodetic

LOCATION N 4904408.0 ;E 307654.3
BOREHOLE TYPE Portable Equipment, Continuous Sampling, BW Casing, Wash Boring
DATE February 24 and 25, 2009

ORIGINATED BY DM
COMPILED BY AT
CHECKED BY KSL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		GR	SA	SI	CL
75.5	GROUND SURFACE									
0.0	PEAT (Fibrous), containing decomposed wood fragments Soft to firm Wet	▤	1	SS	8					
			2	SS	2	∇				
			3	SS	4					
			4	SS	4					
			5	SS	4					
72.5 3.0	SILTY CLAY, slightly organic Firm Greenish grey Wet	▨	6	SS	6					
71.0 4.4	SILTY CLAY, trace sand Very soft to stiff Grey Wet	▨	7	TO	PM		0	1	49	50
			8	SS	5					
			9	SS	11					
			10	SS	10					
			11	SS	9					
			12	SS	9					
65.4 10.1	END OF BOREHOLE									



NOTE:

1. Water level in open borehole at a depth of 0.9 m below ground surface (Elev. 74.6 m) upon completion of drilling.

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

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

RECORD OF BOREHOLE No E11

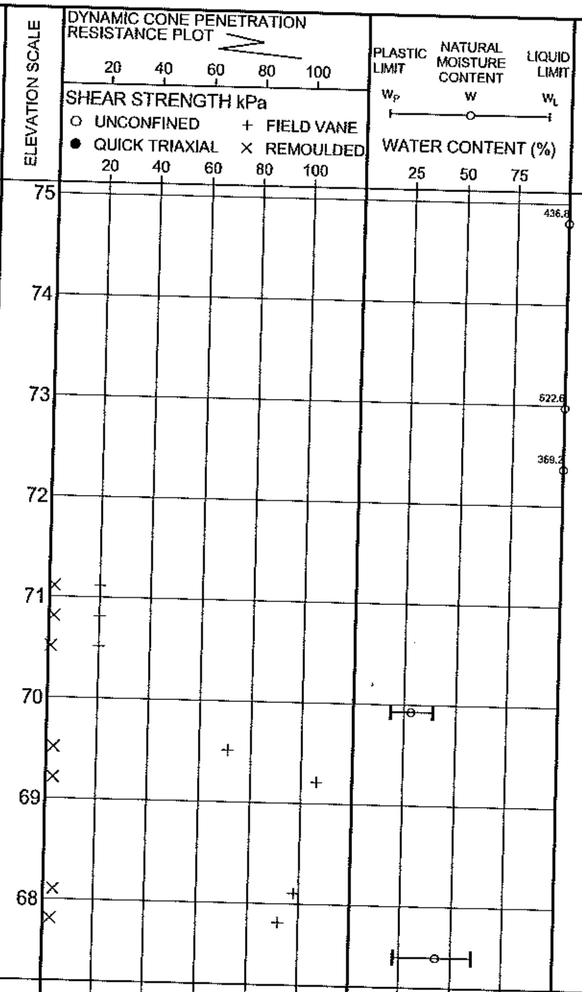
1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ______ HWY 401
DATUM Geodetic
LOCATION N 4904429.0 ;E 307709.2
BOREHOLE TYPE Portable Equipment, Continuous Sampling, BW Casing, Wash Boring
DATE February 23 and 24, 2009
ORIGINATED BY DM
COMPILED BY AT
CHECKED BY KSL

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS
75.1	GROUND SURFACE				
0.0	PEAT (Fibrous) Very soft to firm Black to dark brown Wet	1	SS	5	▽
		2	SS	WH	
		3	SS	2	
		4	SS	2	
		5	SS	3	
72.1 3.1	SILTY CLAY, trace sand Soft to very stiff Greenish grey Wet	6	SS	3	
		7	TO	PM	
69.0 6.1	CLAY, some silt, trace sand Stiff Grey Wet	8	SS	8	
		9	SS	9	
67.2 7.9	END OF BOREHOLE				

NOTE:

1. Water level in open borehole at a depth of 0.3 m below ground surface (Elev. 74.8 m) upon completion of drilling.



UNIT WEIGHT γ kN/m³

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

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

Golder Associates

RECORD OF BOREHOLE No E12

1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ______ HWY 401
DATUM Geodetic

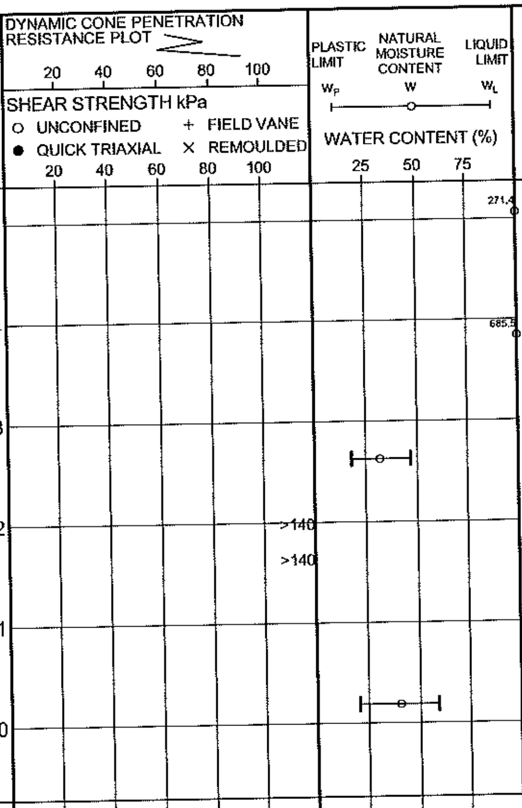
LOCATION N 4904443.7 ;E 307753.6
BOREHOLE TYPE Portable Equipment, Continuous Sampling
DATE February 23, 2009

ORIGINATED BY DM
COMPILED BY AT
CHECKED BY KSL

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
75.4	GROUND SURFACE						
0.0	PEAT (Fibrous) Very soft to soft Dark brown to black Wet		1	SS	21	∇	75
			2	SS	1		
			3	SS	3		74
			4	SS	3		73
72.9 2.4	SILTY CLAY, trace sand Very stiff to hard Greenish grey Moist		5	SS	7		
							72
71.7 3.7	CLAY, some silt, trace sand Stiff to very stiff Grey Moist		6	SS	28		
			7	SS	21		71
			8	SS	18		70
			9	SS	10		
69.3 6.1	END OF BOREHOLE						

NOTE:

1. Water level in open borehole at a depth of 0.3 m below ground surface (Elev. 75.1 m) upon completion of drilling.



UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL

+³, ×³: Numbers refer to Sensitivity

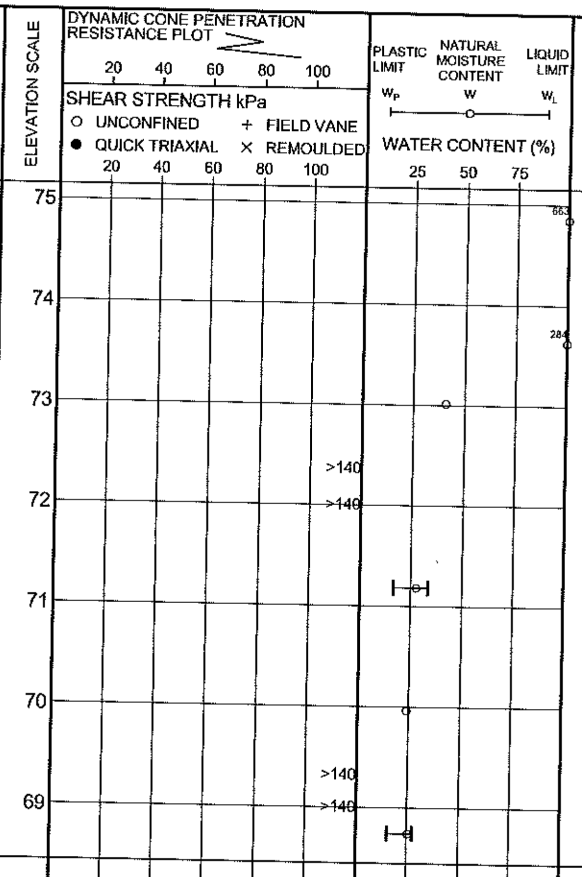
○³% STRAIN AT FAILURE

RECORD OF BOREHOLE No E13

1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ____ HWY 401
DATUM Geodetic
LOCATION N 4904461.3 ;E 307804.2
BOREHOLE TYPE Portable Equipment, Continuous Sampling
DATE February 19, 2009
ORIGINATED BY DM
COMPILED BY AT
CHECKED BY KSL

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
75.2	GROUND SURFACE										
0.0	PEAT (Fibrous), containing rootlets Very soft Black to dark brown Wet		1	SS	2	∇					
			2	SS	WH						
			3	SS	1						
73.3 1.8	Organic SILTY CLAY, trace sand Firm to hard Greenish grey Wet		4	SS	6			OC = 5.7%			
72.1 3.1	CLAYEY SILT, trace sand Stiff to very stiff Brown to grey Moist		5	SS	24						
			6	SS	25			0	4	61	35
			7	SS	10						
			8	SS	8						
			9	SS	13			0	1	65	34
68.5 6.7	END OF BOREHOLE NOTE: 1. Water level in open borehole at a depth of 0.2 m below ground surface (Elev. 75.0 m) upon completion of drilling.										



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

Golder Associates

RECORD OF BOREHOLE No E14

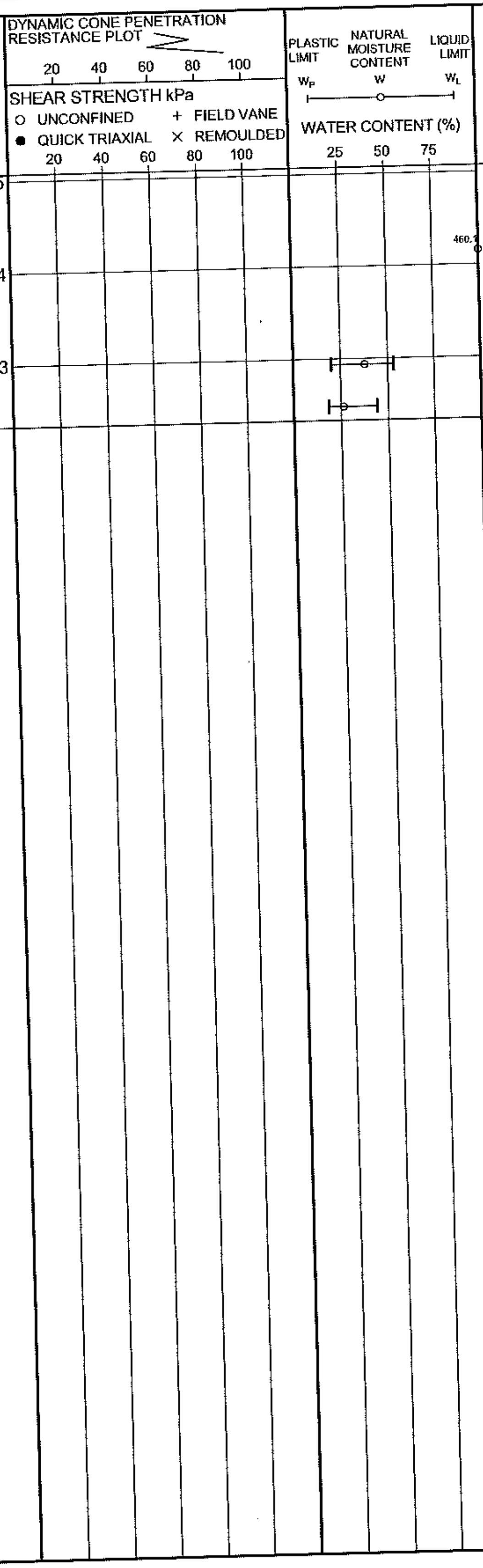
1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ____ HWY 401
DATUM Geodetic

LOCATION N 4904473.4 ;E 307848.6
BOREHOLE TYPE Portable Equipment, Continuous Sampling
DATE February 19, 2009

ORIGINATED BY JEB
COMPILED BY AT
CHECKED BY KSL

SOIL PROFILE			SAMPLES							REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³		GR	SA	SI	CL
75.1	GROUND SURFACE												
0.0	PEAT (Fibrous), trace sand Very soft Black Wet		1	SS	10	∇	75						
			2	SS	WH		74						
			3	SS	WH								
73.1 2.0	Organic CLAY, trace sand, containing rootlets Stiff Grey Moist		4	SS	11		73		OC = 7.7%				
72.6			5	SS	60/0.10					0	3	45	42
72.3 2.7	SILTY CLAY, trace sand Hard Black Moist												
	END OF BOREHOLE SPOON REFUSAL												



NOTES:

1. Water level in open borehole at a depth of 0.3 m below ground surface (Elev. 74.8 m) upon completion of drilling.

2. Two Dynamic Cone Penetration Test were advanced 2 m South and 2.8 m Southwest of Borehole E14, refusal encountered at a depth of 2.9 m and 3.1 m below ground surface (Elev. 72.2 m and 72.0 m) upon completion of drilling.

See Record of DCPT E14C1 and E14C2 for further details.

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

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

RECORD OF PENETRATION TEST No E14C1

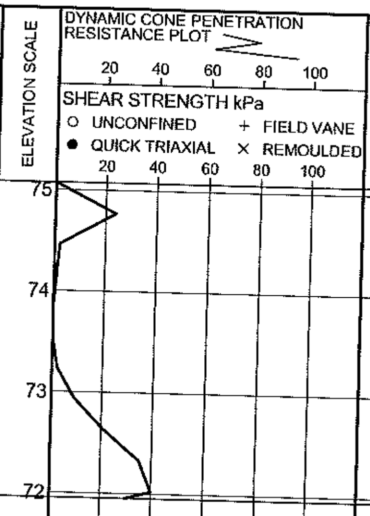
1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ______ HWY 401
DATUM Geodetic

LOCATION N 4904471.5 ;E 307849.2
BOREHOLE TYPE Portable Equipment, Dynamic Cone Penetration Test
DATE February 23, 2009

ORIGINATED BY JEB/DM
COMPILED BY AT
CHECKED BY KSL

SOIL PROFILE			SAMPLES									
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 ○ 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
75.1	GROUND SURFACE										
0.0	Start of Dynamic Cone Penetration Test (DCPT)						75				
							74				
							73				
72.0 3.1	END OF DCPT Refusal to further penetration (30 blows/0.1 m) NOTE: 1. The Dynamic Cone Penetration Test is located 2 m South of Borehole E14.						72				



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

Golder Associates

RECORD OF PENETRATION TEST No E14C2

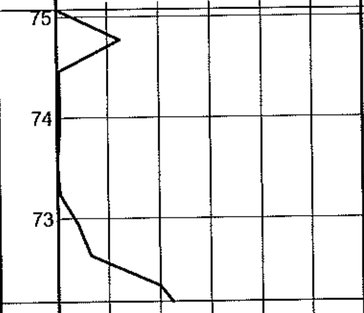
1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ______ HWY 401
DATUM Geodetic

LOCATION N 4904470.9 ;E 307847.3
BOREHOLE TYPE Portable Equipment, Dynamic Cone Penetration Test
DATE February 23, 2009

ORIGINATED BY JEB/DM
COMPILED BY AT
CHECKED BY KSL

SOIL PROFILE			SAMPLES									
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 ○ 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
75.1	GROUND SURFACE										
0.0	Start of Dynamic Cone Penetration Test (DCPT)						75 74 73				
72.2 2.9	END OF DCPT Refusal to further penetration (45 blows/0.15 m) NOTE: 1. The Dynamic Cone Penetration Test is located 2.8 m Southwest of Borehole E14.										



