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Golder Associates

REPORT ON

FOUNDATION INVESTIGATION AND DESIGN
EB CN RAIL OVERPASS EAST AND WEST APPROACH
EMBANKMENT WIDENING
HIGHWAY 417
W.P. 258-98-00
CASSELMAN, ONTARIO

Submitted to:

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FOUNDATION INVESTIGATION AND DESIGN
EB CN RAIL OVERPASS EAST AND WEST APPROACH
EMBANKMENT WIDENING
HIGHWAY 417
W.P. 258-98-00
CASSELMAN, ONTARIO

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by Morrison Hershfield (MH) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out a foundation investigation as part of the detailed design for the upgrading of Highway 417 between the Limoges Road and Casselman Road interchanges.

The terms of reference for the scope of work are outlined in Golder Associates Ltd. proposal numbered P41-2114, dated January 11, 2005, that forms part of the Consultant's Agreement (Number P.O. 4005-A-000316) for this project. A scope change related to additional borehole investigation work at the high fill embankments for the S-E ramp at the Casselman Road interchange, and extending along the EBL CN Rail Overpass approach embankments, is outlined in Golder Associates' proposal dated January 11, 2005.

This report addresses the proposed widening, described above, of the east and west approach embankments to the EBL CN Rail Overpass near Casselman, Ontario, in relation to the new S-E ramp from the Casselman Road interchange. The work was carried out in accordance with the Quality Control Plan for this project dated February 2004.

2.0 SITE DESCRIPTION

The Casselman Road interchange is located approximately 50 kilometres southeast of Ottawa. That interchange is to be provided with a new S-E Ramp. That construction of that ramp will require the widening to the south of the east and west approach embankments (and structure) of the EBL CN Rail Overpass, which is located only about 800 m east of the Casselman Road interchange. At its maximum height (i.e., adjacent to the structure), those approach embankments are about 9.5 high.

The land to the south of the existing embankment is generally either agricultural or fallow. At the time of the investigation the site was snow covered. The site is generally flat-lying and varies between about Elevation 65 m to 66 m along the investigation area.

Golder Associates has carried out a foundation investigation for the proposed widening of the EBL CN Rail Overpass structure. The results of that investigation have been provided in Golder Associates report (still in draft version) to Morrison Hershfield titled "*Foundation Investigation and Design, CN Rail Overpass Widening, Highway 417 Eastbound, Structure Site 27-213/7, W.P. 258-98-00, Casselman, Ontario*" dated August 2004 (report number 04-1120-013-5000).

The results of that previous foundation investigation completed for the structure widening indicate subsurface conditions generally consisting of stiff silty clay overlying glacial till with the bedrock surface at about 5 to 6 m depth. However published geologic mapping indicate a thicker deposit of silty clay to the east of this site.

3.0 INVESTIGATION PROCEDURES

The field work for this investigation was carried out between February 18 and February 28, 2005. During this time, a total of nineteen (19) sampled boreholes were advanced adjacent to the toe of the existing eastbound lane approach embankments, between about Stations 23+825 and 24+700, at an approximately 50 metre spacing. Boreholes 05-201 to 05-209, inclusive, and 05-217 were advanced along the east approach embankment. Boreholes 05-210 to 05-216, including 05-211A and 05-215A, were advanced along to the west approach embankment. Boreholes 05-211A and 05-215A were supplemental boreholes, put down in close proximity to boreholes 05-211 and 05-215, respectively, to retrieving Shelby tube samples from specific depth intervals within the silty clay.

In addition to the boreholes, four shallow test pits (numbered 05-218 to 05-221, inclusive) were excavated into the flank of the embankment at about Stations 23+900, 24+100, 24+420, and 24+620.

The boreholes were advanced using 108 mm inside diameter (I.D.) continuous flight hollow stem augers on a track-mounted drill rig, supplied and operated by Marathon Drilling Ltd. of Ottawa, Ontario. The boreholes were generally advanced to depths ranging from 2.3 to 8.1 metres below the existing ground surface prior to encountering auger refusal or being terminated within the glacial till.

Soil samples were obtained at intervals ranging from 0.75 m to 1.5 m of depth, using a 50 mm outer diameter (O.D.) split-spoon sampler in accordance with Standard Penetration Test (SPT) procedures. In-situ vane testing (N vanes) was carried out within the cohesive deposits. Relatively undisturbed, 75-millimetre diameter thin-walled Shelby tube (ASTM D1587) samples of the silty clay where retrieved using a fixed piston sampler.

The water levels in the open boreholes were noted upon completion of the drilling operations.

Standpipe piezometers were installed in boreholes 05-203, 05-206, 05-209, 05-211A, and 05-215A to monitor the groundwater levels at the site. The standpipes consist of 50 mm diameter rigid PVC pipe with a 1.5 m long slotted screen section, installed within silica sand backfill and sealed below minimum 0.3 m long sections of bentonite pellet backfill. The water levels in the standpipe piezometers were measured on March 11, 2005.

The boreholes were backfilled with bentonite pellets, mixed with native soils, and the site conditions restored following completion of work. The standpipe piezometers have not as yet been decommissioned.

The four test pits were excavated using a rubber tired backhoe supplied and operated by Gagne Construction Ltd. of Casselman, Ontario. The test pits were excavated at a height of about 4 m above the embankment toe, and were extended to a depth of about 2 m into the embankment. The soils exposed on the sides of the test pits were classified by visual and tactile examination. Chunk samples were obtained from the major soil strata encountered in the test pits. The test pits were backfilled upon completion of excavating and sampling.

The field work was supervised throughout by members of our engineering and technical staff, who located the boreholes and test pits, supervised the drilling, excavating, sampling and in-situ testing operations, logged the boreholes and test pits, and examined and cared for the soil samples. The samples were identified in the field, placed in appropriate containers, labelled and transported to our Ottawa geotechnical laboratory where the samples underwent further detailed visual examination and laboratory testing, including grain size distribution, water content, and Atterberg limit testing. Laboratory oedometer consolidation testing was carried out on four of the Shelby tube samples. All of the laboratory tests were carried out to MTO and/or ASTM Standards as appropriate.

The borehole and test pit locations were selected by Golder Associates personnel. The ground surface elevations at the borehole and test pit locations were provided by Morrison Hershfield and are understood to be referenced to Geodetic datum.

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 and 2.

Borehole / Test Pit No.	Borehole / Test Pit Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
05-201	East approach embankment	5019183.46	417080.81	65.3
05-202	East approach embankment	5019182.29	417101.19	65.3
05-203	East approach embankment	5019191.69	417144.93	65.6
05-204	East approach embankment	5019201.03	417189.78	65.4
05-205	East approach embankment	5019216.06	417238.11	65.5
05-206	East approach embankment	5019226.89	417286.63	65.3
05-207	East approach embankment	5019238.62	417335.66	65.2
05-208	East approach embankment	5019251.26	417395.22	65.6
05-209	East approach embankment	5019264.91	417434.18	64.9
05-210	West approach embankment	5019159.59	416927.45	65.6
05-211	West approach embankment	5019150.83	416870.26	65.4
05-211A	West approach embankment	5019150.83	416870.26	65.4
05-212	West approach embankment	5019143.23	416821.27	65.3

Borehole / Test Pit No.	Borehole / Test Pit Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)
05-213	West approach embankment	5019139.00	416772.34	65.4
05-214	West approach embankment	5019133.98	416722.71	65.1
05-215	West approach embankment	5019130.12	416673.05	65.2
05-215A	West approach embankment	5019130.12	416673.05	65.2
05-216	West approach embankment	5019123.67	416622.45	65.0
05-217	East approach embankment	5019276.92	417482.21	64.8
05-218	West approach embankment	5019145.22	416695.62	68.8
05-219	West approach embankment	5019168.88	416895.31	69.4
05-220	East approach embankment	5019224.43	417206.47	69.8
05-221	East approach embankment	5019269.69	417407.26	67.9

4.0 SITE GEOLOGY AND STRATIGRAPHY

4.1 Regional Geological Conditions

The study area for this assignment is within the minor physiographic region known as the Ottawa Valley Clay Plain that lies within the major physiographic region of the Ottawa-St. Lawrence Lowland (Chapman and Putnam, 1984). This physiographic region is underlain primarily by limestones of the Ottawa Formation that are, in turn, underlain by a series of sedimentary rocks, consisting of sandstones, dolostones, limestones and shales. These sedimentary formations are underlain by igneous and metamorphic bedrock of the Precambrian Shield. The Ottawa Valley Clay Plain region, present along Highway 417 in this area, is characterized by relatively thick deposits of sensitive marine clay, silt and silty clay that were deposited within the Champlain Sea basin. These deposits, known as the Champlain Sea clay or Leda clay, overlie relatively thin, commonly reworked glacial till and glaciofluvial deposits, that in turn overlie bedrock (Chapman and Putnam, 1984). The current study area is located within a small surficially discontinuous region of the Ottawa Valley Clay Plain. This area lies within an abandoned channel of the South Nation River and the silty clays have been mostly removed by fluvial erosion to expose a till plain. Thin layers of clay and silt overlie the glacial till in some portions of the study area.