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

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

PROJECT 08-1111-0044

RECORD OF BOREHOLE No E15

1 OF 1 **METRIC**

G.W.P. 78-99-01

LOCATION N 4904490.0 ;E 307901.8

ORIGINATED BY JEB

DIST ______ HWY 401

BOREHOLE TYPE Portable Equipment, Continuous Sampling

COMPILED BY AT

DATUM Geodetic

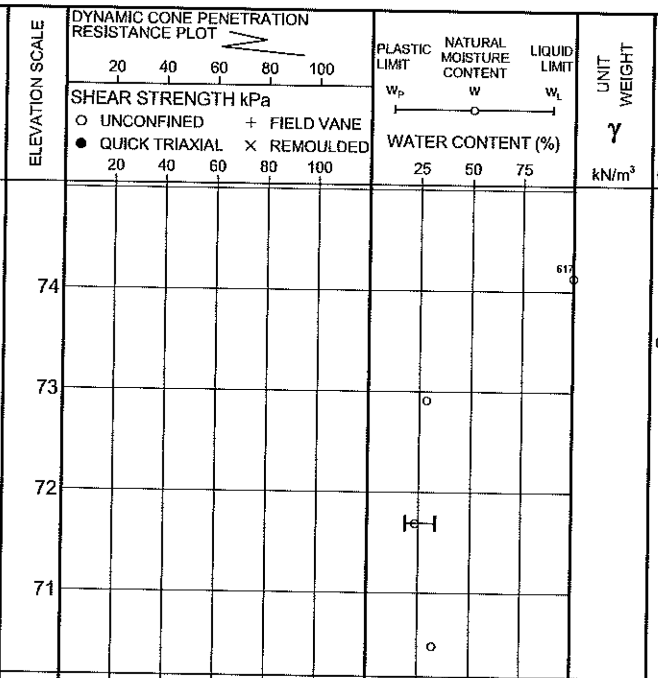
DATE February 23, 2009

CHECKED BY KSL

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES		GR	SA	SI	CL
75.0	ICE SURFACE								
0.0	ICE								
0.2	PEAT (Fibrous), trace sand Very soft Wet	1	SS	22	∇				
		2	SS	WH					
73.8 1.2	Silty PEAT, trace sand Soft Black Wet	3	SS	2		OC = 24.8%			
73.4 1.7	CLAYEY SILT, trace sand, containing rootlets and slightly organic to a depth of 1.8 m Very stiff to hard Grey to brown Moist	4	SS	17					
		5	SS	70					
		6	SS	74		0	2	63	35
		7	SS	52					
70.2		8	SS	33					
4.9	END OF BOREHOLE								

NOTE:

1. Water level in open borehole at a depth of 0.3 m below ice surface (Elev. 79.7 m) upon completion of drilling.



+³, ×³: Numbers refer to Sensitivity

○³% STRAIN AT FAILURE

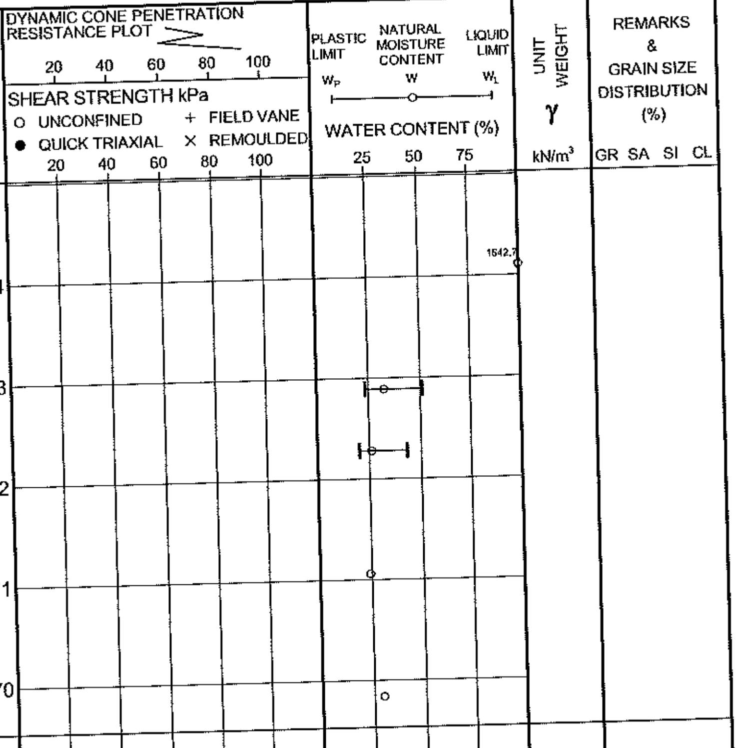
Golder Associates

RECORD OF BOREHOLE No E16

1 OF 1 **METRIC**

PROJECT 08-1111-0044	LOCATION N 4904505.2 ;E 307944.3	ORIGINATED BY JEB
G.W.P. 78-99-01	BOREHOLE TYPE Portable Equipment, Continuous Sampling	COMPILED BY AT
DIST ___ HWY 401	DATE February 23 and 24, 2009	CHECKED BY KSL
DATUM Geodetic		

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES		
75.0	GROUND SURFACE					
0.0	PEAT (Fibrous), trace sand Very soft Black Wet	1	SS	1	∇	
		2	SS	WH		74
		3	SS	WH		
73.2 1.8	CLAY, some silt, trace sand, containing rootlets Firm Grey Moist	4	SS	6		73
72.6 2.4	SILTY CLAY, some sand Very stiff to hard Grey to brown Moist	5	SS	33		
		6	SS	100/0.20		72
		7	SS	60		71
		8	SS	32		
		9	SS	21		70
69.5 5.5	END OF BOREHOLE					



NOTE:

1. Water level in open borehole at a depth of 0.3 m below ground surface (Elev. 74.7 m) upon completion of drilling.

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

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

PROJECT 08-1111-0044

RECORD OF BOREHOLE No E17

1 OF 1 **METRIC**

G.W.P. 78-99-01

LOCATION N 4904524.2 ;E 307992.5

ORIGINATED BY JEB

DIST ________ HWY 401

BOREHOLE TYPE Portable Equipment, Continuous Sampling

COMPILED BY AT

DATUM Geodetic

DATE February 24, 2009

CHECKED BY KSL

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
75.0	ICE SURFACE
0.0	ICE
0.2	PEAT (Fibrous), trace sand Very soft Black Wet
73.8 1.2	CLAY, some silt, trace sand Soft Grey Moist
73.2 1.8	SILTY CLAY/CLAYEY SILT, trace gravel, trace sand Firm to hard Grey Moist
68.3 6.9	Silty SAND, trace to some gravel, trace to some clay Very dense Grey Moist
	END OF BOREHOLE SPOON REFUSAL

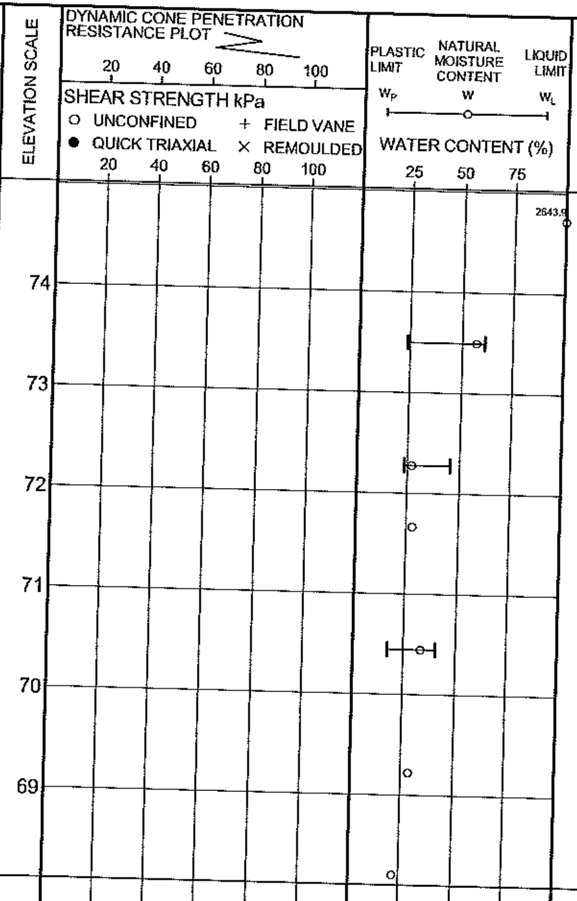
NOTE:

1. Water level in open borehole at ice surface (Elev. 75.0 m) upon completion of drilling.

SAMPLES

NUMBER	TYPE	"N" VALUES
1	SS	2
2	SS	WH
3	SS	3
4	SS	12
5	SS	36
6	SS	47
7	SS	21
8	SS	13
9	SS	6
10	SS	6
11	SS	8
12	SS	60/0.15

GROUND WATER CONDITIONS ▽



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Sample	GR	SA	SI	CL
6	0	4	54	42
12	11	43	30	16

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

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

Golder Associates

RECORD OF BOREHOLE No E18

1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ______ HWY 401
DATUM Geodetic

LOCATION N 4904527.8 ;E 308038.5
BOREHOLE TYPE Portable Equipment, Continuous Sampling
DATE February 24 and 25, 2009

ORIGINATED BY JEB
COMPILED BY AT
CHECKED BY KSL

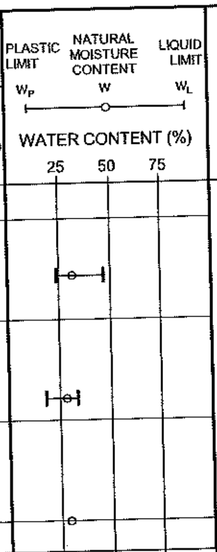
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	GR	SA	SI	CL
75.4	ICE SURFACE				∇					
0.0 0.1	ICE PEAT (Fibrous), trace sand Hard Black Wet	1	SS	39		75				
74.8 0.6	SILTY CLAY, trace to some sand Very stiff to hard Grey to brown Moist	2	SS	27						
		3	SS	43		74				
73.5 1.8	CLAYEY SILT, trace sand Hard Grey to brown Moist	4	SS	56		73	0	4	61	35
		5	SS	61						
		6	SS	51		72				
71.7 3.7	END OF BOREHOLE									

NOTE:

1. Water level in open borehole at a depth of 0.1 m below ice surface (Elev. 75.3 m) upon completion of drilling.

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

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

RECORD OF BOREHOLE No E19

1 OF 1 **METRIC**

PROJECT 08-1111-0044

G.W.P. 78-99-01

LOCATION N 4904744.9 ;E 308704.2

DIST ______ HWY 401

BOREHOLE TYPE Portable Equipment, Continuous Sampling

DATUM Geodetic

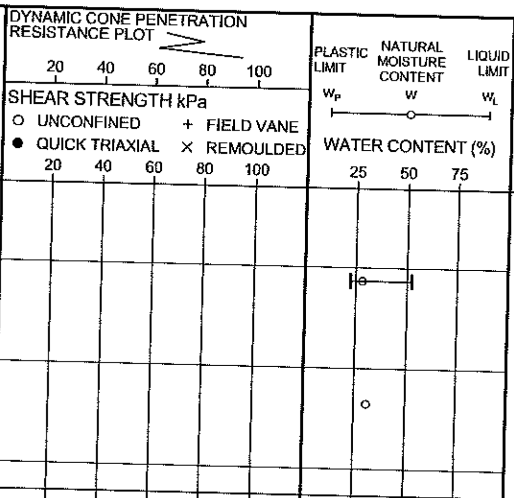
DATE February 25, 2009

ORIGINATED BY JEB

COMPILED BY AT

CHECKED BY KSL

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
75.8 0.0	ICE SURFACE ICE					▽			
	CLAY, trace gravel, trace sand Stiff to hard Brown Moist	▨	1	SS	15				
		▨	2	SS	38		75		
		▨	3	SS	120				
		▨	4	SS	165		74		
72.7 3.1		▨	5	SS	116		73		
	END OF BOREHOLE								



NOTE:

1. Water level in open borehole at ice surface (Elev. 75.8 m) upon completion of drilling.

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

Golder Associates

RECORD OF BOREHOLE No E20

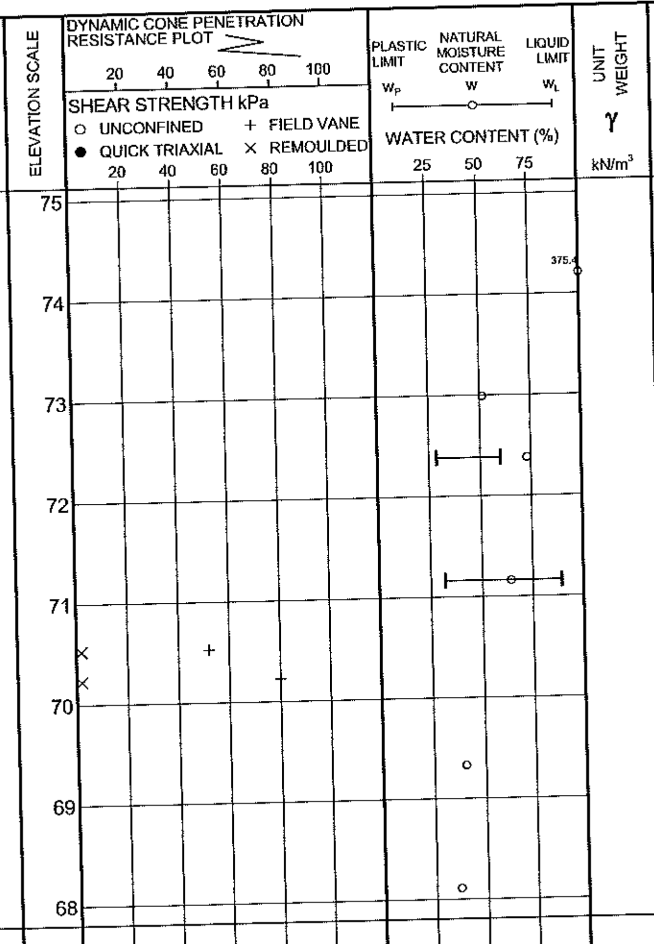
1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ______ HWY 401
DATUM Geodetic

LOCATION N 4904771.2 ;E 308753.9
BOREHOLE TYPE Portable Equipment, Continuous Sampling
DATE March 4, 2009

ORIGINATED BY JEB
COMPILED BY AT
CHECKED BY KSL

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
75.1	GROUND SURFACE				∇	
0.0	PEAT (Fibrous), trace sand Very soft to firm Black Wet	1	SS	5		
		2	SS	1		
		3	SS	2		
73.6 1.5	Organic CLAY, some silt, trace sand, containing rootlets Very soft to firm Grey to black Moist	4	SS	5		OC = 8.5%
		5	SS	3		
		6	SS	2		
71.5 3.7	Organic CLAY, trace sand Soft Grey/black Moist	7	SS	2		
70.9 4.3	SILTY CLAY, trace sand Grey Stiff to very stiff Moist					
		8	SS	16		
		9	SS	17		
		10	SS	18		
		11	SS	16		
67.8 7.3	END OF BOREHOLE NOTE: 1. Water level in open borehole at a depth of 0.1 m below ground surface (Elev. 75.0 m) upon completion of drilling.					