4.2 Site Stratigraphy

The detailed subsurface soil, bedrock and groundwater conditions as encountered in the boreholes and test pits 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 sheets and the Record of Test Pits (Table 1). The stratigraphic boundaries shown on the Record of Borehole sheets are inferred from non-continuous sampling, observations of drilling progress and the results of Standard Penetration Tests (SPTs). These boundaries, therefore, represent transitions between soil types rather than exact planes of geological change. Further, subsurface conditions will vary between and beyond the borehole and test pit locations.

In general, the subsurface conditions consist of a thin veneer of layered silty sand, silt, and clayey silt overlying sensitive silty clay. The upper zone of the silty clay, or the full thickness within the central portion of the site, has been weathered to a stiff to very stiff grey brown crust and varies in thickness from about 0.5 to 2.5 m. The silty clay below the depth of weathering, typically 0.5 to 5.0 m thick, is grey in colour and has a soft to firm consistency. Glacial till underlies the silty clay at most locations and, where present, was proven to extend to depths varying between 3.5 to 8 m.

A more detailed description of the subsurface conditions encountered in the boreholes and test pits put down for the present investigation, as well as in boreholes 04-105 and 04-107 put down during Golder's previous investigation is provided in the following sections, and stratigraphic profiles and sections of this site are shown on Drawings 1 and 2.

4.2.1 Embankment Fill Materials

A topsoil dressing exists at ground surface along the embankment side slope and varies from about 0.2 to 0.3 m in thickness. Underlying the topsoil, the embankment material consists predominantly of layered fine to medium sand, with variable amounts of silt and gravel.

In test pits 05-218 and 05-219, red brown and grey brown silty clay was also encountered within the embankment fill. . In test pit 05-219, the silty clay layer was only about 0.2 m thick, however in test pit 05-218, the silty clay extended to a depth of at least 2 metres into the embankment.

4.2.2 Fill Material and Topsoil

Fill material exists at ground surface in Boreholes 05-208, 05-213, and 04-107 as well as buried beneath the topsoil in borehole 04-105. The fill material varies from about 0.3 to 0.6 m thick. The fill material is variable in composition consisting of intermixed topsoil, sand, silt, and clay

Topsoil exists buried beneath the fill material in Boreholes 05-208, 05-213, and 04-107 and at ground surface in all of the remaining boreholes with the exception of borehole 04-210 where no topsoil exists. The thickness of the topsoil ranges from approximately 0.1 to 0.4 m.

4.2.3 Shallow Sandy Silt, Silt, and Silty Sand Deposit

The topsoil in Boreholes 04-107, 05-201, 05-203, 05-204, 05-206, and 05-210 to 05-214 (inclusive) is underlain by a discontinuous surficial deposit of generally silty sand, silt, and sandy silt. This deposit ranges from about 0.2 to 0.6 m in thickness. The results of grain size distribution testing carried out on one sample from this deposit are provided on Figure 1 and indicate that particular sample to be a silty gravel with a trace of clay (gap graded), although that result is not considered reflective of the overall deposit.

4.2.4 Silty Clay and Clayey Silt

The surficial deposits of topsoil as well as the shallow sandy silt, silt, and silty sand (where present) are underlain by a deposit of silty clay. In Boreholes 05-204 and 05-215 portions of the deposit are considered to be a clayey silt.

The deposit (combined clayey silt and silty clay) varies substantially in thickness, ranging from 0.2 m at borehole 04-210 to about 5.9 m at borehole 05-217. The silty clay and clayey silt deposit typically increases in thickness towards the east and the west of the investigation limits.

Along portions of the investigation, particularly where the thickness of the clay deposit is least, the full thickness of silty clay and clayey silt has been weathered to a grey brown crust. Where the clay is thicker, only the upper portion of the deposit has been weathered and the underlying un-weathered portions are grey in colour. The varying conditions along the embankment are summarized below. However, in general, standard penetration tests carried out within the weathered crust gave N values ranging from 2 to 9 blows per 0.3 m of penetration. The results of in situ vane testing in the lower portions of the weathered crust gave undrained shear strengths ranging from 50 to 59 kPa. These results indicate a stiff to very stiff consistency for the weathered crust.

In the underlying un-weathered silty clay, where present, standard penetration test N values ranged from 'weight of hammer' to 2 blows. The results of in situ vane testing in this material gave undrained shear strengths ranging from a 19 to 42 kilopascals just below the weathered crust, increasing with depth to 40 to 62 kilopascals near the bottom of the deposit, indicating an overall soft to stiff consistency. In situ vane testing carried out on remoulded silty clay gave undrained shear strengths ranging from 2 to 14 kilopascals, with corresponding sensitivities ranging from 2 to 13. A summary of the results of the in situ vane testing is provided on Figure 10.

A more detailed summary of the silty clay and clayey silt stratum along the embankment is as follows:

<u>Stations 23+825 (west limit) to 23+900: Boreholes 05-216 and 05-215</u>

The silty clay and clayey silt has been weathered to a depth of about 1.8 and 1.7 m, respectively. The underlying grey silty clay and clayey silt is about 2.4 and 2.0 m in thickness, respectively (apparently thickening to the west). The undrained shear strengths range from about 19 to 38 kilopascals just below the weathered crust and increase to 50 to 52 kilopascals near the bottom of the deposit.

Stations 23+900 to 23+950: Borehole 05-214

The full deposit of the silty clay, approximately 1.7 m in thickness, has been weathered.

Stations 23+950 to 24+100: Boreholes 05-213, 05-212, and 05-211

The silty clay has been weathered to depths varying from 1.4 to 2.1 m. The underlying unweathered silty clay varies in thickness from about 0.6 to 1.1 m and its undrained shear strength ranges from about 27 to 40 kilopascals, which indicates a firm consistency.

Stations 24+100 to 24+200 (at the west bridge abutment): Boreholes 05-210, 04-107, and 04-105

The full deposit of the silty clay has been weathered and ranges in thickness from about 0.2 to 1.1 m.

Stations 24+250 (at the east bridge abutment) to 24+300: Borehole 05-201

The silty clay has been weathered to a depth of 2.3 m. The underlying grey silty clay is about 1.7 metres thick and its undrained shear strength ranges from about 23 to 32 kilopascals, indicating a soft to firm consistency.

Stations 24+300 to 24+575: Boreholes 05-202, 05-203, 05-204, 05-206, and 05:207

The full thickness of the silty clay been weathered and ranges from about 0.5 to 2.1 m in thickness.

Stations 24+575 to 24+700 (east limit) - Boreholes 05-208, 05-209, and 05-217

The silty clay has been weathered to a depth of about 1.4 to 2.0 metres. The underlying grey silty clay ranges from about 2.0 m to 5.0 m in thickness (thickening to the east) and its undrained shear strength generally ranges from about 21 to 42 kilopascals (soft to firm)..

The results of grain size distribution testing carried out on four samples of this deposit are provided on Figure 2 and indicate it to be generally a silty clay to clayey silt with trace to some sand.