+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

RECORD OF BOREHOLE No E21

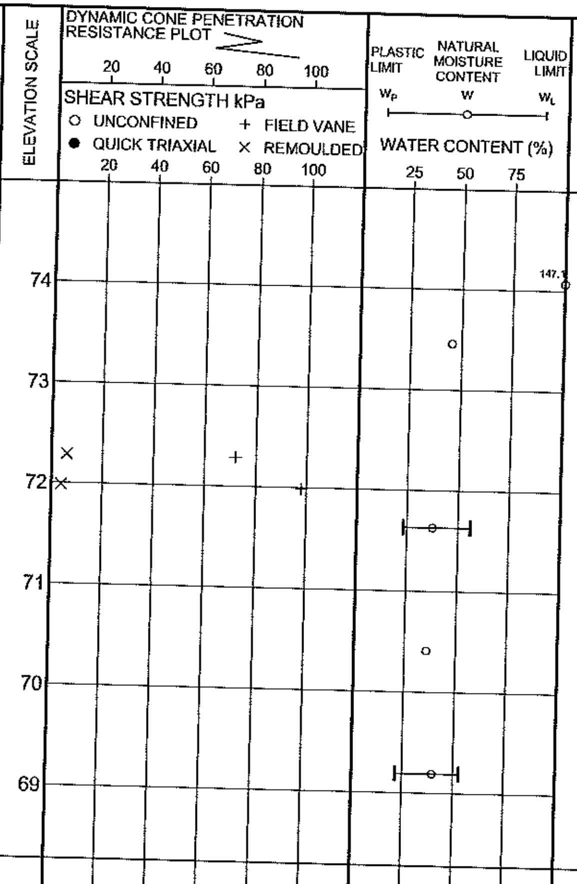
1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ____ HWY 401
DATUM Geodetic
LOCATION N 4904783.0 ;E 308797.6
BOREHOLE TYPE Portable Equipment, Continuous Sampling
DATE March 5, 2009
ORIGINATED BY JEB
COMPILED BY AT
CHECKED BY KSL

ELEV / DEPTH	SOIL PROFILE — DESCRIPTION	SAMPLES — NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
75.0	ICE SURFACE				
0.0	ICE				
74.7	WATER	1	SS	WH	
74.4 0.6	PEAT (Fibrous), trace sand Very soft Black Wet	2	SS	WH	
73.8 1.2	Organic CLAYEY SILT, containing rootlets Firm Grey Moist	3	SS	4	OC = 5.7%
		4	SS	7	
72.5 2.4	CLAY, some silt, trace gravel, trace sand Firm to stiff Grey to brown Moist				
		5	SS	14	
		6	SS	16	
		7	SS	18	
		8	SS	15	
		9	SS	13	
		10	SS	12	
68.3 6.7	END OF BOREHOLE				

NOTE:

1. Water level in open borehole at Ice surface (Elev. 75.0 m) upon completion of drilling.



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

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

Golder Associates

RECORD OF BOREHOLE No E22

1 OF 1 **METRIC**

PROJECT 08-1111-0044

G.W.P. 78-99-01

DIST ______ HWY 401

DATUM Geodetic

LOCATION N 4904801.4 ;E 308850.1

BOREHOLE TYPE Portable Equipment, Continuous Sampling

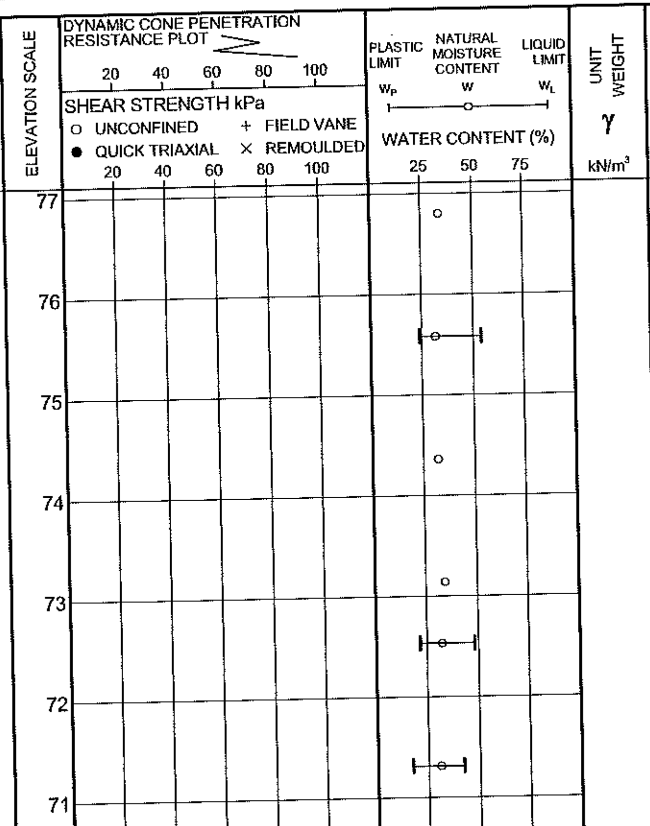
DATE March 5 and 6, 2009

ORIGINATED BY JEB/DM

COMPILED BY AT

CHECKED BY KSL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	▽	GR	SA	SI	CL
77.1	GROUND SURFACE									
0.0	CLAY, trace gravel, trace sand Very stiff to hard Brown Moist	▨	1	SS	26					
			2	SS	61					
			3	SS	88					
			4	SS	73					
74.7 2.4	SILTY CLAY, trace gravel, trace sand Very stiff to hard Brown and grey Moist	▨	5	SS	75					
			6	SS	73					
			7	SS	45					
			8	SS	19					
	Becoming grey at a depth of 4.9 m		9	SS	22					
			10	SS	16		0	5	45	50
			11	SS	70					
70.4 6.7	END OF BOREHOLE NOTE: 1. Water level in open borehole at ground surface (Elev. 77.1 m) upon completion of drilling.									



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

RECORD OF BOREHOLE No E23

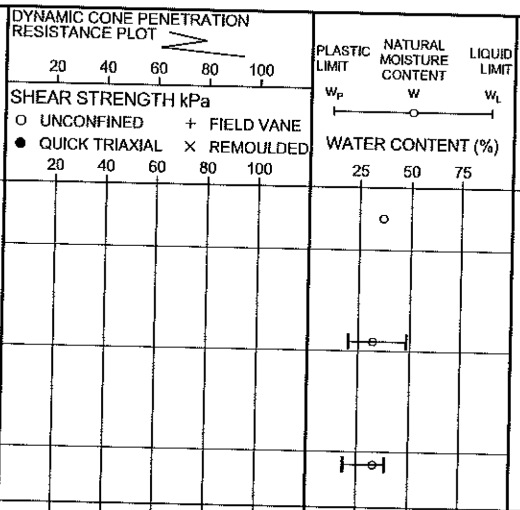
1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ______ HWY 401
DATUM Geodetic

LOCATION N 4904814.1 ;E 308903.4
BOREHOLE TYPE Portable Equipment, Continuous Sampling
DATE March 9, 2009

ORIGINATED BY DM
COMPILED BY AT
CHECKED BY KSL

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
76.6	GROUND SURFACE											
0.0	Clayey silt, trace sand, containing rootlets (FILL) Soft Brown Moist	▩	1	SS	3							
76.0 0.6	SILTY CLAY, trace sand Very stiff to hard Brown to grey Moist	▨	2	SS	15		76					
			3	SS	38		75		0	4	48	48
			4	SS	53	∇						
			5	SS	35		74					
73.4 3.2	END OF BOREHOLE SPOON REFUSAL		6	SS	100/0.13							



NOTES:

1. Water level in open borehole at a depth of 2.1 m below ground surface (Elev. 74.5 m) upon completion of drilling.

2. A Dynamic Cone Penetration Test was advanced 0.8 m South of Borehole E23, refusal encountered at a depth of 3.2 m below gound surface (Elev. 73.4 m).

3. An additional borehole was advanced 2 m West of Borehole E23, refusal encountered at a depth of 3.1 m below ground surface (Elev. 73.5 m).

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

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

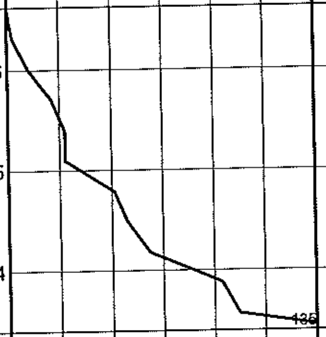
Golder Associates

RECORD OF PENETRATION TEST No E23A

1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ______ HWY 401
DATUM Geodetic
LOCATION N 4904813.3 ;E 308903.6
BOREHOLE TYPE Portable Equipment, Dynamic Cone Penetration Test
DATE March 9, 2009
ORIGINATED BY DM
COMPILED BY AT
CHECKED BY KSL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ 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						
76.6 0.0	GROUND SURFACE Start of Dynamic Cone Penetration Test (DCPT)						76 75 74				
73.4 3.2	END OF DCPT Refusal to further Penetration (135 blows/0.3 m)										



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

RECORD OF BOREHOLE No E23B

1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST HWY 401
DATUM Geodetic
LOCATION N 4904813.3 ;E 308902.0
BOREHOLE TYPE Portable Equipment, Continuous Sampling
DATE March 9, 2009
ORIGINATED BY DM
COMPILED BY AT
CHECKED BY KSL

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa ○ 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

SOIL PROFILE			SAMPLES					REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	GR	SA	SI	CL
76.7	GROUND SURFACE										
0.0	Clayey silt, some sand, containing rootlets (FILL) Soft Brown Moist		1	SS	3		76				
76.1 0.6	SILTY CLAY/CLAYEY SILT, trace to some sand, trace gravel Firm to hard Brown Moist		2	SS	14						
			3	SS	18	∇	75	0	3	51	46
			4	SS	40						
			5	SS	39		74	0	6	48	46
73.6 3.1	END OF BOREHOLE SPOON REFUSAL		6	SS	75/0.05						

NOTE:

1. Water level in open borehole at a depth of 1.8 m below ground surface (Elev.74.9 m) upon completion of drilling.

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

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

Golder Associates

RECORD OF BOREHOLE No E24

1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ____ HWY 401
DATUM Geodetic

LOCATION N 4904820.5 ;E 308930.1
BOREHOLE TYPE Portable Equipment, Continuous Sampling
DATE March 10, 2009

ORIGINATED BY DM
COMPILED BY AT
CHECKED BY KSL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	▽					GR	SA	SI	CL
75.1	GROUND SURFACE													
0.0	Peat (Fibrous), trace sand, containing rootlets and decomposed wood fragments (FILL)	▩	1	SS	39		75		105.1 ○					
74.5 0.6	SILTY CLAY, trace to some sand, trace gravel Hard Black to brown Wet	▨	2	SS	29		74							
	Very stiff Brown to grey Wet	▨	3	SS	27				├○┤		0	8	47	45
		▨	4	SS	100/0.15				○					
73.2 2.1	SILT, some sand, trace gravel Very dense Brown Wet						73							
	END OF BOREHOLE SPOON REFUSAL													

NOTE:

1. Water level in open borehole at ground surface (Elev.75.1 m) upon completion of drilling.

2. Two Dynamic Cone Penetration Tests were advanced 1.8 m East and 2.0 m South of Borehole E24, refusal encountered at a depth of 1.8 m and 2.0 m below gound surface (Elev.73.3 m and 73.1 m).

MIS-MTO 001 08-1111-0044.GPJ GAL-MISS.GDT 10/23/12 DD

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

PROJECT 08-1111-0044

RECORD OF PENETRATION TEST No E24A

1 OF 1 **METRIC**

G.W.P. 78-99-01

LOCATION N 4904819.9 ;E 308931.4

ORIGINATED BY DM

DIST ____ HWY 401

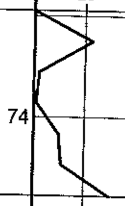
BOREHOLE TYPE Portable Equipment, Dynamic Cone Penetration Test

COMPILED BY AT

DATUM Geodetic

DATE March 10, 2009

CHECKED BY KSL



ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ 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
75.1	GROUND SURFACE										
0.0	Start of Dynamic cone Penetration Test (DCPT)						74				
73.2 1.8	END OF DCPT Refusal to further Penetration (Hammer Bouncing)										

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

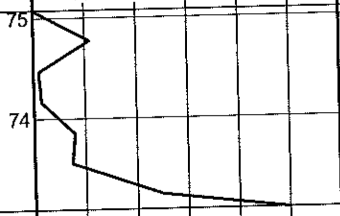
Golder Associates

RECORD OF PENETRATION TEST No E24B

1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ____ HWY 401
DATUM Geodetic
LOCATION N 4904818.7 ;E 308929.8
BOREHOLE TYPE Portable Equipment, Dynamic Cone Penetration Test
DATE March 10, 2009
ORIGINATED BY DM
COMPILED BY AT
CHECKED BY KSL

SOIL PROFILE		SAMPLES									
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 ○ 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
75.1	GROUND SURFACE						75				
0.0	Start of Dynamic cone Penetration Test (DCPT)						74				
73.1 2.0	END OF DCPT Refusal to Further Penetration (100 blows/0.15 m)										



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

RECORD OF BOREHOLE No S-1

1 OF 2 **METRIC**

PROJECT 08-1111-0044

G.W.P. 78-99-01

DIST ______ HWY 401

DATUM Geodetic

LOCATION N 4904370.0 ;E 307410.0

BOREHOLE TYPE Power Auger, 200mm Diam. Hollow Stem

DATE February 9, 2010

ORIGINATED BY DWM

COMPILED BY JM

CHECKED BY EO

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa ○ 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

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES			GR	SA	SI	CL
83.0	GROUND SURFACE									
0.0 82.6	Sand and gravel (BASE) Grey Frozen									
0.4	Sand, trace gravel and cobbles (SUBBASE) Compact Brown Moist	1	SS	28		82				
81.5 1.5	Fine Rock FILL, contains cobbles and boulders Loose to compact Grey Dry to moist	2	SS	11		81				
		3	SS	39			54	40	(6)	
		4	SS	50/0.30		80				
		5	SS	8		79				
		6	SS	10		78				
		7	SS	5						
						77				
		8	SS	6						
76.0 7.0	Fine Rock FILL, with silty matrix Compact Grey-brown Moist	9	SS	6		76				
	- Wet below 8.4 m depth	10	SS	9		75				
		11	SS	14	▽					
						74				
		12	SS	26						
		13	SS	12		73				
		14	SS	19		72	55	31	(14)	
71.5 11.5	Fine Rock FILL Compact Grey Wet	15	SS	22		71				
		16	SS	28						
		17	SS	19		70				
		18	SS	16		69				
68.0		19	SS	13						

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No S-1

2 OF 2 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ____ HWY 401
DATUM Geodetic

LOCATION N 4904370.0 ;E 307410.0
BOREHOLE TYPE Power Auger, 200mm Diam. Hollow Stem
DATE February 9, 2010

ORIGINATED BY DWM
COMPILED BY JM
CHECKED BY EO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ 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 —										
15.1	PEAT Brown Wet End of Borehole Note: Water level in open borehole at 8.5 m depth below ground surface upon completion of drilling										

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

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

Golder Associates

RECORD OF BOREHOLE No S-2

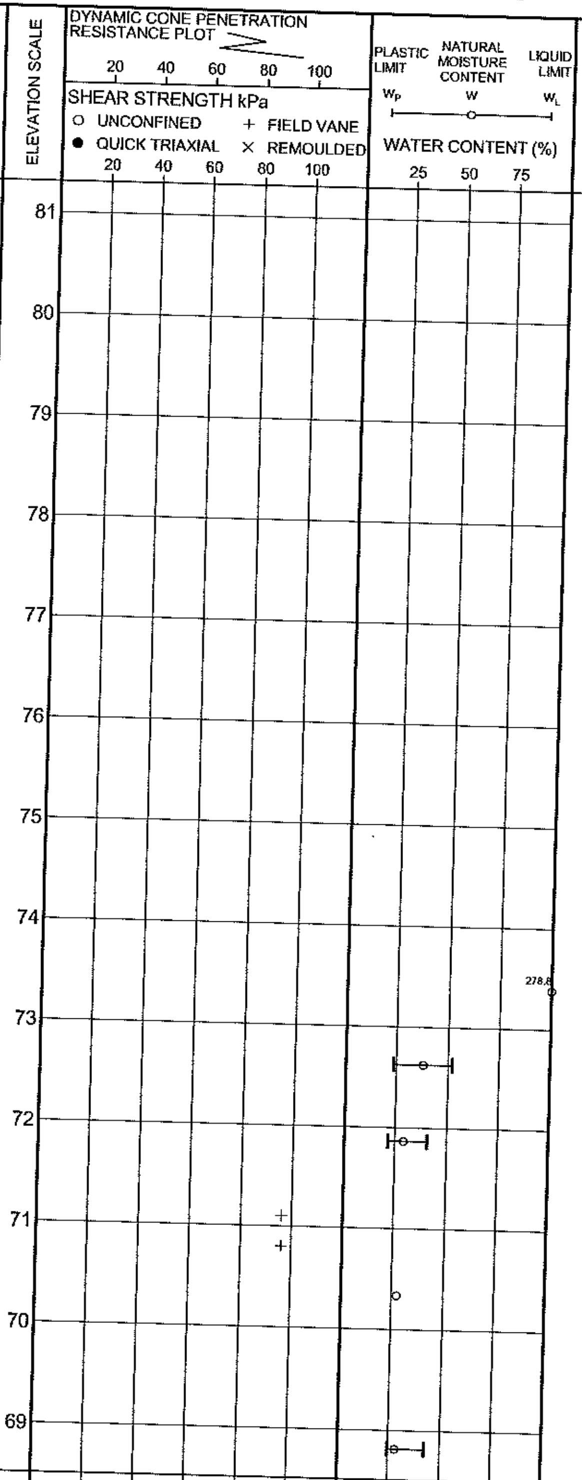
1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ____ HWY 401
DATUM Geodetic

LOCATION N 4904400.0 ;E 307503.2
BOREHOLE TYPE Power Auger, 200mm Diam. Hollow Stem
DATE February 8, 2010

ORIGINATED BY DWM
COMPILED BY JM
CHECKED BY EO

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS
81.3	GROUND SURFACE				
0.0	Sand and gravel (BASE) Grey				
80.9 0.4	Sand, some fine rock fill (FILL) Loose Brown Moist	1	SS	25	
		2	SS	7	
		3	SS	4	
78.3 3.1	Fine Rock FILL, some silty sand, contains cobbles and boulders Loose to very loose Grey-brown Moist to wet	4	SS	5	
		5	SS	8	
		6	SS	7	
		7	SS	3	
		8	SS	6	
	- *Trace organics below 6.7 m depth*	9	SS	31	
73.7 7.8	Silty clay (FILL) Grey Wet	10	SS	5	∇
72.9 8.4	PEAT, some decomposed wood Firm Brown to black Moist	11	SS	8	
72.2 9.1	Organic SILTY CLAY, some marl, trace rootlets Stiff Grey-brown Moist	12	SS	9	
	SILTY CLAY, trace sand, trace clayey silt seams Very stiff Grey Moist				
70.6 10.7	SILTY CLAY, trace silty sand seams Very stiff Grey Moist	13	SS	11	
		14	SS	10	
68.5 12.8	End of Borehole Note: Water level in open borehole at 7.6 m depth below ground surface upon completion of drilling				



UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL

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

Golder Associates

RECORD OF BOREHOLE No S-3

1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ____ HWY 401
DATUM Geodetic
LOCATION N 4904443.0 ;E 307704.1
BOREHOLE TYPE Power Auger, 200mm Diam. Hollow Stem
DATE February 10, 2010
ORIGINATED BY DWM
COMPILED BY JM
CHECKED BY EO

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLES – NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR	SA	SI	CL
78.8	GROUND SURFACE									
78.8 0.0	Sand and gravel (BASE) Grey Frozen									
78.5 0.4	Sand, some gravel (SUBBASE) Brown					78				
77.6 1.2	Fine Rock FILL, contains cobbles and boulders Loose to compact Grey-brown Dry to wet	1	SS	5		77				
		2	SS	9		76				
		3	SS	15		75				
		4	SS	36	▽					
		5	SS	9		74				
73.5 5.3	PEAT, some fibers Firm Dark brown Wet	6	SS	5		73				
72.6 6.3	SILTY CLAY Very stiff Grey Wet	7	SS	5		72				
		8	TP	PH		71				
		9	SS	14		70	0	2	49	49
69.9 9.0	End of Borehole Note: Water level in open borehole at 4.6 m depth below ground surface upon completion of drilling									

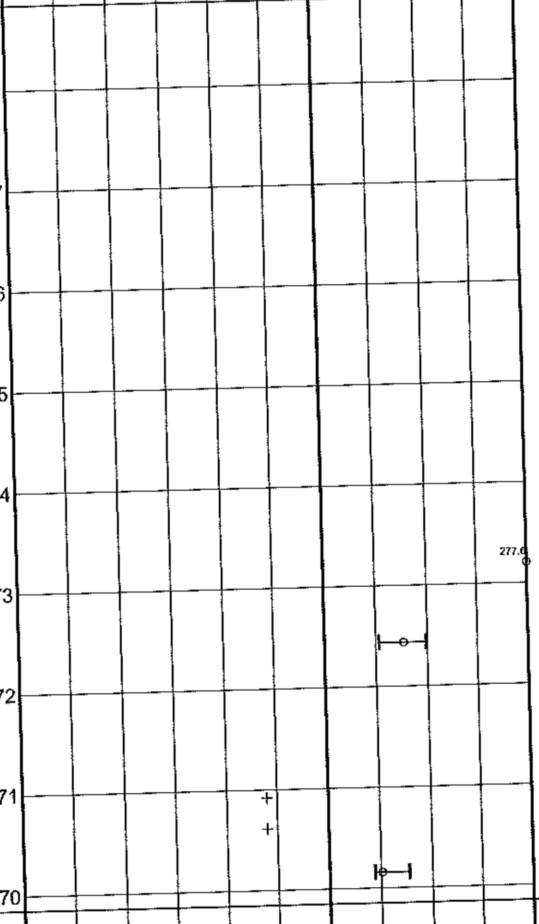
DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa
○ 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



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

PROJECT 08-1111-0044

RECORD OF BOREHOLE No S-4

1 OF 1 **METRIC**

G.W.P. 78-99-01

LOCATION N 4904788.9 ;E 308748.0

ORIGINATED BY DWM

DIST ____ HWY 401

BOREHOLE TYPE Power Auger, 200mm Diam. Hollow Stem

COMPILED BY JM

DATUM Geodetic

DATE February 11, 2010

CHECKED BY EO

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	
81.4	GROUND SURFACE					
0.0	Sand and gravel (BASE) Grey					
0.2	Sand and gravel (SUBBASE) Grey-brown					
80.7 0.7	Sand, some gravel, contains cobbles (FILL) Compact Red brown Moist		1	SS	21	
78.3 3.1	Fine Rock FILL, contains cobbles and boulders Loose to compact Grey-brown Dry *- Boulder at 4.0 m depth* *- Wet below 5.9 m depth* *- Trace organics below 6.4 m depth*		2	SS	9	
			3	SS	6	
						∇
			4	SS	26	
74.5	PEAT Brown Wet		5	SS	12	
74.2 7.2	Organic CLAY		5A	SS	12	
73.7 7.7	SILTY CLAY, trace silty sand, trace silt seams Very stiff Grey Moist		6	TP	PH	
			7	SS	21	
71.6 9.8	End of Borehole Note: Water level in open borehole at 5.9 m depth below ground surface upon completion of drilling					

ELEVATION SCALE

DYNAMIC CONE PENETRATION RESISTANCE PLOT

20 40 60 80 100

SHEAR STRENGTH kPa

○ UNCONFINED + FIELD VANE

● QUICK TRIAXIAL × REMOULDED

20 40 60 80 100

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

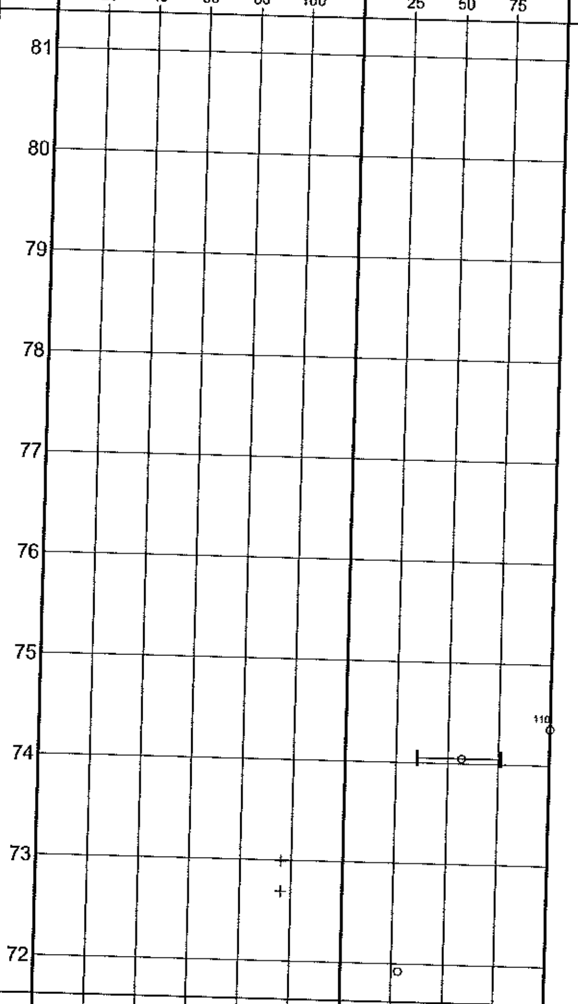
WATER CONTENT (%)

25 50 75

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

GR SA SI CL



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

Golder Associates

RECORD OF BOREHOLE No W1

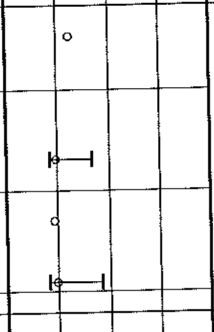
1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ______ HWY 401
DATUM Geodetic

LOCATION N 4904355.6 ;E 307285.1
BOREHOLE TYPE Portable Equipment, Continuous Sampling
DATE February 11, 2009

ORIGINATED BY DM
COMPILED BY AT
CHECKED BY KSL

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L WATER CONTENT (%) 25 50 75	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						GR	SA	SI	CL
75.8	GROUND SURFACE													
0.0	Silty sand to sandy silt, some gravel, some clay (FILL) Loose Brown Moist	▩	1	SS	8									
75.2 0.6	Clayey silt, trace to some gravel, trace to some sand, containing wood fragments (FILL) Stiff Brown Moist	▩	2	SS	12		75							
74.6 1.2	SILTY CLAY, trace gravel, trace sand Hard Brown to grey Moist	▨	3	SS	34									
		▨	4	SS	47		74				0	3	53	44
		▨	5	SS	62		73							
72.8 3.1	END OF BOREHOLE NOTE: 1. Open borehole dry upon completion of drilling.													



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

RECORD OF BOREHOLE No W2

1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ______ HWY 401
DATUM Geodetic

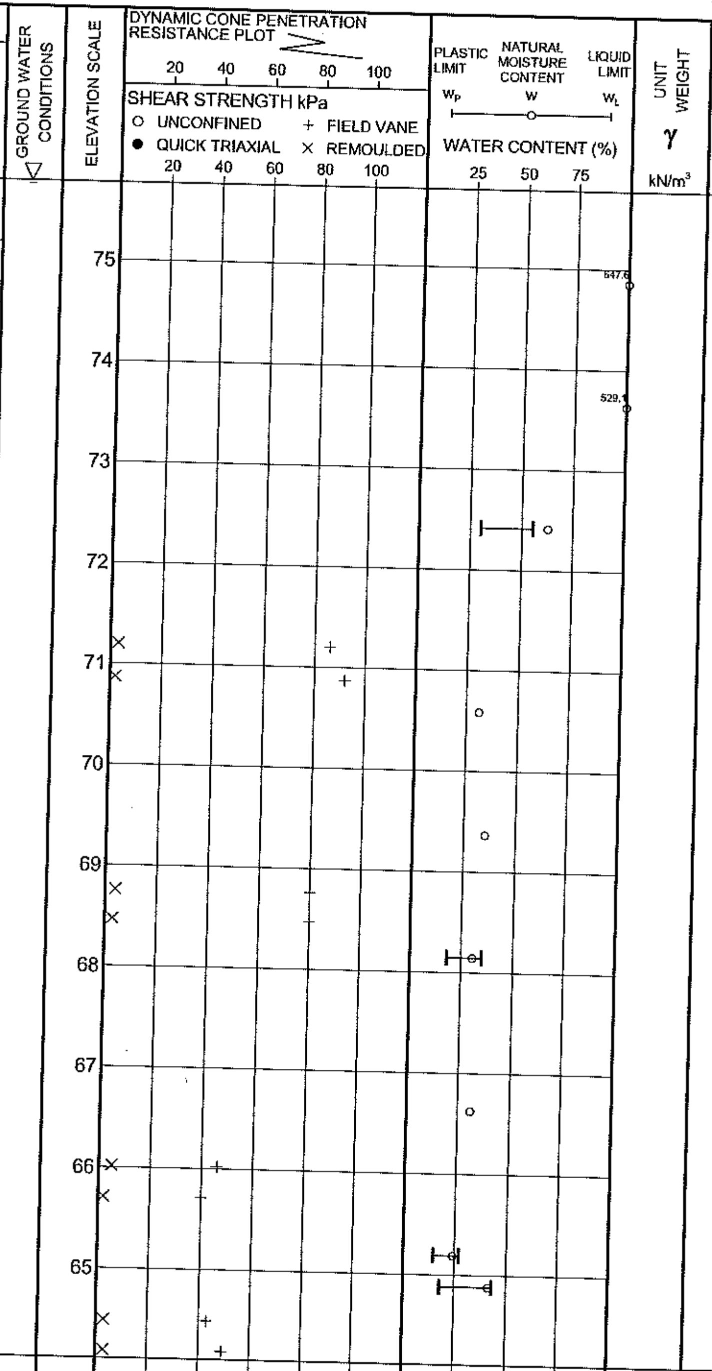
LOCATION N 4904361.4 ;E 307310.1
BOREHOLE TYPE Portable Equipment, Continuous Sampling, BW Casing, Wash Boring
DATE February 5 and 6, 2009

ORIGINATED BY DM
COMPILED BY AT
CHECKED BY KSL

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
75.8	GROUND SURFACE				
0.0	PEAT (Fibrous) Very soft to firm Dark brown to black Wet	1	SS	3	
		2	SS	2	OC=77.8%
	Becoming reddish brown at a depth of 1.2 m	3	SS	1	
	Containing decomposed wood fragments at a depth of 1.8 m	4	SS	4	
72.8		5	SS	6	
3.0	Organic SILTY CLAY Soft Grey to greenish brown Moist	6	SS	2	
		7	SS	4	
71.2 4.6	SILTY CLAY, trace sand, trace gravel Firm to stiff Grey Moist to wet				
		8	SS	10	
		9	SS	12	
		10	SS	6	
		11	SS	8	
		12	SS	9	
		13	SS	5	
65.1 10.7	CLAYEY SILT Firm Grey Wet	14	TO	PM	
64.1 11.7	END OF BOREHOLE SPOON REFUSAL				

NOTE:

1. Water level at ground surface (Elev. 75.8 m) upon completion of drilling



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

Golder Associates

RECORD OF BOREHOLE No W3

1 OF 1 **METRIC**

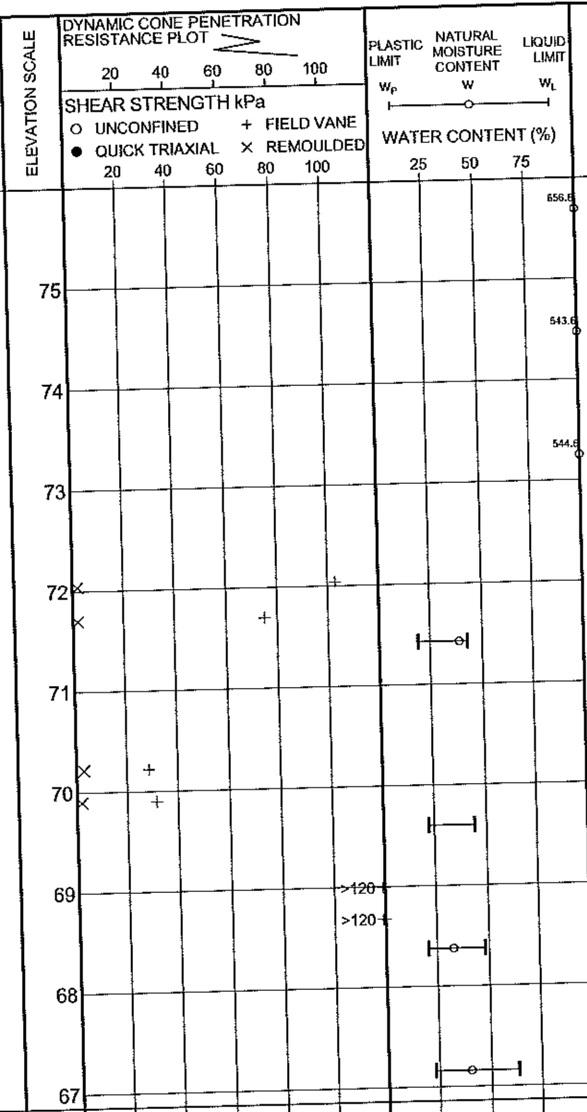
PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ____ HWY 401
DATUM Geodetic

LOCATION N 4904375.9 ;E 307356.4
BOREHOLE TYPE Portable Equipment, Continuous Sampling, BW Casing, Wash Boring
DATE February 11 and 12, 2009

ORIGINATED BY DM
COMPILED BY AT
CHECKED BY KSL

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
76.0	GROUND SURFACE					
0.0	PEAT (Fibrous) Very soft to firm Black to reddish brown Wet	1	SS	2		
		2	SS	1		
		3	SS	3		
74.2 1.8	Silty PEAT (Fibrous) Firm Brown Wet Containing wood fragments between depths of 2.4 m and 3.6 m	4	SS	7		
		5	SS	7		
		6A 6B	SS	4		OC = 3.9%
72.4 3.6	SILTY CLAY, trace sand, containing shell fragments to a depth of 5.8 m Firm to very stiff Grey Wet					
		7	SS	7		0 0 38 62
		8	SS	3		
		9	TO	PM	20.4	0 1 32 67
						0 2 34 64
		10	SS	20		
		11	SS	16		
67.5 8.5	CLAY, some silt, trace sand Very stiff Grey Wet	12	SS	17		
66.9 9.1	END OF BOREHOLE					