Atterberg Limit testing carried out on eight samples of the silty clay (including samples of both the weathered and unweathered clay, which yielding consistent results) gave plasticity index values ranging from 23 to 56 percent and liquid limit values ranging from 49 to 82 percent, but more typically of about 52 to 66 percent. These results are summarized on the Plasticity Chart, Figure 3, and indicate a material ranging from silty clay of intermediate plasticity to clay of high plasticity. The measured water content of the samples of the unweathered grey silty clay ranges from approximately 55 to 89 percent, which is generally at or above the measured liquid limit. The measured water content of the weathered crust typically ranges from 22 to 59 percent and is at or below the liquid limit value. The Atterberg limit and water content data are summarized on Figure 10.

Laboratory oedometer consolidation tests were performed on four samples from this stratum. The results are summarized below.

Borehole (Sample)	Elevation (Depth) (m)	σ_{vo}' (kPa)	σ_p' (kPa)	OCR	σ_p' - σ_{vo}' (kPa)	e_o	C_r	C_c
05-201 (3)	61.9 (3.4)	40	105	2.6	65	1.89	0.018	1.35
05-209 (3)	62.3 (2.6)	30	75	2.5	45	1.47	0.023	0.70
05-211A (1)	62.8 (2.6)	40	170	4.3	130	2.33	0.020	2.49
05-216 (3)	62.3 (2.7)	35	65	1.9	30	1.65	0.016	0.93

where : σ_{vo}' is the calculated effective overburden pressure in kPa
σ_p' is the pre-consolidation pressure in kPa
OCR is the overconsolidation ratio (σ_p'/σ_{vo}')
σ_p' - σ_{vo}' is the available overconsolidation
e_o is the initial void ratio
C_r is the recompression index
C_c is the compression index

Summaries of the results of the above testing are provided on Figures 4 to 7. As noted in the above table, the magnitude of overconsolidation (the difference between the measured preconsolidation pressure and calculated existing effective stress level, **σ_p'- σ_{vo}'**) is indicated to range from about 30 to 130 kPa. A summary of the consolidation properties is also provided on Figure 10.

A summary of the coefficient of consolidation (c_v) data from the laboratory oedometer consolidation testing is provided on Figure 11. These results indicate that the coefficient of consolidation of the deposit at stress levels below the preconsolidation pressure typically ranges from about 2 x 10^{-3} to 2x 10^{-2} cm^2/s. Above the deposit's preconsolidation pressure, the coefficient of consolidation is indicated to be about 10^{-3} to 10^{-4} cm^2/s. It is noted however that relatively small load increments (i.e., not a doubling of each load step, as is conventionally the case) needed to be used for the consolidation tests on these clay samples since the material is quite sensitive and therefore smaller load increments are needed to properly define the sharp break in the load-deformation curve which identifies preconsolidation pressure. Those smaller load increments can result in somewhat of an underestimation of the coefficient of consolidation, since the excess pore pressures generated during each load increment are relatively smaller than if conventional load increments are used. Therefore the actual coefficient of consolidation is likely near the higher end of the ranges specified above.

4.2.5 Deep Sandy Silt, Silt, and Silty Sand Deposit

A relatively thin deposit of sandy silt underlies the silty clay and clayey silt in the following locations:

- Along the west embankment, at Boreholes 05-213 (Station 23+975) and 04-105 (Station 24+225) only

- Along the east embankment, essentially from Stations 24+300 to 24+500 (Boreholes 05-202 to 05-206, inclusive, with the exception of 05-204 where the layer was absent) and then again at the east limit (Borehole 05-217)

This layer was fully penetrated by the individual boreholes and varies between 0.1 and 1.5 m in thickness. The results of grain size distribution testing carried out on two samples of this deposit are provided on Figure 8 and indicate those materials to range in composition from silt with some sand to a silty sand, with a trace of gravel and clay. Standard penetration test N values of 6 to 21 blow per 0.3 m of penetration indicate this deposit to be loose to compact.

4.2.6 Sandy Silt Till

Glacial till underlies the silty clay and deep sandy silt/silt/silty sand deposits in all of the boreholes with the exception of boreholes 05-214 and 05-216 where auger refusal was encountered immediately upon penetrating the silty clay deposit.

The glacial till was penetrated to depths ranging from about 3.6 m to 8.1 m (i.e. elevation 56.7 to 61.7) below ground surface.

The results of grain size distribution testing carried out two samples of the glacial till are provided on Figure 9. Those results of the testing indicate the material to be a silt with sand, gravel, and a trace of clay, which is consistent with glacial tills in this area. Grinding on cobbles and boulders within the glacial till where noted during the drilling operations.

Standard penetration test 'N' values for this material ranging from 4 to greater than 100 blows per 0.3 metres of penetration indicate a loose to very dense state of packing. However the higher N values may reflect impact of the sampler on cobbles and boulders.

The measured natural water content of samples of the glacial till ranges from approximately 8 to 12 percent.

4.2.7 Auger Refusal

Practical refusal to augering was encountered in all of the boreholes along the west approach embankment at depths varying from 2.3 to 6.1 (i.e. elevation 59.4 to 62.8). Borehole 05-210 penetrated apparent weathered bedrock for about 0.4 metres before practical refusal to augering was encountered.

Along the east approach embankment, practical refusal to augering was encountered in boreholes 05-201, 05-202, and 05-203, and 05-206 at depths ranging from approximately 4.4 to 6.2 metres below the existing ground surface (i.e. elevations 59.4 metres to 60.9 metres).

4.3 Groundwater Conditions

The piezometers in Boreholes 05-203 and 05-206 were installed within the glacial till deposit. The piezometers in Boreholes 05-209, 05-211A, and 05-215A were sealed into the silty clay. The groundwater level in the standpipes were measured on March 11, 2005 and again on April 21, 2005. The observations are summarized in the following table:

Borehole No.	Water Level on March 11, 2005		Water Level on April 21, 2005	
	Elevation (m)	Depth (m)	Elevation (m)	Depth (m)
05-203	64.2	1.4	65.3	0.26
05-206	64.1	1.2	64.7	0.63
05-209	63.9	1.0	64.8	0.10
05-211A	63.3	2.1	64.5	0.95
05-215A	63.0	2.2	64.2	0.98

Although none of the boreholes were completed with multi-level standpipe installations, there is no evidence of a vertical hydraulic gradient between the silty clay and the underlying glacial till.

It should be noted that groundwater levels are expected to fluctuate seasonally and are expected to rise during wet periods of the year.

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PART B

FOUNDATION INVESTIGATION AND DESIGN
EB CN RAIL OVERPASS EAST AND WEST APPROACH
EMBANKMENT WIDENING
HIGHWAY 417
W.P. 258-98-00
CASSELMAN, ONTARIO

5.0 ENGINEERING RECOMMENDATIONS

5.1 General

This section of the report provides foundation design recommendations for the proposed high fill embankment widening of the east and west approach embankments to the EB CN Rail Overpass structure, to be carried out in conjunction with the construction of a new S-E ramp at the Casselman Road interchange, as part of the overall upgrading of Highway 417 between Limoges and Casselman. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the subsurface investigation at this site.

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

5.1 General

The project involves the construction of a new S-E ramp at the Casselman Road interchange which will require the widening of the approach embankments and structure for the EB CN Rail Overpass. At its maximum height (i.e., adjacent to the structure), those approach embankments are about 9.6 high. The current investigation and report addresses those locations where the existing embankments are more than about 4.5 m high.

The widening will typically be about 1 lane wide (about 4 m). However, at the west end of the alignment, where the new ramp joins the EB lanes, the widening will be as much as about 10 m. At the east limit of the investigation, near the end of the ramp taper, the widening will be les than 1 m wide.

5.2 Embankment Widening

5.2.1 Subgrade Preparation and Embankment Construction

All topsoil and organic matter should be stripped from the embankment side slope and from the subgrade of the embankment widening.

Where granular soils (e.g., sand and silt) are exposed at subgrade level, the subgrade should be proof-rolled prior to fill placement; clayey subgrade soils should not be proof rolled.

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

The new embankment fills should be benched into the existing embankment in accordance with OPSD 208.010.

5.2.2 Approach Embankment Stability

It is understood that the desired side slope inclination for the embankment widening is 2 horizontal to 1 vertical (2H:1V). Flatter side slopes would require property acquisition, at least for the widening of the higher embankment areas where the existing embankment toe is closest to the limit of the right-of-way. The existing embankment side slopes are generally somewhat flatter than are proposed for the widening, being typically inclined no steeper than about 2.2H:1V, except near the bridge itself, where the embankments are about 9 m high and the upper half of the slope is inclined at 2H:1V, while the lower part is inclined at about 2.5H:1V.

Static slope stability analyses for this embankment configuration were carried out using the following parameters:

Soil Deposit	Bulk Unit Weight (kN/m^3)	Effective Friction Angle (degrees)	Undrained Shear Strength
Embankment Fill	20.5	32	–
Shallow sandy silt	19.5	28	
Weathered Silty Clay Crust	17.5	–	75 kPa
Unweathered Grey Silty Clay	15.7	–	Varies : 14-32 KPa
Deep sand/silt and sandy silt till	Impenetrable		

The unit weights of the weathered silty clay crust and the unweathered silty clay were inferred from the measured water content data for these deposits.

Due to the significant difference in shear strength and stress-strain response of the (marine) silty clay versus the underlying sand/silt and granular till deposits, the failure surface for a deep-seated instability of the embankment is unlikely to penetrate those lower deposits and these strata were therefore treated as being impenetrable.

The analyses were carried out for undrained (i.e., short-term) conditions, which represents the critical condition experienced during and immediately following construction of the widening. With time, the excess pore water pressures generated in the silty clay deposit as a result of the additional loading would dissipate and 'drained' conditions would exists, with a higher factor of safety against instability.

The mobilized/available undrained shear strength of the unweathered silty clay was inferred for each analyzed cross section from the results of the in situ vane testing and from the following correlation between the mobilized undrained shear strength at failure (Su) and the preconsolidation pressure $Su = 0.22\ \sigma_p'$. It should be noted however that for the higher sections of existing embankment the calculated effective stress level within the silty clay deposit beneath the slope of the existing embankment exceeds the measured preconsolidation pressure; the laboratory oedometer consolidation tests were all carried out on samples taken from boreholes put down outside of the existing embankment footprint. It is therefore considered that the clay beneath the existing embankment slopes (which have been in-place for about 30 years and therefore fully consolidated) has been locally pre-loaded above its original preconsolidation pressure. Therefore, where appropriate, the assessment of the undrained shear strength of the unweathered silty clay that could be mobilized at failure, based on the relation $Su = 0.22\ \sigma_p'$, was made with σ_p' being the calculated existing effective stress level beneath the embankment side slope (i.e., the pre-loaded clay has now been normally consolidated to the embankment load). Noting that only that portion of the failure surface passing beneath the higher portion of the existing slope would penetrate clay that has been pre-loaded in this manner. The average undrained shear strength along the failure surface was therefore approximately pro-rated based on the length of failure surface that has been pre-loaded and that which has not.

The analyses also, conservatively, treated the entire embankment as 'new' construction, rather than recognizing just the additional loading from the embankment widening. This is a conservative assessment because the magnitude of the loading from the widening alone would be much less and, in particular, the pore water pressure increases in the unweathered silty clay would also be much less. The actual impact on the stability of the embankment resulting from the widening alone is difficult to model realistically using conventional limit equilibrium methods. However, as described below, the stability analyses indicate a factor of safety against deep-seated instability of the embankment under static conditions of at least 1.3, which is acceptable.

The stability of the widened embankment was also evaluated under seismic loading conditions, with the proposed 2H:1V geometry. Those analyses indicate that, although some shallow sloughing could occur of the embankment side slopes during seismic loading, the factor of safety against deep-seated instability of the embankment, through the unweathered clay deposits, would be 1.1 (the minimum value that is typically required), or greater. A horizontal seismic coefficient of 0.1 was used for the analyses, consistent with the zonal acceleration ratio for this area of 0.2. In addition, and where appropriate, a strength increase for the soils of up to 10 percent was considered, noting that most soils exhibit at least that amount of additional strength when subjected to the very rapid loading generated during a seismic event.

Noting that there are only discrete lengths of the embankment that are underlain by the weaker unweathered clay deposits, the critical embankment locations selected for the stability analyses were based on the presence and thickness of that clay deposit, its measured strength, and the height of the embankment being widened. The results of those analyses are summarized below.

Station / Borehole	Embankment Height (m)	Undrained shear strength of unweathered silty clay (kPa)	Factor of safety – static conditions	Factor of safety – seismic conditions
Sta. 23-825 BH 05-216	4.7	14.3	2.0	1.54
Sta. 24+075 BH 05-211	8.1	30	1.5[1]	1.13[1]
Sta. 24+275 BH 05-201	9.6	31.7	1.3	1.10
Sta. 24+600 BH 05-208	5.4	15	1.4[2]	1.17[2]
Sta. 24+700 BH 05-217	3.8	15.4	1.7	1.1

Note: 1 – Graphical illustration of the slope geometry, soil parameters, and failure surfaces at Station 24+075 (BH 05-211) are provided on Figures A1 and A2 in Appendix A.

2 – Graphical illustration of the slope geometry, soil parameters, and failure surfaces at Station 24+600 (BH 05-208) are provided on Figures A3 and A4 in Appendix A.

It is therefore considered that, with appropriate subgrade preparation and proper placement and compaction of embankment fill materials, the up to 9.6 m high widened embankments with side slopes maintained at up to (2H:1V) will have an acceptable factor of safety of at least 1.3 against deep-seated slope instability for the undrained conditions during and immediately after embankment construction and at least 1.1 under seismic loading conditions.

Where the approach embankment height is equal to or greater than 8 m, a mid-height berm at least 2 m in width is required for maintenance purposes. To reduce surface water erosion on the embankment side slopes, placement of topsoil and seeding or pegged sod is recommended.

5.2.3 Embankment Settlements

The settlements of the widened embankment will result from both compression of the native soils and from compression of the embankment fills themselves.

Compression of the embankment fill is expected to be less than 25 mm, provided that the embankment material consists of select subgrade material or clean earth fill and is compacted as described above. The use of granular fill for the embankment construction would reduce this magnitude of embankment settlement since the majority of the settlement of granular fills will occur during construction, whereas the majority of the settlement of cohesive fill, if used, could occur after construction.

As described previously, the existing embankment ranges up to about 9.6 m in height. However the actual height of the widening will be somewhat less, in that the widening is largely being built onto the existing side slope. However the width of the widening of the top of the embankment is not consistent along the project length, being greater at the west end of the project, where the new ramp will join the eastbound lanes, and least at the east end of the project, at the end of the ramp taper. The magnitude of the stress increase on the underlying subgrade therefore varies both with the height of the embankment but also with the width of the widening.

The magnitude of the settlement also varies with the thickness and compressibility of the clay deposit. As noted previously, there are four discrete sections of embankment underlain by compressible unweathered silty clay deposits, as follows:

- Stations 23+825 (west limit) to 23+900 – Boreholes 05-216 and 05-215;
- Stations 23+950 to 24+100 – Boreholes 05-213, 05-212, and 05-211;
- Stations 24+250 (at the east bridge abutment) to 24+300 – Borehole 05-201; and,
- Stations 24+575 to 24+700 (east limit) – Boreholes 05-208, 05-209, and 05-217.

Along the remaining sections of the alignment, where the unweathered silty clay is absent, the subgrade soils are still expected to compress under the weight of the embankment widening, however the coefficient of consolidation of the weathered silty clay crust, typically being a fissured soil and being stressed within its re-compression limits, is relatively high. Therefore the subgrade settlements resulting from compression of the weathered silty clay crust would be expected to occur quite rapidly, likely entirely during embankment construction, such that the post-construction settlements of the embankment surface would not be expected to noticeably exceed the compression of the embankment fill itself.