GROUND WATER CONDITIONS ▽



NOTE:

1. Water level at ground surface (Elev. 76.0 m) upon completion of drilling.

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

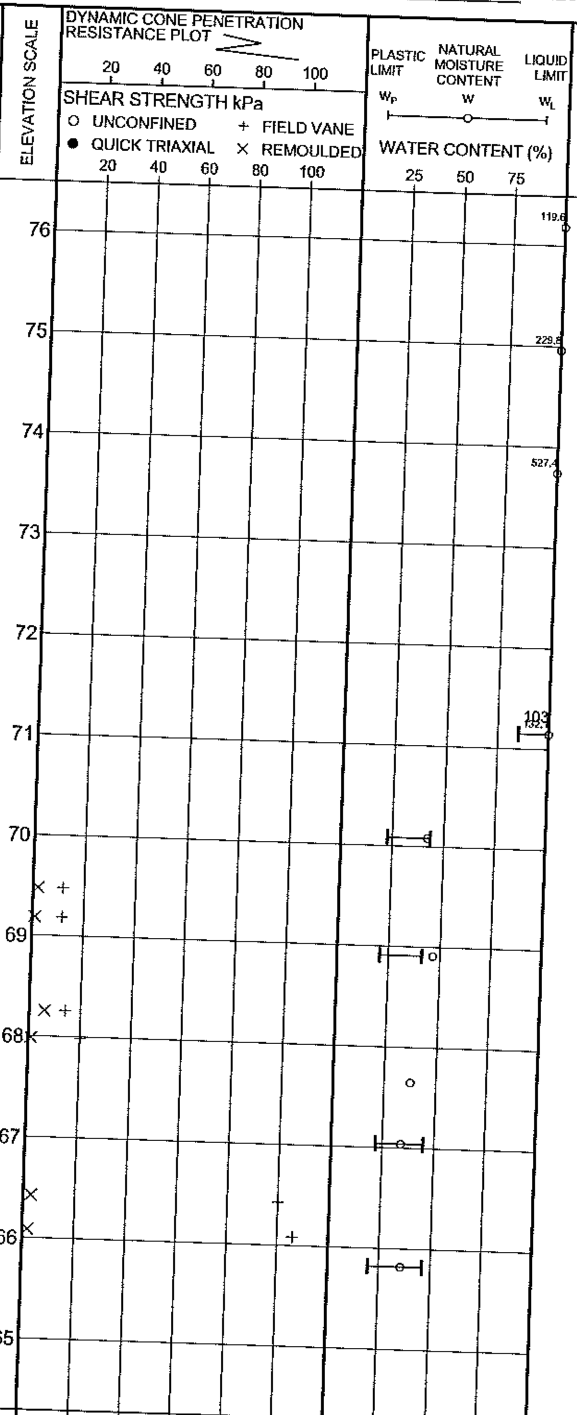
$\circ^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No W4

1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST HWY 401
DATUM Geodetic
LOCATION N 4904389.5 ;E 307407.6
BOREHOLE TYPE Portable Equipment, Continuous Sampling, BW Casing, Wash Boring
DATE February 12 and 13, 2009
ORIGINATED BY DM
COMPILED BY AT
CHECKED BY KSL

SOIL PROFILE		SAMPLES				
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
76.5	GROUND SURFACE					
0.0	Sandy silt to silty sand, trace gravel, trace clay (FILL) Very loose Dark brown to brown	1	SS	3	∇	
75.9	Silty PEAT (Fibrous), containing shell and wood fragments Soft to stiff Black to reddish brown Wet	2	SS	7		
		3	SS	5		OC = 44.3%
		4	SS	3		
		5	SS	7		
		6	SS	5		
		7	SS	8		
		8	SS	10		
71.3 5.2	Organic SILT, some sand, trace to some clay, trace gravel, containing shell fragments Stiff Light brown Wet	9	SS	14		
		10	SS	9		1 16 72 11
70.4 6.1	SILTY CLAY, trace sand Very soft to very stiff Grey Wet	11	SS	1		
		12	TO	PM		
		13	SS	6		
		14	TO	PM		
		15	SS	6		
		16	SS	13		
		17	SS	12		
64.3 12.2	END OF BOREHOLE					



UNIT WEIGHT γ kN/m³

NOTE:

1. Water level in open borehole at a depth of 0.6 m below ground surface (Elev. 75.9 m) upon completion of drilling.

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

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

Golder Associates

RECORD OF BOREHOLE No W5

1 OF 2 **METRIC**

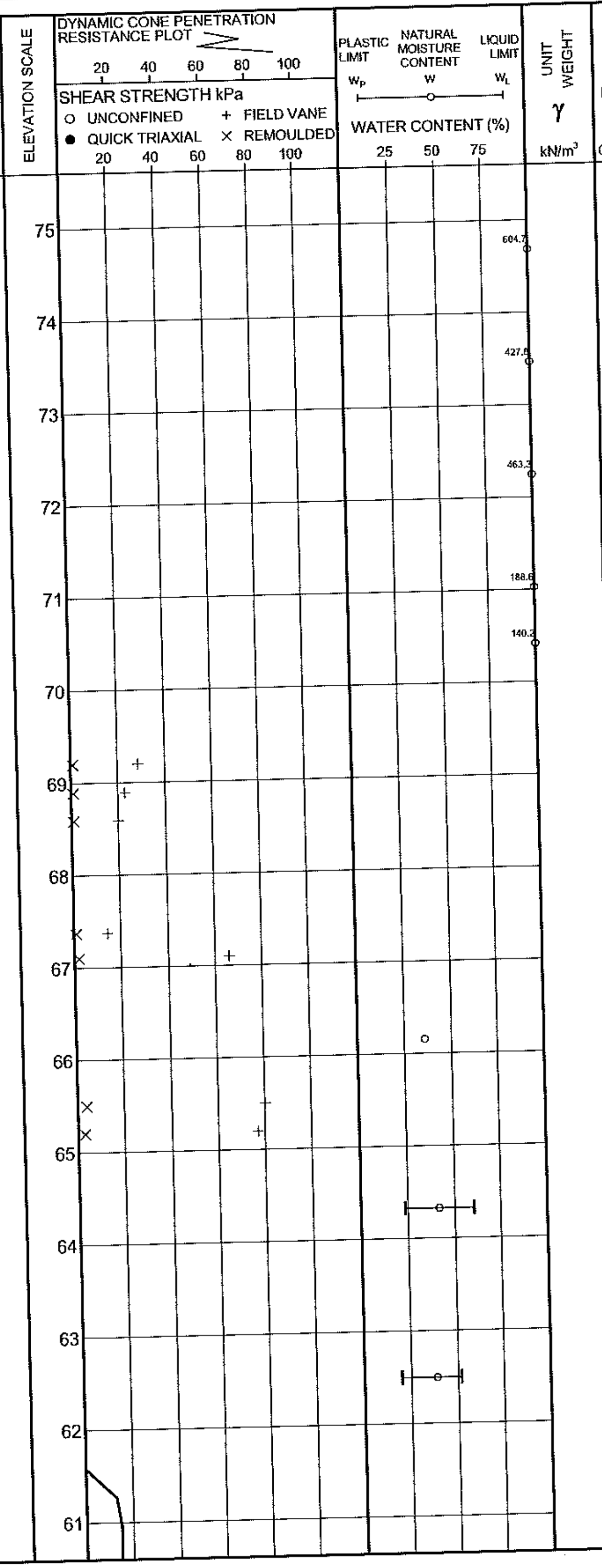
PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ____ HWY 401
DATUM Geodetic

LOCATION N 4904404.0 ;E 307451.1
BOREHOLE TYPE Portable Equipment, Continuous Sampling, BW Casing, Wash Boring and DCPT
DATE February 17 and 18, 2009

ORIGINATED BY DM
COMPILED BY AT
CHECKED BY KSL

ELEV DEPTH	DESCRIPTION	STRAT PLOT
75.6	GROUND SURFACE	
0.0	PEAT (Fibrous), trace sand Soft to firm Black to dark brown Wet Containing decomposed wood fragments below a depth of 2.4 m	▤
71.4 4.2	Organic CLAYEY SILT, trace to some sand, containing shell fragments Firm Wet	▤
70.1 5.5	CLAY, some silt, trace sand Soft to stiff Grey Wet	▨
61.6 14.0	END OF BOREHOLE	

NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	NATURAL MOISTURE CONTENT w (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	3	∇		
2	SS	2		604.7	
3	SS	2			
4	SS	2		427.0	
5	SS	2			
6	SS	4		463.3	OC=75.3%
7	SS	5			
8	SS	5		188.6	
9	SS	4		140.2	0 8 71 21
10	SS	WH			
11	TO	PM			
12	SS	7			
13	SS	8			
14	SS	11			
15	SS	9			
16	SS	12			
17	SS	10			0 1 32 67
18	SS	12			



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

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

RECORD OF BOREHOLE No W5

2 OF 2 **METRIC**

PROJECT 08-1111-0044

G.W.P. 78-99-01

DIST ______ HWY 401

DATUM Geodetic

LOCATION N 4904404.0 ;E 307451.1

BOREHOLE TYPE Portable Equipment, Continuous Sampling, BW Casing, Wash Boring and DCPT

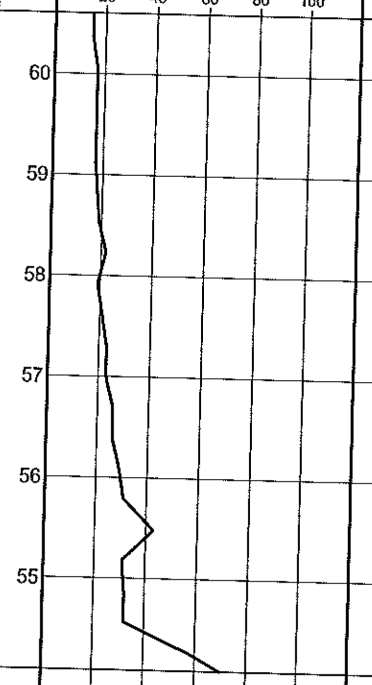
DATE February 17 and 18, 2009

ORIGINATED BY DM

COMPILED BY AT

CHECKED BY KSL

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ 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 — CONTINUED FROM PREVIOUS PAGE —	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
54.1 21.5	END OF DCPT Refusal to further penetration (Hammer bouncing) NOTE: 1. Water level in open borehole at a depth of 0.3 m blow ground surface (Elev. 75.3 m) upon completion of drilling.										



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

Golder Associates

RECORD OF BOREHOLE No W6

1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ______ HWY 401
DATUM Geodetic

LOCATION N 4904417.9 ;E 307497.2
BOREHOLE TYPE Portable Equipment, Continuous Sampling
DATE February 18, 2009

ORIGINATED BY JEB
COMPILED BY AT
CHECKED BY KSL

ELEV DEPTH	SOIL PROFILE DESCRIPTION
75.5	GROUND SURFACE
0.0	PEAT (Fibrous), trace sand Very soft to firm Black Wet Clayey silt layer at a depth of 1.1 m Contains decomposed wood between depths of 3.1 m and 3.6 m
71.4 4.3	CLAYEY SILT, containing peat Firm Grey Wet
	SILTY CLAY, trace sand Firm to hard Grey Moist
67.6 7.9	END OF BOREHOLE

SAMPLES NUMBER	TYPE	"N" VALUES
1	SS	2
2	SS	4
3	SS	WH
4	SS	2
5	SS	6
6	SS	4
7	SS	7
8	SS	6
9	SS	24
10	SS	42
11	SS	70
12	SS	104

GROUND WATER CONDITIONS ▽

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

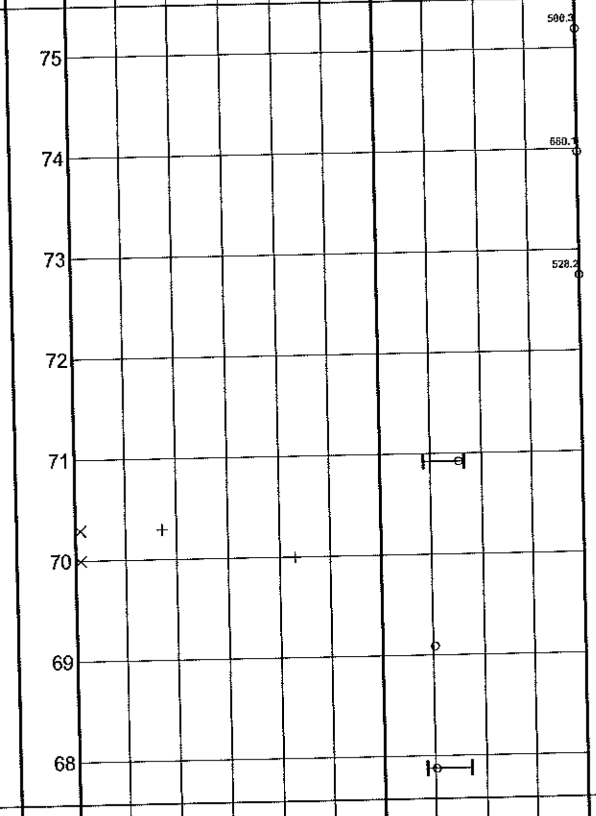
SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × REMOULDED
20 40 60 80 100

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



NOTE:

1. Water level in open borehole at ground surface (Elev. 75.5 m) upon completion of drilling.

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

MIS-MTO 001 08-1111-0044.GPJ GAL-MISS.GDT 10/23/12 DD

RECORD OF BOREHOLE No W7

1 OF 1 **METRIC**

PROJECT 08-1111-0044

G.W.P. 78-99-01

LOCATION N 4904432.1 ;E 307544.5

ORIGINATED BY JEB

DIST ______ HWY 401

BOREHOLE TYPE Portable Equipment, Continuous Sampling

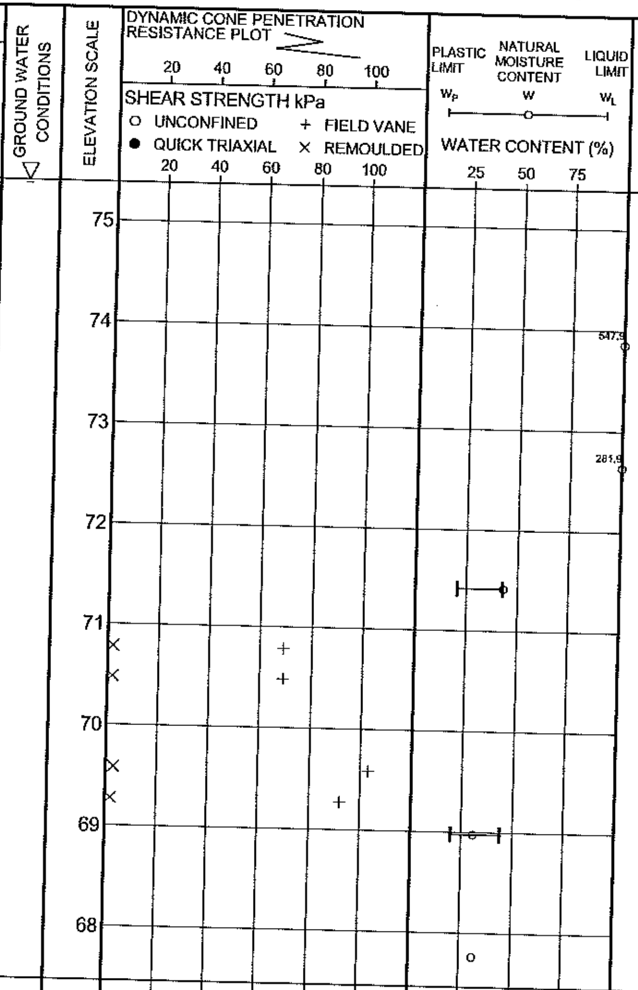
COMPILED BY AT

DATUM Geodetic

DATE February 18, 2009

CHECKED BY KSL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
75.4	ICE SURFACE				
0.0 0.1	ICE WATER	1	SS	2	
		2	SS	WH	
74.2 1.2	Sandy PEAT (Fibrous) Very soft to firm Black Wet	3	SS	WH	OC = 51.8%
		4	SS	WH	
72.9 2.4	Organic CLAYEY SILT Soft to firm Black Wet	5	SS	2	
		6	SS	6	
71.7 3.7	SILTY CLAY, trace sand, slightly organic Soft to stiff Grey Moist	7	SS	4	OC = 3.5%
70.8 4.6	SILTY CLAY, trace sand Stiff to hard Grey Moist	8	SS	10	
		9	SS	34	
		10	SS	24	
		11	SS	60	
67.5 7.9	END OF BOREHOLE				