The critical location in terms of the embankment settlement is considered to be the edge of the top of the new embankment (i.e., essentially the edge of the new ramp lane), since it is only the settlements of the top of the embankment (and not the side slopes) that is a concern with regards to roadway performance. This is also the location with the greatest stress increase on the underlying subgrade. The existing effective stress profile within the silty clay beneath the future embankment edge, and the resulting stress increase from the widening, were calculated using a closed form solution based on elastic stress distribution theory for a 2 dimensional embankment loading (Das 1990).

As described previously in Section 5.2.2 of this report, the calculated effective stress level in the unweathered silty clay deposit generally exceeds the preconsolidation pressure of this deposit indicated by the laboratory oedometer consolidation testing; the laboratory oedometer consolidation tests were all carried out on samples taken from boreholes put down outside of the existing embankment footprint. The silty clay beneath the embankment slopes has been locally pre-loaded above its original preconsolidation pressure. Therefore, for the purposes of the settlement analyses for the edge of the new embankment, the preconsolidation pressure of the deposit was taken as the existing effective stress level (i.e., the clay is normally consolidated). Therefore all of the calculated settlements occur within the 'virgin' compression range, with no contribution from re-compression of the deposit. As described in Section 4.2.4 of this report, the coefficient of consolidation of the deposit is significantly lower at stress levels exceeding the preconsolidation pressure and therefore the calculated settlements will therefore take longer to occur and should largely be manifested after construction of the embankment.

The calculated settlements resulting from primary consolidation of the deposit are as follows:

Station	Borehole Number	Overall Embankment Height (m)	Thickness of Compressible Unweathered Silty Clay (m)	Slope of Existing Embankment	Width of Widening (m)	Proposed Additional Fill Thickness Beneath Future Edge of Pavement	Calculated Post-Construction Settlement (mm)
23+825	05-216	4.7	2.4	3.0 H:1V	10	2.8	175
23+875	05-215	5.1	1.5	2.7 H:1V	8	2.3	100
23+925	05-214	5.9	0	2.5 H:1V	5	1.9	-
23+975	05-213	6.6	0.6	2.4 H:1V	4.5	1.5	-
24+025	05-212	7.4	1.1	2.2 H:1V	4.5	1.6	100
24+075	05-211	8.1	0.8	2.1 H:1V	4	1.6	50
24+150	05-210	8.9	0	2.0 H:1V	3.7	1.6	-
24+275	05-201	9.6	1.7	2.0 H:1V	3.7	1.6	50
24+300	05-202	9.6	0	2.0 H:1V	3.3	1.3	-

Station	Borehole Number	Overall Embankment Height (m)	Thickness of Compressible Unweathered Silty Clay (m)	Slope of Existing Embankment	Width of Widening (m)	Proposed Additional Fill Thickness Beneath Future Edge of Pavement	Calculated Post-Construction Settlement (mm)
24+350	05-203	8.6	0	2.1 H:1V	3.7	1.6	-
24+400	05-204	7.5	0	2.2 H:1V	3.8	1.6	-
24+450	05-205	6.4	0	2.1 H:1V	3.7	1.5	-
24+500	05-206	6.1	0	2.4 H:1V	4.0	1.5	-
24+550	05-207	5.7	0	2.3 H:1V	1.0	0.6	-
24+600	05-208	5.4	2.0	2.5 H:1V	1.0	0.8	25
24+650	05-209	4.9	2.3	2.9 H:1V	< 1.0	0.4	25
24+700	05-217	3.8	5.0	3.7 H:1V	1.0	0.5	50

It should be noted that, due to the limited thickness of the clay deposit along much of the embankment, the time required for these settlements to occur is fairly limited. Figure 12 summarizes the calculated settlements for time periods of 3 months, 6 months, 9 months, and 12 months following construction. Figure 13 also summarizes the calculated settlement versus time for stations 23+825, 24+025, 24+280, 24+650, and 24+700. In making the assessment shown on Figures 12 and 13, it has been assumed that the rate of construction of the widening will be relatively rapid and therefore all of the primary consolidation settlements will occur after construction; this is a somewhat conservative assumption. The estimates provided on Figures 12 and 13 are also based on a coefficient of consolidation of 10^{-3} cm^2/s. That value is near the upper range of the measured values, for stress levels in excess of the preconsolidation pressure, but is considered to be a realistic assessment of the bulk behavior of this soil.

The results shown on Figures 12 and 13 indicate that, along most of the embankment, the settlements should largely be complete by about 3 months. The exceptions are the extreme east and west ends of the embankment, where the clays are thicker.

It should be noted that the settlements indicated on Figures 12 and 13 account for post-construction primary consolidation settlements of the subgrade only. Settlements of the embankment fill itself would be in addition to those values and, as discussed above, for sections of the embankment underlain by only the weathered portions if the clay deposit and for which the subgrade settlements should occur failure rapidly, the *post-construction* subgrade settlements should, from a practical perspective, be effectively nil.

In the longer term, these settlements would increase due to secondary compression (creep) of the deposit. It is expected that over a period of 10 years following construction (the likely approximate time until the next repaving) secondary compression could increase these settlements by about 25 percent.

It should also be noted that the settlement estimates given above correspond to the edge of the embankment widening, where the settlement will be greatest. The load from embankment widening will also cause some settlement of the edge of the existing lanes. However those settlements are estimated at typically less than about one quarter of the settlement at the edge of the widening and therefore, for most locations, no more than about 10 mm.

It is considered that the post-construction settlement estimates provided above are somewhat excessive in the area of Stations 23+825 to 23+875 and 24+025, where the settlements will approach or exceed 100 mm. Distortion of the roadway in those areas could be excessive and impact on the serviceability and/or safety of the roadway. The embankment settlements calculated at Station 24+280 (behind the east bridge abutment), though relatively limited in magnitude at about 50 mm, may also be problematic considering that the pile supported bridge structure itself will be un-settling.

If the settlement estimates given above can not be tolerated, then consideration could be given to the following seven options, which are also summarized on Table 2, along the advantages, disadvantage, relative costs, and risks/consequences of each:

Option 1 – Allow Embankments to Settle.

The embankments could be allowed to settle, with the expectation that it would be necessary to mill and re-pave the new lanes is the near future (say, 1 year after construction) to return the lane to an acceptable profile and cross fall. It should be noted however that, for the period prior to re-paving, the settlements could have a detrimental impact on the serviceability and safety of the roadway.

Option 2 – Pre-Loading.

The new ramp could be pre-loaded and allowed to settle prior to paving. Based on the analysis results indicated on Figures 12 and 13, a pre-load time of only 6 months should be sufficient to allow almost all of the primary consolidation settlements to occur; even 3 months time should be sufficient for most of the embankment, with only the far west limit experiencing further significant settlement, though still only about 30 mm in magnitude. However these times are estimates only, and the actual pre-load time would need to be confirmed by monitoring of the settlements.

Depending upon the overall construction schedule and the planned time between construction start-up and opening of the ramp, these anticipated pre-load times may or may not impact on the schedule for ramp paving.

A variation on this option (Option 2b in Table 2) would be to construct the ramp and place the base course asphalt only, allow the ramp to settle, put the ramp in-service, and only place the surface course after sufficient time has elapsed for the settlements to occur. This option is essentially the same as Option 1, except that the surface course is not placed until the settlements have occurred, therefore saving the cost for milling to remove the surface course.

If either of these options is selected, an instrumentation monitoring program, with plans, details, and specifications will need to be developed.

Option 3 – Pre-load with Surcharge

A minor surcharge could potentially be considered to accelerate the settlements and reduce the pre-load time. However it is not expected that the surcharge could be very large, since the width of the surcharge would be limited by the existing lane (i.e., the surcharge would have to slope up from the edge of the existing lane). Also, the surcharge will interfere with roadway drainage. In addition, larger surcharges would require widening of the embankment which would be expensive and, at the higher embankment areas, there is locally insufficient right-of-way within which to widen. It is also noted that the expected pre-load times are already rather short and there may therefore be limited benefit in also surcharging, considering the complications involved. One exception to this assessment is in the area between Stations 23+825 and about 23+925 where the embankment is relatively short (5 to 6 m), the widening relatively more (up to 10 m), and therefore it may be more practical to place a surcharge. The clay is also thicker such that the settlements will likely be slower to occur in this area (up to 6 months, versus the 3 months expected elsewhere along the alignment) and therefore the surcharge may have particular benefit.

For that area, a 1.5 m surcharge left in-place for 3 months should be sufficient to obtain all of the expected primary consolidation settlements. The results of stability analyses indicate that a 2H:1V embankment slope inclination should have an acceptable factor of safety for the temporary higher embankment slopes. However the impacts of this surcharge on roadway drainage would need to be addressed.

It should however be noted that, as with Option 2, the actual duration of the pre-loading/surcharge can only be determined by monitoring the embankment settlements.

Option 4 – Lightweight Fill

Lightweight fill (such as EPS) could be used for at least a part of the embankment construction and thereby reduce the stress increase on the compressible clay deposit to a level such that the foundation settlements will be within acceptable tolerances. As a preliminary guideline, it is considered that an approximately 2 m thickness of the new embankment fill would need to be replaced with EPS fill to achieve a sufficiently lower stress increase that the post-construction settlements would be in the order of 50 mm. This treatment would only be required within the limits of the embankment with predicted excessive settlements. A more detailed assessment of the limits and thickness of EPS fill can be provided, if this option is selected.

It should also noted that suitable frost tapers would need to be provided at the ends of the EPS fill treatment to avoid differential frost heaving of the overlying pavement surface.

It is considered that, given the short time required for the preceding three options, and the relatively much higher cost associated with EPS will, this option is unlikely to be preferred.

Other light weight fill materials could also be considered, such as blast furnace slags which have a unit weight between about 11.5 and 14 kN/m^3. As a preliminary guideline, if such materials were to be considered, essentially the full extent of embankment widening, within the critical areas, would need to be constructed with the slag.

The preceding four options are all considered to be technically feasible. However the following three additional options have also been considered.

Option 5 – Pre-loading with Wick Drains.

The use of wick-drains in conjunction with pre-loading would decrease the required pre-load time. However it would be difficult to install wick drains into the clay beneath the future lanes, since that clay underlies the existing embankment side slope. The embankment widening would therefore need to be built up to at or just below the finished grade, such that a rather narrow platform would exist upon which the equipment could work to advance with wick drains into the target zone beneath the future lanes. The wick-drain holes might have to be pre-augered to penetrate through the embankment fill. However, considering that the pre-load time is already rather short, and given the challenges and costs involved, this option is considered only marginally feasible.

Option 6 – Excavate and Replace the Silty Clay

The silty clay deposit could be entirely excavated from beneath the embankment widening, down to the surface of the underlying sand/silt and glacial till, and replaced with suitably compacted embankment fill. However, although the silty clay deposit is generally quite thin, the material which will compress and result in settlement of the future lanes is located beneath the current embankment side slope. That slope would therefore need to be shored such that the excavation could access the clay deposit without undermining the roadway. Given the challenges and costs involved, this option is not considered feasible.

Option 7 – Lower the Profile Grade

If the profile grade of the new ramp could be lowered, the stress increase on the clay deposit would be also lowered and the settlements would be reduced. However the profile grade is controlled by the profile of the existing lanes and bridge and by the vertical clearances over the railway. This option is therefore not considered to be feasible.

Option 5, 6, and 7 are therefore considered to not be feasible.

A summary comparison of the advantages, disadvantages, relative costs, and risks associated with the above options, from a geotechnical perspective, is presented in Table 2 following the text of this report. Based on the understanding that the construction schedule for this project would permit the ramps to be constructed up to about one year before the ramp needs to be in-service, it is considered that Option 2a (preloading) is preferred, in that it has probably the lowest cost and potentially little or no impact on the overall construction schedule.

It should be noted that the above options are only applicable to those locations where the predicted embankment settlements are considered to be excessive, however, for some of the options, the mitigation measures would impact on the full alignment. For example, if the embankment subgrade is to be preloaded, even those sections which do not require the pre-loading could not be put in-service until the pre-loading of the critical locations was complete.

5.3 Construction Considerations

5.3.1 Groundwater and Surface Water Control

Based on the water levels measured in the piezometers, and depending upon the time of year during which the construction is carried out, the groundwater level could be at relatively shallow depth below ground surface. However it expected that, during subgrade preparation, surface water inflow and any shallow groundwater inflow can be handled by providing suitable drainage outlets and/or by pumping from properly filtered sumps within the excavation.

5.7.2 Excavations

No significant excavations are expected for the construction of the embankment widening, other than removal of the existing topsoil.

The contractor should be aware that the exposed clayey subgrade is sensitive to disturbance from construction traffic.

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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

II. PENETRATION RESISTANCE

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

Dynamic 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	$C_{u2}S_u$ 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. τ=c'σ' tan □'
2. Shear strength=(Compressive strength)/2

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RECORD OF BOREHOLE No 05-201 — 1 OF 1 — METRIC

PROJECT 04-1120-013-7000

W.P. 258-98-00 — LOCATION N 5019183.46 ; E 417080.81 — ORIGINATED BY P.A.H.

DIST ___ HWY 417 — BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger — COMPILED BY M.I.C.

DATUM Geodetic — DATE February 21, 2005 — CHECKED BY M.I.C.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ (kN/m³)
65.3	Ground Surface				
0.0 / 65.0	TOPSOIL, Dark brown				
64.7 / 0.6	Sandy SILT, Grey brown, Moist				
	Silty CLAY, Very stiff to stiff, Grey brown, Wet	1	SS	3	
		2	SS	2	
63.0 / 2.3	Silty CLAY, Soft to firm, Grey, Wet				
		3	TP	PH	15.7
61.3 / 4.0	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL), Dense, Grey, Wet	4	SS	42	
60.9 / 4.4	End of Borehole, Auger Refusal				

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (65, 64, 63, 62, 61); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (25, 50, 75); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) (GR SA SI CL)

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

MISS_MTO 04-1120-013-7000.GPJ ON_MOT.GDT 5/10/05

RECORD OF BOREHOLE No 05-202

METRIC

PROJECT 04-1120-013-7000
W.P. 258-98-00
LOCATION N 5019182.29 ; E 417101.19
ORIGINATED BY P.A.H.
DIST HWY 417
BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
COMPILED BY M.I.C.
DATUM Geodetic
DATE February 21, 2005
CHECKED BY M.I.C.

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)
65.3	Ground Surface				
0.0	TOPSOIL				
0.2	Dark brown Silty CLAY Very stiff to stiff Grey brown Wet	1	GRAB		
		2	SS	4	w_P ≈ 25, w ≈ 50, w_L ≈ 65
		3	SS	2	
63.4 / 1.9	Sandy SILT Very loose Grey brown Wet				
63.0 / 2.3	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Loose to compact Grey Wet	4	SS	4	
		5	SS	6	w ≈ 10
		6	SS	13	
60.8		7	SS	>100	
4.5	End of Borehole Auger Refusal				

Column headings: SOIL PROFILE; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (65, 64, 63, 62, 61); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa — O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (25, 50, 75); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

Golder Associates

RECORD OF BOREHOLE No 05-203

METRIC

PROJECT 04-1120-013-7000

W.P. 258-98-00

DIST ______ HWY 417

DATUM Geodetic

LOCATION N 5019191.69 ; E 417144.93

BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger

DATE February 22, 2005

ORIGINATED BY P.A.H.

COMPILED BY M.I.C.

CHECKED BY M.I.C.