UNIT WEIGHT γ kN/m³

NOTE:

1. Water level in open borehole at ice surface (Elev. 75.4 m) upon completion of drilling.

+³, ×³: Numbers refer to Sensitivity

○³% STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No W8

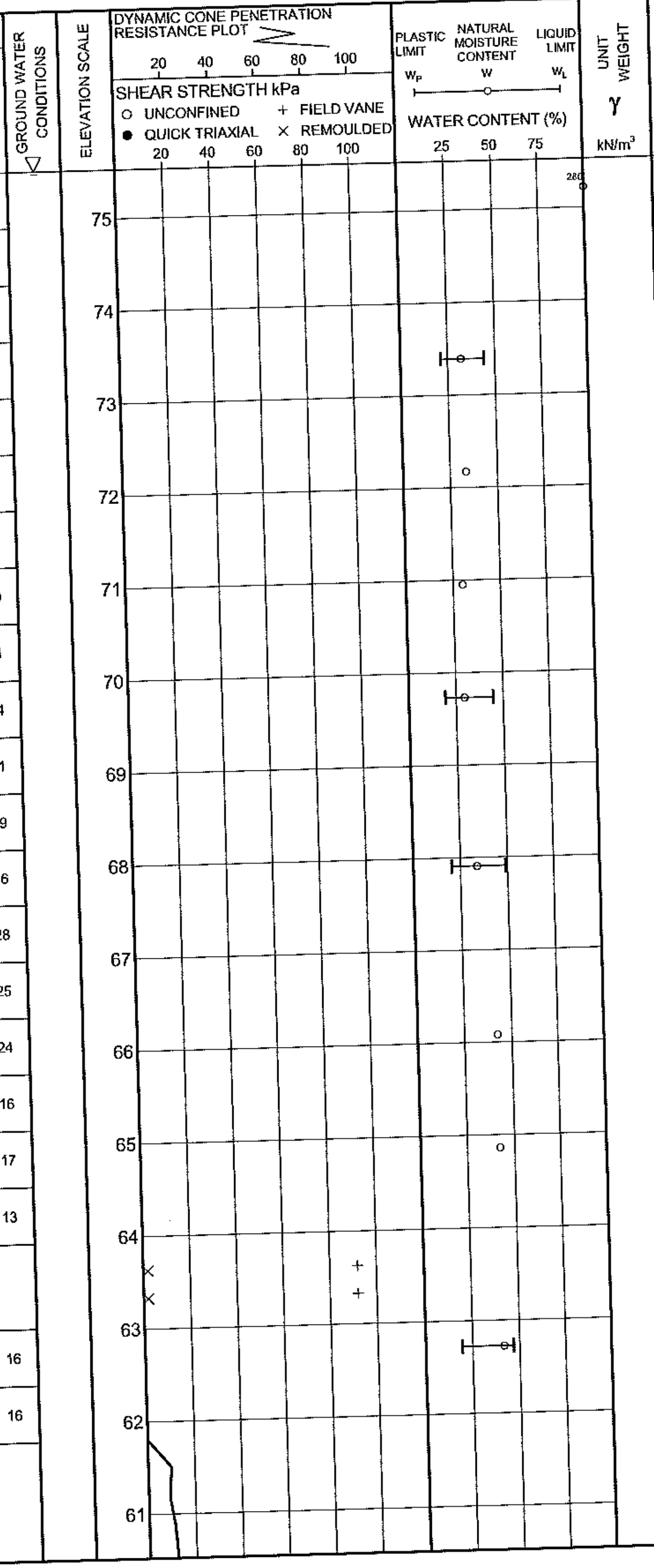
1 OF 2 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ____ HWY 401
DATUM Geodetic

LOCATION N 4904446.9 ;E 307595.5
BOREHOLE TYPE Portable Equipment, Continuous Sampling, and DCPT
DATE February 12 and 13, 2009

ORIGINATED BY JEB
COMPILED BY AT
CHECKED BY KSL

SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
75.5	GROUND SURFACE							
0.0	PEAT (Fibrous), trace sand	1	SS	52				
74.9	ICE and WATER	2	SS	2				
74.6 0.9	SILTY CLAY, containing peat (Fibrous) Soft Black Wet	3	SS	4				
73.7 1.8	SILTY CLAY, trace sand Stiff to hard Grey to brown Moist	4	SS	16				
		5	SS	13				
		6	SS	23				
		7	SS	46				
		8	SS	40				
	Trace gravel below a depth of 4.9 m	9	SS	31				
		10	SS	34				
		11	SS	41				
		12	SS	19				
		13	SS	16	0	2	33	65
		14	SS	28				
		15	SS	25				
		16	SS	24				
		17	SS	16				
		18	SS	17				
		19	SS	13				
		20	SS	16				
		21	SS	16				
61.8 13.7	END OF BOREHOLE							



Continued Next Page

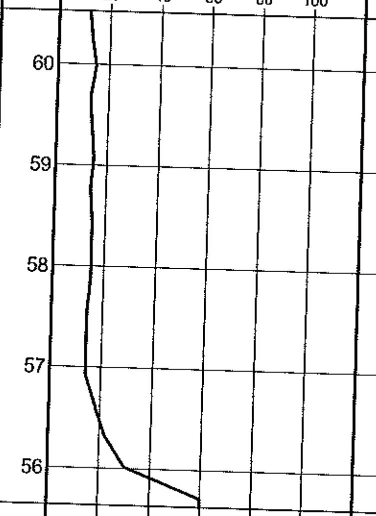
$+^3, \times^3$: Numbers refer to Sensitivity ○ $^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No W8

2 OF 2 **METRIC**

PROJECT 08-1111-0044			
G.W.P. 78-99-01	LOCATION N 4904446.9 ;E 307595.5	ORIGINATED BY JEB	
DIST ____ HWY 401	BOREHOLE TYPE Portable Equipment, Continuous Sampling, and DCPT	COMPILED BY AT	
DATUM Geodetic	DATE February 12 and 13, 2009	CHECKED BY KSL	

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ 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 — CONTINUED FROM PREVIOUS PAGE —	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
55.6 19.9	END OF DCPT Refusal to Further Penetration (60 blows/0.1 m)										



NOTES:

1. Water level in open borehole at ground surface (Elev. 75.5 m) upon completion of drilling.

2. Open borehole caved at a depth of 13.4 m below ground surface (Elev. 61.5 m) upon completion of drilling.

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

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

Golder Associates

RECORD OF BOREHOLE No W9

1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ______ HWY 401
DATUM Geodetic

LOCATION N 4904462.0 ;E 307639.8
BOREHOLE TYPE Portable Equipment, Continuous Sampling
DATE February 17, 2009

ORIGINATED BY JEB
COMPILED BY AT
CHECKED BY KSL

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
75.4	ICE SURFACE					∇	
0.0	ICE						
0.2	WATER		1	SS	WH		75
			2	SS	WH		
							74
73.8 1.7	PEAT (Fibrous), trace sand Very soft Black Wet		3	SS	WH		
73.1 2.3	SILTY CLAY, trace to some sand Soft to hard Grey Moist Containing rootlets to a depth of 2.9 m		4	SS	4		73
			5	SS	10		72
			6	SS	28		
			7	SS	7		71
			8	SS	31		70
			9	SS	16		
			10	SS	15		69
68.7 6.7	END OF BOREHOLE NOTE: 1. Water level in open borehole at ice surface (Elev. 75.4 m) upon completion of drilling.						

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

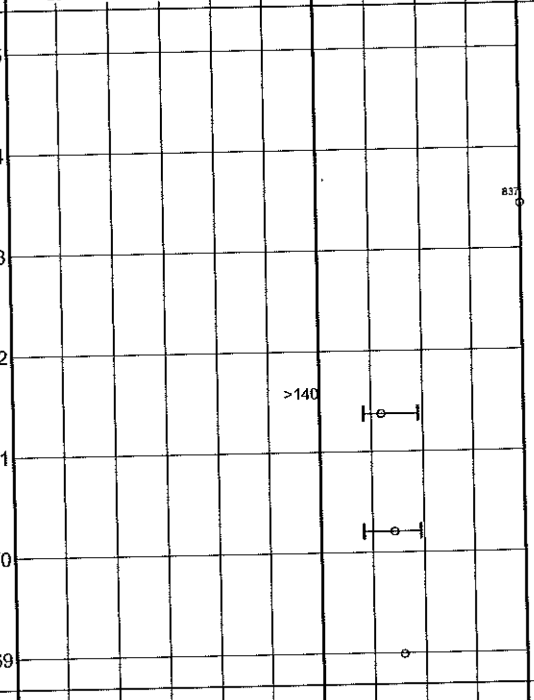
SHEAR STRENGTH kPa
○ UNCONFINED + FIELD VANE
● QUICK TRIAXIAL × REMOULDED
20 40 60 80 100

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

PROJECT 08-1111-0044

RECORD OF BOREHOLE No W10

1 OF 1 **METRIC**

G.W.P. 78-99-01

LOCATION N 4904475.4 ;E 307687.8

ORIGINATED BY JEB

DIST ____ HWY 401

BOREHOLE TYPE Portable Equipment, Continuous Sampling

COMPILED BY AT

DATUM Geodetic

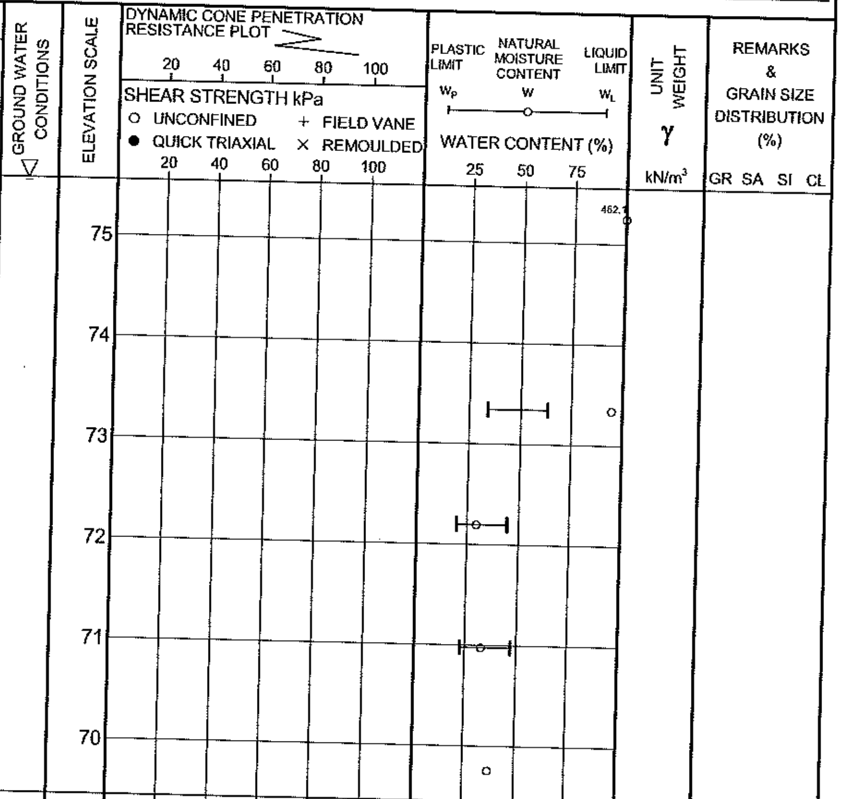
DATE February 12, 2009

CHECKED BY KSL

SOIL PROFILE			SAMPLES		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES
75.6	GROUND SURFACE				
0.0	PEAT (Fibrous), trace sand Very soft to stiff Black Wet	▤	1	SS	12
			2	SS	3
			3	SS	2
73.6 2.0	Organic SILT, trace sand, trace clay Soft Black Moist	▤	4	SS	3
73.1 2.4	SILTY CLAY, trace sand Stiff to hard Grey and brown Moist Containing rootlets to a depth of 3.1 m Becoming grey at a depth of 3.7 m	▨	5	SS	13
			6	SS	18
			7	SS	40
			8	SS	57
70.7 4.9	CLAYEY SILT, trace sand Very stiff Grey Moist	▨	9	SS	24
			10	SS	20
69.5 6.1	END OF BOREHOLE				

NOTE:

1. Water level in open borehole at ground surface (Elev. 75.6 m) upon completion of drilling.



+³, ×³: Numbers refer to Sensitivity

○³% STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No W11

1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST HWY 401
DATUM Geodetic

LOCATION N 4904492.0 ;E 307735.9
BOREHOLE TYPE Portable Equipment, Continuous Sampling
DATE February 11, 2009

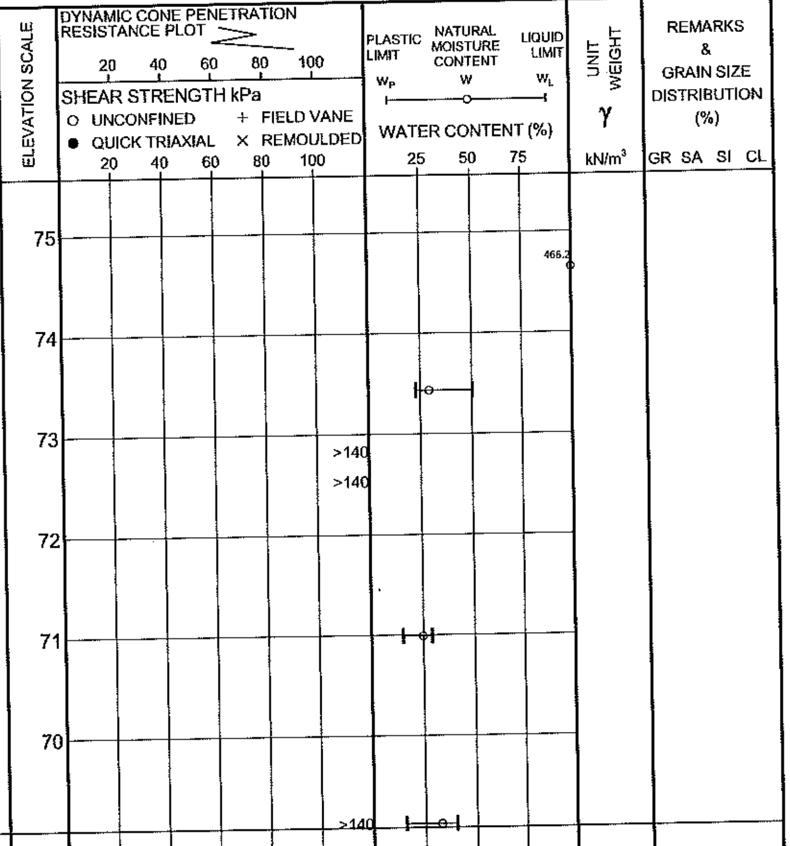
ORIGINATED BY JEB
COMPILED BY AT
CHECKED BY KSL

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS
75.6	GROUND SURFACE				
0.0	PEAT (Fibrous), trace sand, trace silt, trace clay Very soft to soft Black Wet	1	SS	4	∇
		2	SS	1	
		3	SS	4	
73.9 1.7	CLAY, some silt, trace sand, containing rootlets Firm Black to grey Moist	4	SS	9	
72.8 2.7	CLAYEY SILT, trace gravel, trace sand Very stiff to hard Grey to brown Moist				
		5	SS	54	
		6	SS	39	
		7	SS	48	
		8	SS	20	
		9	SS	19	
69.3 6.5	SILTY CLAY, some sand, trace gravel Hard Grey	10	SS	97/0.23	

END OF BOREHOLE
SPOON REFUSAL

NOTE:

1. Water level in open borehole at a depth of 0.2 m below ground surface (Elev. 75.4 m) upon completion of drilling.



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

RECORD OF BOREHOLE No W12

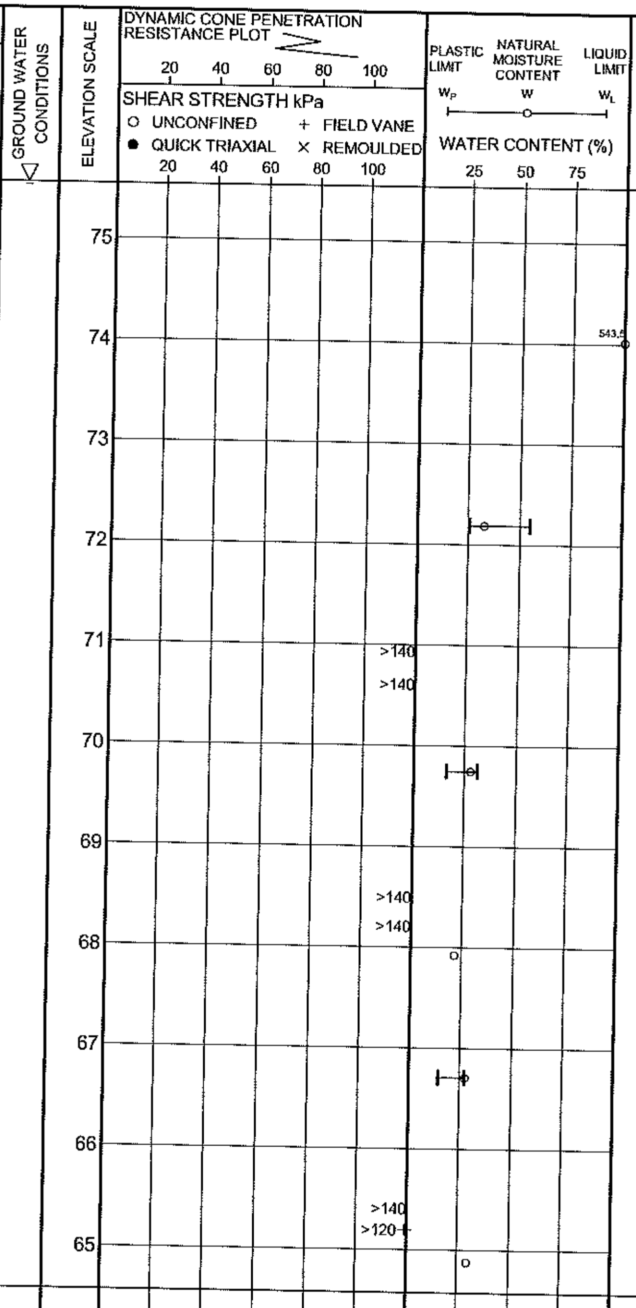
1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ______ HWY 401
DATUM Geodetic

LOCATION N 4904507.7 ;E 307785.2
BOREHOLE TYPE Portable Equipment, Continuous Sampling
DATE February 4, 2009

ORIGINATED BY DM
COMPILED BY AT
CHECKED BY KSL

SOIL PROFILE		SAMPLES			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
75.5	GROUND SURFACE							
0.0	PEAT (Fibrous) Very soft to very stiff Dark brown Wet	1	SS	19				
		2	SS	1				
		3	SS	2				
		4	SS	1				
73.3 2.4	SILTY CLAY, slightly organic Grey Wet							
	SILTY CLAY to CLAY, trace to some sand Very stiff to hard Grey greenish brown Moist	5	SS	13				
		6	SS	31	0	9	35	56
		7	SS	15				
70.7 4.9	CLAYEY SILT, trace sand Stiff to very stiff Grey Moist to wet	8	SS	11				
		9	SS	9	0	0	67	33
		10	SS	9				
		11	SS	15				
		12	SS	18				
		13	SS	15	0	1	66	33
		14	SS	10				
		15	SS	9				
64.6 11.0	END OF BOREHOLE							



NOTE:

1. Water level in open borehole at ground surface (Elev. 75.5 m) upon completion of drilling.

+^{3}, ×^{3}: Numbers refer to Sensitivity

○$^{3\%}$ STRAIN AT FAILURE

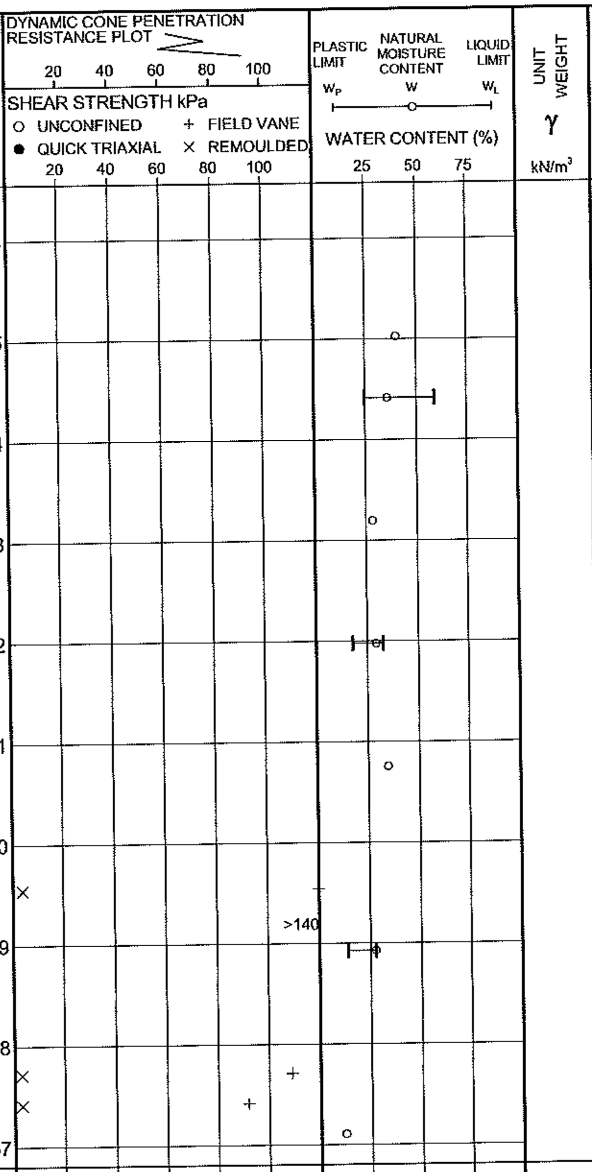
Golder Associates

RECORD OF BOREHOLE No W13

1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ____ HWY 401
DATUM Geodetic
LOCATION N 4904522.7 ;E 307834.5
BOREHOLE TYPE Portable Equipment, Continuous Sampling
DATE February 3, 2009
ORIGINATED BY DM
COMPILED BY AT
CHECKED BY KSL

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES			GR	SA	SI	CL
76.6	GROUND SURFACE									
0.0	Clayey silt, some sand, trace gravel, containing rootlets (FILL) Firm to very stiff Dark brown to brown Moist	1	SS	14		76				
		2	SS	7						
75.2 / 1.4	Organic SILTY CLAY Firm Grey to brown Moist	3	SS	5		75	OC = 9.3%			
74.7 / 1.8	CLAY, containing rootlets Stiff Grey to brown Moist	4	SS	9						
74.1 / 2.4	CLAYEY SILT, trace sand Firm to very stiff Brown Moist	5	SS	18	∇	74				
		6	SS	18		73				
		7	SS	15						
		8	SS	14		72	0	1	62	37
		9	SS	11						
		10	SS	8		71				
	Becoming grey below a depth of 6.1 m	11	SS	8		70				
		12	SS	13		69	0	1	69	30
		13	SS	7		68				
67.0		14	SS	12		67				
9.8	Silty SAND, trace gravel END OF BOREHOLE									



NOTE:

1. Water level in open borehole at a depth of 2.9 m below ground surface (Elev. 73.7 m) upon completion of drilling.

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

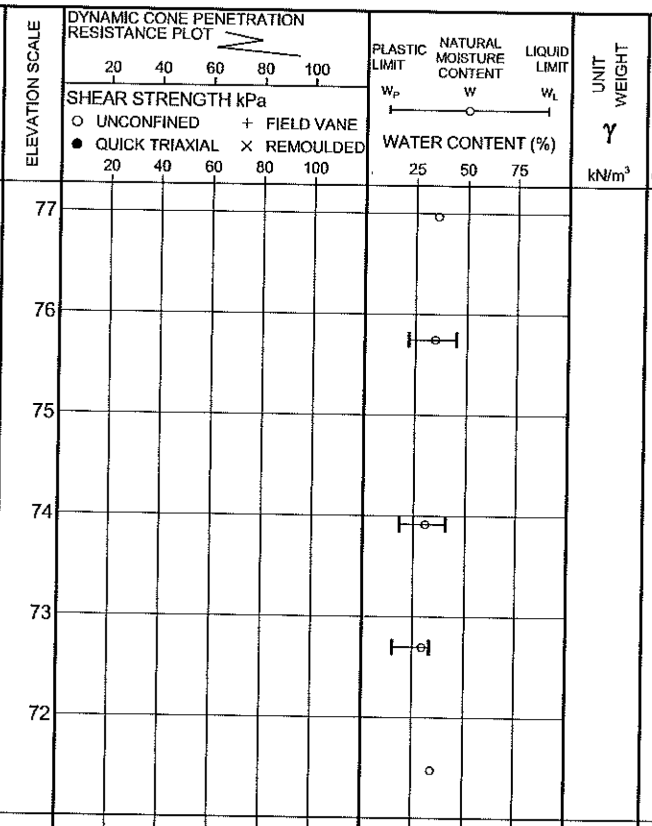
$○^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No W14

1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST
HWY 401
DATUM Geodetic
LOCATION N 4904536.9 ;E 307880.7
BOREHOLE TYPE Portable Equipment, Continuous Sampling
DATE February 2, 2009
ORIGINATED BY DM
COMPILED BY AT
CHECKED BY KSL

SOIL PROFILE		SAMPLES				REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	GR	SA	SI	CL
77.3	GROUND SURFACE								
0.0	Silty clay, containing rootlets, trace gravel (FILL) Stiff Brown to dark brown Moist	1	SS	12					
76.7 0.6	SILTY CLAY, trace sand Stiff to very stiff Brown to grey Moist	2	SS	9					
		3	SS	12					
		4	SS	14					
		5	SS	17	∇				
		6	SS	13					
		7	SS	14					
73.0 4.3	CLAYEY SILT/SILTY CLAY, trace sand Firm to stiff Brown to dark brown Moist	8	SS	13		0	1	62	37
		9	SS	15					
		10	SS	6		0	1	44	55
		11	SS	8					
71.0 6.3	END OF BOREHOLE SPOON REFUSAL								



NOTE:

1. Water level in open borehole at a depth of 2.7 m below ground surface (Elev. 74.6 m) upon completion of drilling.

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

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

Golder Associates

RECORD OF BOREHOLE No W15

1 OF 1 **METRIC**

PROJECT 08-1111-0044
G.W.P. 78-99-01
DIST ______ HWY 401
DATUM Geodetic
LOCATION N 4904551.7 ;E 307926.3
BOREHOLE TYPE Portable Equipment, Continuous Sampling
DATE February 2, 2009
ORIGINATED BY DM
COMPILED BY AT
CHECKED BY KSL

SOIL PROFILE

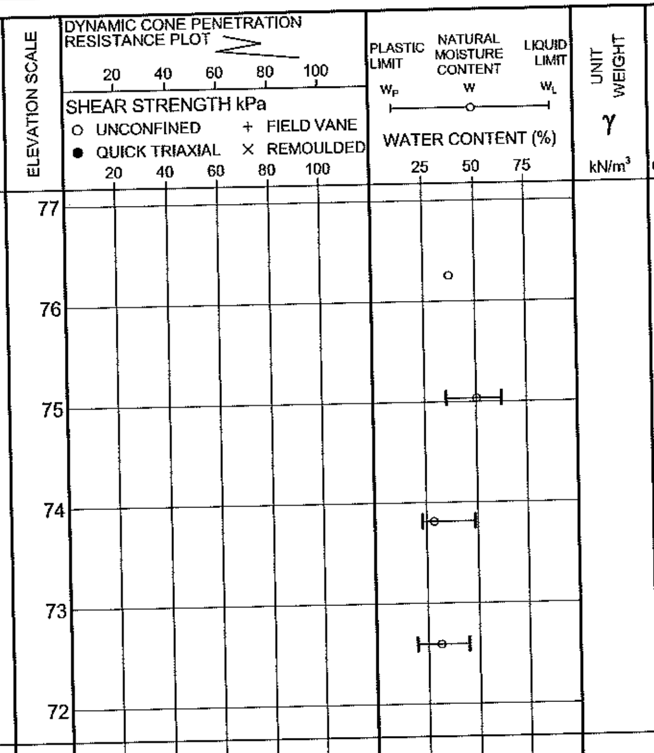
ELEV / DEPTH	DESCRIPTION
77.2	GROUND SURFACE
0.0	Sandy clayey silt, containing rootlets (FILL) Firm Brown to dark brown Moist
76.6 0.6	Silty clay, trace gravel, containing rootlets (FILL) Firm Brown to dark brown Moist
75.5 1.7	Organic SILTY CLAY Firm Grey to black Moist
74.6 2.6	SILTY CLAY, trace to some sand Stiff to very stiff Brown to grey Moist
71.8 5.5	CLAYEY SILT Grey Moist END OF BOREHOLE

NOTE:

1. Open borehole dry upon completion of drilling.

SAMPLES

NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	7		
2	SS	7		
3	SS	6		
4	SS	8		OC = 13.7%
5	SS	11		
6	SS	19		0 8 44 48
7	SS	27		
8	SS	18		
9	SS	16		



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

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

Record of Test Pits (Eastbound Toe of Slope)

26+800 Rt	Test excavation locations inaccessible due to open water	
26+850 Rt		
26+900 Rt		
26+960 Rt		
27 + 020 20.5m Rt	0.0 – 2.4	Grey fine to medium rock fill, some sand and gravel, occasional large rock slab (FILL)
	2.4	End of test pit
		Note: Excavated up to 2.5 m inward from the toe of slope. Moved to excavate 1.0 m inward from the toe of slope (see 27+020, 23m Rt)
27 + 020 23m Rt	0.0 – 1.0	Grey fine to medium rock fill (FILL)
	1.0 – 1.1	Black fibrous PEAT
	1.1 – 2.0	Grey brown SILTY CLAY, trace rootlets
	2.0	End of test pit
		Note: Excavated up to 1.0 m inward from the toe of slope. Large limestone rock slabs at toe of slope. Test pit located +/- 5m east of small CSP culvert. Sample #1: 1.0 – 1.1 m w_n = 291%, o.c. = 53% Sample #2: 1.1 – 1.4 m w_n = 57%
27 + 120 22m Rt	0.0 – 3.4	Grey fine to medium rock fill, occasional rock slabs (FILL)
	3.4	Unstable test pit walls End of test pit
		Notes: Excavated up to 2.5 m inward from the toe of slope. Water seepage at about 2.9 m depth. Test pit located +/- 5-10m east of concrete box culvert.

27 + 350 20.4m Rt	0.0 - 1.7	Grey fine rock fill, trace to some sandy silt and gravel (FILL)
	1.7 – 1.9	Brown to black fibrous PEAT
	1.9 – 2.5	Brown to black fibrous to amorphous PEAT, some silt, trace white shells
	2.5 – 2.6	Grey SILTY CLAY
	2.6	End of test pit
		Note: Excavated up to 2.0 m inward from the toe of slope. Test pit dry upon completion. Sample #1: 1.7 – 1.9 m w_n = 327%, o.c. = 61% Sample #2: 1.9 – 2.1 m w_n = 333%, o.c. = 60% Sample #3: 2.1 – 2.5 m w_n = 144%, o.c. = 27%
27 + 600 16m Rt	0.0 – 0.3	Grey crushed stone (FILL)
	0.3 – 0.53	Brown fine to medium sand, some gravel (FILL)
	0.53 – 0.8	Brown SILTY CLAY, trace sand (possible FILL)
	0.8	End of test pit
		Notes: Excavated up to 1.1 m inward from the toe of slope. Test pit dry upon completion.