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
65.6	Ground Surface
0.0	TOPSOIL Dark brown
65.3	Sandy SILT Grey brown Wet
65.0 0.6	Silty CLAY with silty sand seams Very stiff Grey brown Wet
64.5 1.2	Sandy SILT Grey brown Wet
	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Compact Grey brown Wet
61.3 4.3	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Dense to very dense Grey Wet
59.4 6.2	End of Borehole Auger Refusal

SAMPLES

NUMBER	TYPE	"N" VALUES
1	GRAB	
2	SS	4
3	SS	21
4	SS	14
5	SS	14
6	SS	29
7	SS	55
8	SS	48
9	SS	>50

ELEVATION SCALE: 65, 64, 63, 62, 61, 60

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20, 40, 60, 80, 100

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

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

WATER CONTENT (%): 25, 50, 75

UNIT WEIGHT γ kN/m³

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

Note:

W.L. in Screen at 0.26 m depth below ground surface on Apr. 21, 2005

+ 3, × 3: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 05-204

METRIC

PROJECT 04-1120-013-7000
W.P. 258-98-00
LOCATION N 5019201.03 ; E 417189.78
ORIGINATED BY P.A.H.
DIST HWY 417
BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
COMPILED BY M.I.C.
DATUM Geodetic
DATE February 22, 2005
CHECKED BY M.I.C.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	GR	SA	SI	CL
65.4	Ground Surface									
0.0	TOPSOIL									
65.1	Dark brown									
	Sandy SILT Grey Brown Wet	1	GRAB			65				
64.8 / 0.6	Silty CLAY with silty sand seams Very stiff Grey brown Wet	2	SS	4			0	6	(94)	
64.0 / 1.4	Clayey SILT, trace sand and gravel Compact Grey brown Wet	3	SS	12	∇	64	10	5	(84)	
63.6 / 1.8	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Compact Grey Wet	4	SS	22		63				
		5	SS	27		62	24	15	(61)	
61.7 / 3.7	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Compact to very dense Grey Wet	6	SS	>100						
		7	SS	>100		61				
		8	SS	31		60				
		9	SS	73		59				
		10	SS	>100		58				
		11	SS	29						
57.3 / 8.1	End of Borehole									

Note:

Water level in open borehole at 1.5 m depth below ground surface upon completion of drilling.

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

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

MISS_MTO 04-1120-013-7000.GPJ ON_MOT.GDT 5/10/05

RECORD OF BOREHOLE No 05-205

METRIC

PROJECT 04-1120-013-7000
W.P. 258-98-00
LOCATION N 5019216.06 ; E 417238.11
ORIGINATED BY P.A.H.
DIST ___ HWY 417
BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
COMPILED BY M.I.C.
DATUM Geodetic
DATE February 22, 2005
CHECKED BY M.I.C.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
65.5	Ground Surface					
0.0 / 65.1	TOPSOIL Dark brown	1	GRAB			
0.4	Silty CLAY with fine sand layers, trace gravel. Very stiff. Grey brown. Moist	2	SS	6		
64.1 / 1.4	SILT, trace gravel, sand and clay. Compact. Grey brown. Wet	3	SS	13	∇	4 9 (87)
63.5 / 2.0	Silty SAND, trace gravel. Compact. Brown. Wet	4	SS	21		7 55 (38)
62.6 / 2.9	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL). Compact to very dense. Grey. Wet	5	SS	28		
		6	SS	36		
		7	SS	62		
		8	SS	30		
		9	SS	22		
		10	SS	>100		
58.3 / 7.2	End of Borehole					

Elevation scale: 65, 64, 63, 62, 61, 60, 59

Dynamic Cone Penetration Resistance Plot: 20 40 60 80 100
Shear Strength kPa: O UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × REMOULDED — 20 40 60 80 100
Plastic Limit W_P, Natural Moisture Content W, Liquid Limit W_L — Water Content (%): 25 50 75
Unit Weight γ kN/m³

Note:

Water level in open borehole at 1.5 m depth below ground surface upon completion of drilling.

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

RECORD OF BOREHOLE No 05-206

METRIC

PROJECT 04-1120-013-7000			
W.P. 258-98-00	LOCATION N 5019226.89 ; E 417286.63		ORIGINATED BY P.A.H.
DIST ___ HWY 417	BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger		COMPILED BY M.I.C.
DATUM Geodetic	DATE February 23, 2005		CHECKED BY M.I.C.

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	NATURAL MOISTURE CONTENT w (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
65.3	Ground Surface							
0.0	TOPSOIL Dark brown				65			
64.8 0.6	Sandy SILT Grey brown Moist							
	Silty CLAY with silty sand layers Very stiff Grey brown Wet	1	SS	4	64			
		2	SS	2				
63.2 2.1	Silty SAND, some gravel Compact Brown Wet	3	SS	21	63			
62.6 2.7	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Compact to dense Grey Wet	4	SS	19	62			
		5	SS	16	61			
		6	SS	36				
60.2 5.1	End of Borehole Auger Refusal Note: W.L. in Screen at 0.63 m depth below ground surface on Apr. 21, 2005							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

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

RECORD OF BOREHOLE No 05-207

1 OF 1 **METRIC**

PROJECT 04-1120-013-7000
W.P. 258-98-00
LOCATION N 5019238.62 ; E 417335.66
ORIGINATED BY P.A.H.
DIST ____ HWY 417
BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
COMPILED BY M.I.C.
DATUM Geodetic
DATE February 23, 2005
CHECKED BY M.I.C.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)
65.2	Ground Surface						
64.9 / 0.0	TOPSOIL, Dark brown	1	GRAB			65	
0.3	Silty CLAY, Very stiff, Red brown and grey brown, Moist	2	SS	5			~48
63.8 / 1.4	Silty CLAY, Stiff, Grey brown, Wet	3	SS	2		64	~49
62.9 / 2.4	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL), Dense to compact, Grey, Wet	4	SS	42	∇ (approx. el. 63)	63	
		5	SS	54		62	~10
		6	SS	22		61	
60.9 / 4.3	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL), Compact, Grey, Wet	7	SS	24		60	
		8	SS	29			
59.3 / 5.9	End of Borehole						

Note:

Water level in open borehole at 2.3 m depth below ground surface upon completion of drilling.

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

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

RECORD OF BOREHOLE No 05-208

METRIC

PROJECT 04-1120-013-7000			
W.P. 258-98-00	LOCATION N 5019251.26 ; E 417395.22		ORIGINATED BY P.A.H.
DIST	HWY 417	BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger	COMPILED BY M.I.C.
DATUM Geodetic	DATE February 23, 2005		CHECKED BY M.I.C.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES
65.6	Ground Surface			
0.0	Topsoil and silty clay (FILL) Dark brown			
65.1	TOPSOIL Dark brown	1	GRAB	
0.7	Silty CLAY Very stiff Grey brown Wet	2	SS	9
		3	SS	3
63.6 / 2.0	Silty CLAY Firm Grey Wet	4	TP	PH
61.6 / 4.0	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Compact to very dense Grey Wet	5	SS	15
		6	SS	17
		7	SS	74
59.2 / 6.4	End of Borehole			

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; ELEVATION SCALE (65, 64, 63, 62, 61, 60); 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 05-209

METRIC

PROJECT 04-1120-013-7000

W.P. 258-98-00 LOCATION N 5019264.91 ; E 417434.19 ORIGINATED BY P.A.H.

DIST HWY 417 BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger COMPILED BY M.I.C.

DATUM Geodetic DATE February 24, 2005 CHECKED BY M.I.C.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	UNIT WEIGHT γ kN/m³
64.9	Ground Surface				
0.0	TOPSOIL				
64.5 / 0.4	Dark brown	1	GRAB		
	Silty CLAY, Stiff, Red brown and grey brown, Wet	2	SS	2	
63.5 / 1.4	Silty CLAY, Soft to stiff, Grey, Wet	3	TO	PH	16.7
61.2 / 3.7	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL), Dense to compact, Grey, Wet	4	SS	44	
		5	SS	13	
59.9 / 5.0	End of Borehole				

Note:

W.L. in Screen at 0.1 m depth below ground surface on Apr. 21, 2005

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

Elevation scale: 64, 63, 62, 61, 60

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

Golder Associates

RECORD OF BOREHOLE No 05-210

METRIC

PROJECT 04-1120-013-7000

W.P. 258-98-00 LOCATION N 5019159.59 ; E 416927.45 ORIGINATED BY H.E.C.

DIST HWY 417 BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger COMPILED BY M.I.C.