Record of Test Pits (Westbound Toe of Slope)

Test Pit Station	Depth (metres)	Description
26 + 800 29m Lt 26 + 795 29 m Lt	0.0 – 0.5 0.5	Fine to coarse rock fill (FILL) Large rock slab at 0.5 m below ground surface. No advancement. Excavator refusal. End of test pit
26 + 850 27 m Lt	0.0 – 1.5 1.5 – 1.8 1.8 – 2.1 2.1	Grey fine to medium rock fill, occasional large rock slabs. (FILL) Dark brown to black fibrous PEAT Light brown silty MARL, trace shells Embedded rock slab near surface limits excavation depth to 2.1 m End of test pit Notes: Excavated up to 1.5 m inward from the toe of slope. Water seepage about 1.0 m depth. Due to water, difficult to see if peat/marl extended under rock fill and under side slopes. Sample #1: 1.5 – 1.8 m w_n = 456%, o.c. = 79% Sample #2: 1.8 – 2.1 m w_n = 117%, o.c. = 6%
26 + 900 26m Lt	0.0 - 0.8 0.8 – 2.0 2.0 – 3.0 3.0	Grey fine to coarse rock fill (FILL) Dark brown amorphous PEAT, contains fibres. Light brown silty MARL, some sand, trace shells Embedded rock slab near surface limits excavation depth to 3 m End of test pit Note: Excavated up to 1.2 m inward from the toe of slope. Water inflow from drainage channel along the toe of slope. Sample #1: 0.8 – 1.3 m w_n = 455%, o.c. = 60% Sample #2: 2.0 – 2.3 m w_n = 196%

Test Pit Station	**Depth (metres)**	**Description**
26 + 960 26m Lt	0.0	Large rock slabs.
		Excavator Refusal End of test pit
		Note: Could not excavate past frozen rock fill and large rock slabs at toe of slope.
27 + 020 23m Lt	0.0 – 0.5	Grey fine rock fill, some sand and gravel (FILL)
	0.5 – 2.3	Brown to black fibrous PEAT, with rootlets
	2.3 – 2.5	Grey SILTY CLAY
	2.5	End of test pit
		Note: Excavated up to 1.2 m inward from the toe of slope.
		Sample #1: 0.5 – 1.0 m w_n = 497%, o.c. = 84% Sample #2: 1.0 – 2.0 m w_n = 518%, o.c. = 87% Sample #3: 2.3 – 2.4 m w_n = 49%
27 + 137 22m Lt	0.0 – 0.5	Grey rock fill (FILL)
	0.5	Excavator refusal in frozen rock fill End of test pit
		Note: Old rock dump at 27+120. Test pit moved 17 m east. Frozen material encountered from ground surface to about 0.5 m depth. Multiple attempts made. No advance.

Test Pit Station	Depth (metres)	Description
27 + 240 21m Lt	0.0 – 1.3	Grey fine to medium rock fill, some silty sand and gravel (FILL)
	1.3 – 2.6	Black to dark brown fibrous PEAT
	2.6 – 2.9	Black silty PEAT, traces of sand, clay and roots
	2.9 – 3.1	Grey SILTY CLAY
	3.1	End of test pit
		Notes: Excavated up to 1.3 m inward from the toe of slope. Test pit dry upon completion.
		Sample #1: 1.3 – 1.5 m w_n = 323%, o.c. = 67% Sample #2: 1.8 – 2.1 m w_n = 366%, o.c. = 53% Sample #3: 2.6 – 2.9 m w_n = 140% Sample #4: 2.9 – 3.1 mw_n = 42%
27 + 350 16.5m Lt	0.0 – 0.2	Black to grey silty sand, some gravel, trace organic matter (FILL)
	0.2 – 0.7	Brown fine to medium sand, trace gravel and cobbles (FILL)
	0.7 – 0.9	Brown SILTY CLAY
	0.9	End of test pit
		Notes: Excavated up to 1.7 m inward from the toe of slope. Test pit dry upon completion.
27 + 400 16.5m Lt	0.0 – 0.3	Grey to black silty sand, trace gravel and organic matter (FILL)
	0.3 – 0.5	Brown sand and gravel, some silt (FILL)
	0.5 – 0.65	Brown, SILTY CLAY
	0.65	End of test pit
		Notes: Excavated up to 1.8 m inward from the toe of slope. Test pit dry upon completion.

Test Pit Station	Depth (metres)	Description
27 + 600 16m Lt	0.0 – 0.1	Grey crushed stone (FILL)
	0.1 – 0.5	Brown silty sand, trace gravel (FILL)
	0.5 – 0.7	Brown SILTY CLAY, trace sand
	0.7	End of test pit
		Notes: Excavated up to 1.1 m inward from the toe of slope. Test pit dry upon completion.

APPENDIX B

Non-Special Standard Provisions
Construction of Sliver Fills
Swamp Excavation
Construction of Geogrid Reinforced Temporary Access Road
Settlement Pins

CONSTRUCTION OF SLIVER FILLS

- Item No.

Special Provision

This NSSP pertains to the placement of sliver fills over the existing embankment slopes, where the horizontal width of the fill beyond the existing embankment slopes is less than 2m.

The contractor shall place embankment fill in general accordance with OPSS 209, OPSS 501 and OPSD 208.010 unless indicated otherwise herein.

Benching into existing embankments shall be required for all sliver fills as indicated in OPSD 208.010. The bench width shall extend no less than 300mm and no more than 1.5m into the existing embankment.

Fill material shall consist of rock fill with a maximum particle size of 200 mm. Fill material shall be placed from the base of the slope (i.e. from temporary access road) or from the crest of the embankment using an excavator or equivalent. Fill material placed in sliver fills shall not be end-dumped from a height greater than 2m. Fill material placed in sliver fills shall be compacted using the excavator bucket or better, until the subgrade elevation is reached or the width is sufficient for conventional compaction equipment to work.

At and above the subgrade elevation, the fill material shall be compacted using conventional heavy roller compactors.

Basis of Payment

The contract price for embankment rock fill shall include the cost for placement of sliver fills, including all labour, equipment, and material.

END OF SECTION

SWAMP EXCAVATION

Item No.

Operational Constraint

This NSSP pertains to the removal of peat deposits at the toe of both existing eastbound and westbound embankment slopes between Stations 26+950 and 27+500 and between Stations 28+200 and 28+440. The work shall be performed in accordance with OPSS 209 and OPSD 203.020, unless otherwise indicated herein.

In these areas, native peat deposits shall be removed from the toe of the embankment cut (defined by a line drawn downwards at one horizontal to one vertical (1H:1V) from the crest of the existing highway to the swamp elevation at the toe of slope) to the toe of the proposed new embankment fill, as indicated on Drawing 5.

For embankment fill heights exceeding 5 and 7 metres, the excavation length of the 1H:1V embankment slopes (as measured parallel to the highway direction) should not exceed 200 m and 100 m, respectively, for periods not exceeding 6 weeks.

The removals shall be completed to a maximum depth of 3m below ground surface at the toe of the existing embankment.

The contractor shall consider that the peat removals shall largely be completed in submerged conditions. Temporary drainage of excavations is not permitted.

For subexcavation depths greater than 1 metre depth below the original ground surface at the embankment toe, the excavation shall, at no time, be longer than 3m, as measured at the base of the excavation and parallel to the roadway.

Excavation slopes below ground surface will be carried out near vertical to remove as much organics as possible within the footprint of the widening while maintaining a stable excavation.

Excavation and backfilling operations should be carried out simultaneously in a manner that the excavation is not left open for more than 3 m in length at any given time.

The excavation shall be backfilled with rock fill materials in accordance with OPSS 209.

Some distress to the existing highway embankment may occur during the staged excavation. Provisions for traffic control measures must be included in the Contract to maintain the safe operation of Highway 401 during excavation and backfilling operations.

END OF SECTION

CONSTRUCTION OF GEOGRID REINFORCED TEMPORARY ACCESS ROAD FROM STATION 26+700 to 26+925

Item No.

Special Provision

This NSSP pertains to the design and construction of the temporary access road over peat deposits left in-place in Sections A and B (Station 26+700 to 26+950) at the toe of both the eastbound and westbound lane embankment slopes.

In these areas, removal of the existing peat is not permissible as it adversely impacts the stability of the existing roadway embankment.

The work included in this bid item shall be:

- Surficial removal of trees and stumps greater than 25mm in diameter.
- Placement of a layer of bi-axial geogrid to act as reinforcement and separation
- Placement of an initial layer of Granular B, Type II with a minimum thickness of 300mm.
- Placement of at least one layer of uni-axial geogrid to act as reinforcement.
- Placement of additional layers of uni-axial geogrid, as necessary, and Granular B, Type II to achieve a final above grade thickness of 1.0m.
- Placement of at least 150mm of Granular A to provide a suitable driving surface.
- Submittal of a design and supporting calculations for the temporary access road. The design shall outline key assumptions on material properties and equipment loads, method of analysis, thickness of layers, and types and characteristics of geogrids. The design shall be stamped by a professional engineer in Ontario and submitted 14 days prior to commencement of work. The design shall consider the geometric requirements of the temporary access road based on the specific operational considerations of the construction equipment.
- Placement of additional Granular A material, as required, to maintain trafficability and grades during construction, particularly in consideration of the presence of the underlying compressible peat deposits.

Basis of Payment

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of work.

END OF SECTION

SETTLEMENT PINS – Item No.

Non Standard Special Provision

1.0 SCOPE

1.01 General

This non-standard special provision contains the requirements for the supply and installation of the following geotechnical instruments:

- Settlement Pins (SP);
- Survey Benchmarks (BM);

1.02 Purpose

The purpose of the settlement pins and survey benchmarks is to directly monitor settlement of the widened embankment along preload area. Settlement is measured by survey of the top of the pin with reference to stable, non-settling and non-moving benchmarks.

The wait time before pavement construction in the preload embankment area will be controlled by the instrumentation readings.

The requirement for preloading of the embankment is specified elsewhere in the contract documents. The completed embankment shall remain undisturbed until such time as the monitoring shall indicate that a sufficient degree of consolidation of the foundation soil has been achieved. The minimum consolidation period shall be specified elsewhere in the contract. No pavement construction work shall take place until sufficient consolidation has been achieved as determined by the Contract Administrator.

2.0 REFERENCES

2.01 Subsurface Conditions

A Foundation Investigation Report that describes the subsurface conditions for the embankment preload area is available, as specified elsewhere in the Contract. The Ministry warrants that the information provided in the Foundation Investigation Report can be relied upon with the following limitations and exceptions:

Any interpretation of data or opinions expressed in the reports is not warranted.

2.02 Drawings

Reference shall be made to the following drawings:

- Monitoring Instrument Detail – Settlement Pins

3.0 DEFINITIONS

For the purposes of this specification, the following definitions apply:

Embankment Fill: All fill material placed above the original ground surface or above the existing embankment side slope.

Or Equal: The term, 'or equal' shall be understood to indicate the equal product is the same or better than the specified product in function, performance, reliability, quality and general configuration.

4.0 SUBMISSION REQUIREMENTS

4.01 Submissions

The Contractor shall submit to the Contract Administrator details of the proposed installation schedule and installation methods, including locations and types of survey benchmarks, a minimum of 15 working days before the start of settlement pin installation.

4.02 Reporting

The Contractor shall record and report relevant instrumentation installation details to the Contract Administrator. These include, but are not limited to, the following:

- Settlement pin and benchmark location, easting and northing and elevation;
- Dates of installation and reference benchmark descriptions for elevation and position;
- Installation notes / sketches; and
- Description of settlement pin and benchmark.

5.0 MATERIALS

5.01 General

The Contractor shall supply all materials and equipment required for the installation of the settlement pins.

All materials shall be capable of withstanding the range of temperatures possible for their location. Monitoring should be assumed to be conducted year-round as arranged for by the Contract Administrator.

5.02 Concrete

The Contractor shall supply concrete (OPSS 1350) with strength and set time sufficient to secure the settlement pin within two days of placing.

5.03 Pins

The Contractor shall supply a 25.4 mm minimum diameter reinforcing steel bar (OPSS 905) cut to a length of 0.4 m. The top of the reinforcing steel bar shall be angled or rounded in such a way that a single survey point can be clearly identified and repeated.

6.0 EQUIPMENT

6.01 Survey Equipment

The elevation, northing, and easting of the top of the settlement pins shall be surveyed by an experienced, registered surveyor, retained by the Contractor, to provide the datum readings. The surveyor shall provide suitable equipment capable of surveying settlement rod or pin elevations to an accuracy of ± 2 mm or better and position (i.e. northing, easting) to an accuracy of ± 4 mm or better, after installation of the settlement pins (after any cement grout or other bonding agent has set and the settlement pins are firmly secured).

7.0 CONSTRUCTION

7.01 Survey Bench Marks

The Contractor shall provide a minimum of two non-yielding, deep-seated survey bench marks, located outside of the embankment preload area, and shall establish the geodetic elevation and position of each such benchmark.

The number and locations of bench marks shall be such that direct sighting is possible from all settlement pins to at least one bench mark.

7.03 Settlement Pins (SP)

7.03.01 General Procedure

The settlement pins shall be cast into concrete at the top of the widened embankment, as per the attached drawings. The concrete shall be cast in-situ in a hole dug into the Granular 'B' at the following locations.

7.03.02 Location

The approximate locations of the settlement pins are given in Table 1, below, and shown in the Contract Drawings.

Table 1 - Approximate Settlement Pin (SP) Locations

Location	Station	Offset*	Remarks
Highway 401 Eastbound Lanes	26+680 to 26+710	Right	10 m spacing within 35 m of bridge east abutment (4 pins)
	26+725 to 26+800	Right	25 m spacing within area of widening (4 pins)
	26+850 to 27+500	Right	50 m spacing within area of widening (14 pins)
	28+200 to 28+450	Right	50 m spacing within area of widening (6 pins)
Highway 401 Westbound Lanes	26+720 to 26+740	Left	10 m spacing within 25 m of bridge east abutment (3 pins)
	26+750 to 27+825	Left	25 m spacing within area of widening (4 pins)
	26+850 to 27+300	Left	50 m spacing within area of widening (10 pins)
Total	**45 Settlement Pins**		

Notes: (*) Offset distance from centerline will depend on the width of the widened embankment; pins to be located 1 m north (EB lanes) or south (WB lanes) of the crest of the widened fill slope.

7.03.03 Installation

The Contractor shall install settlement pins as per the detail provided.

Settlement pins are to be installed immediately after the widened embankment has been constructed to the top of the Granular 'B' pavement structure.

7.03.04 Marking and Labelling

Settlement pins shall be clearly labelled in the field, each instrument having a unique identifier. Labelling shall remain legible for the entire period of monitoring.

The location of settlement pins shall be made clearly visible to nearby traffic before, during, and after embankment construction. Marking shall be of sufficient size to be visible from a reversing vehicle and after heavy snow falls.

7.03.05 Surveying of Instruments

Within 2 days of installation, the elevation, easting and northing of the centre of the top of the settlement pins shall be surveyed by a registered surveyor, retained by the Contractor.

7.03.06 Protection of Instruments

The Contractor shall adequately protect all settlement pins and benchmarks such that they are not damaged during construction or by vandalism or construction traffic, throughout the monitoring period. Any instrument damaged by the Contractor's work shall be immediately replaced at the Contractor's cost.

7.04 Ongoing Settlement Monitoring By Others

7.04.01 Personnel/Access

After completing installation of the instruments, the ongoing data collection, interpretation and reporting shall be conducted by others, under the direction of the Contract Administrator. However, the Contractor shall provide access to and assistance to others reading all s during the on-going monitoring. This may include, but not necessarily be limited to, the following: safe access to each instrument location, snow clearing (if required), a stable platform to support the technician and equipment to access instrumentation during construction.

7.04.02 Monitoring Program

The Contractor shall meet with the Contract Administrator and staff responsible for the ongoing monitoring immediately after installation of the instruments. At this meeting, the Contractor shall hand over to the Contract Administrator all records pertaining to the installation of the instruments and any equipment to be supplied by the Contractor.

Monitoring by others for the baseline readings shall commence within seven working days after the "hand-over" meeting. The monitoring shall continue on a schedule to be determined by the Contract Administrator throughout the completion of the embankment preload and for approximately six months following the completion of construction of the preload.

8.0 MEASUREMENT FOR PAYMENT

Measurement of the item, “Settlement Pins”, including all appurtenances, is by quantity. The unit of measurement is each.

9.0 BASIS FOR PAYMENT

Payment of the contract price for the item “Settlement Pins” shall be full compensation for all labour, equipment and material to do the work, including the establishment of the required benchmarks and surveying required to establish the locations and initial base line elevations for each settlement pin, and the required reporting.