DATUM Geodetic DATE February 18, 2005 CHECKED BY M.I.C.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
65.6	Ground Surface					
0.0	Clayey SILT, some sand Loose Grey brown					
65.0	Sandy SILT Loose Brown Moist	1	SS	7		65
1.1	Silty CLAY Stiff to very stiff Grey brown Moist	2	SS	5	∇	64
63.5 / 2.1	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Loose Grey Moist	3	SS	11		63
	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Compact to very dense Grey Wet	4	SS	13		62
		5	SS	50		
61.0		6	SS	>100		61
4.7	Possible BEDROCK					
60.5 / 5.1	End of Borehole Auger Refusal					

Note:

Water level in open borehole at 2.0 m depth below ground surface upon completion of drilling.

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

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

Golder Associates

RECORD OF BOREHOLE No 05-211

METRIC

PROJECT 04-1120-013-7000
W.P. 258-98-00
DIST ___ HWY 417
DATUM Geodetic

LOCATION N 5019150.83 ; E 416870.26
BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
DATE February 21, 2005

ORIGINATED BY W.C.
COMPILED BY M.I.C.
CHECKED BY M.I.C.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS
65.4	Ground Surface				
0.0	TOPSOIL Dark brown				
65.0 0.4	SILT, trace sand Grey brown Moist				
64.6 0.9	Silty CLAY Very stiff Grey brown Moist	1	SS	5	
		2	SS	4	∇
63.4 2.1	Silty CLAY Firm Grey Moist to wet				
62.5 2.9	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Compact to very dense Grey Wet	3	SS	>100	
		4	SS	36	
		5	SS	>100	
		6	SS	27	
59.4 6.1	End of Borehole Auger Refusal				

Note:

Water level in open borehole at 2.0 m depth below ground surface upon completion of drilling.

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20, 40, 60, 80, 100

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

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

WATER CONTENT (%): 25, 50, 75

UNIT WEIGHT γ kN/m³

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

ELEVATION SCALE: 65, 64, 63, 62, 61, 60

MISS_MTO 04-1120-013-7000.GPJ ON_MOT.GDT 5/10/05

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

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 05-211A

METRIC

PROJECT 04-1120-013-7000

W.P. 258-98-00 LOCATION N ; E ORIGINATED BY W.C.

DIST ___ HWY 417 BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger COMPILED BY M.I.C.

DATUM Geodetic DATE February 21, 2005 CHECKED BY M.I.C.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
65.4	Ground Surface							
0.0	TOPSOIL							
65.0 / 0.4	Dark brown SILT, trace sand Grey brown Moist				65			
64.6 / 0.9	Silty CLAY Stiff Grey brown Moist				64			
63.4	Silty CLAY Firm Grey Moist to wet	1	TP	PH			15.0	
63.0 / 2.4	End of Borehole							

Note:
W.L. in Screen at .95 m depth below ground surface on Apr. 21, 2005

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

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

Golder Associates

RECORD OF BOREHOLE No 05-212

METRIC

PROJECT 04-1120-013-7000
W.P. 258-98-00
LOCATION N 5019143.23 ; E 416821.28
ORIGINATED BY P.A.H.
DIST HWY 417
BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
COMPILED BY M.I.C.
DATUM Geodetic
DATE February 24, 2005
CHECKED BY M.I.C.

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
65.3	Ground Surface									
0.0	TOPSOIL									
65.0 / 0.3	Dark brown	1	GRAB			65				
	Sandy SILT, Grey brown, Moist									
64.5 / 0.8	Silty CLAY with silty sand seams, Very stiff, Grey brown, Moist to wet	2	SS	4		64				
63.9 / 1.4	Silty CLAY, Firm, Grey, Wet	3	SS	2	∇	63				
62.8 / 2.5	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL), Compact, Grey, Wet	4	TP	PH						
		5	SS	27		62				
61.7 / 3.6	End of Borehole, Auger Refusal									

Note:

Water level in open borehole at 2.0 m depth below ground surface upon completion of drilling.

Legend: SHEAR STRENGTH kPa — O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED. PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L.

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

Golder Associates

RECORD OF BOREHOLE No 05-213

METRIC

PROJECT 04-1120-013-7000			
W.P. 258-98-00	LOCATION N 5019139.00 ; E 416772.35	ORIGINATED BY P.A.H.	
DIST	HWY 417	BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger	COMPILED BY M.I.C.
DATUM Geodetic	DATE February 25, 2005	CHECKED BY M.I.C.	

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
65.4	Ground Surface												
65.4 / 0.0	Topsoil with silty sand (FILL) Dark brown					65							
65.1	TOPSOIL Dark brown												
64.5 / 0.9	Silty GRAVEL, some sand, trace clay Brown Moist	1	SS	6				o		43	13	(44)	
	Silty CLAY with silty sand seams Very stiff to stiff Grey brown Wet	2	SS	WH		64	× +						
63.6 / 1.8	Silty CLAY Firm Grey Wet				∇		× +						
63.0		3	TP	PH		63							
62.7 / 2.7	Sandy SILT Loose Grey Wet	4	SS	7		62				0	43	(57)	
61.7 / 3.7	Silty SAND, some clay with cobbles and boulders (TILL) Loose Grey Wet												
	End of Borehole Auger Refusal												

Note:

Water level in open borehole at 2.1 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 05-214

METRIC

PROJECT 04-1120-013-7000
W.P. 258-98-00
DIST ____ HWY 417
DATUM Geodetic

LOCATION N 5019134.00 ; E 416722.71
BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
DATE February 25, 2005

ORIGINATED BY P.A.H.
COMPILED BY M.I.C.
CHECKED BY M.I.C.

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) (25 50 75)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
65.1	Ground Surface										
0.0 / 64.8	TOPSOIL Dark brown						65				
64.5 / 0.6	Sandy SILT and Silty SAND Brown Moist										
	Silty CLAY, with silty sand seams Very stiff Grey brown		1	SS	7				o		
63.7 / 1.4	Moist to wet						64				
	Silty CLAY Stiff Grey brown Wet		2	SS	2				o		
62.8 / 2.3	End of Borehole Auger Refusal						63	× +			

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

MISS_MTO 04-1120-013-7000.GPJ ON_MOT.GDT 5/10/05

Golder Associates

RECORD OF BOREHOLE No 05-215

METRIC

PROJECT 04-1120-013-7000		ORIGINATED BY P.A.H.	
W.P. 258-98-00	LOCATION N 5019130.12 ; E 416673.05	COMPILED BY M.I.C.	
DIST ___ HWY 417	BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger	CHECKED BY M.I.C.	
DATUM Geodetic	DATE February 25, 2005		

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
65.2	Ground Surface					
0.0	TOPSOIL	1	GRAB		65	
0.2	Dark brown Clayey SILT, some sand Brown Wet					
64.4 0.8	Silty CLAY, with sand seams Very stiff Red brown and grey brown Wet	2	SS	7	64	0 14 (86)
63.5 1.7	Silty CLAY Firm Grey Wet	3	SS	2	63	
62.0 3.2	Clayey SILT Loose Grey Wet	4	SS	WH	62	
61.5 3.7	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Dense Grey Wet	5	SS	39	61	
		6	SS	>100		
60.5 4.7	End of Borehole Auger Refusal					

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m³

STRAT PLOT; GROUND WATER CONDITIONS

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

MISS_MTO 04-1120-013-7000.GPJ ON_MOT.GDT 5/10/05

Golder Associates

RECORD OF BOREHOLE No 05-215A

METRIC

PROJECT	04-1120-013-7000		
W.P.	258-98-00	LOCATION	N ; E
ORIGINATED BY	P.A.H.		
DIST		HWY	417
BOREHOLE TYPE	Power Auger 108 mm I.D. Hollow Stem Auger	COMPILED BY	M.I.C.
DATUM	Geodetic	DATE	February 28, 2005
CHECKED BY	M.I.C.		

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE
65.2	Ground Surface				
0.0	TOPSOIL				
0.2	Dark brown				65
	Clayey SILT, some sand Brown Wet				
64.4 0.8	Silty CLAY Very stiff Red brown and grey brown Wet				64
63.5 1.7	Silty CLAY Firm Grey Wet	1	TP	PH	63
62.5 2.7	End of Borehole Note: W.L. in Screen at 0.98 m depth below ground surface on Apr. 21, 2005				

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

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

RECORD OF BOREHOLE No 05-216

METRIC

PROJECT	04-1120-013-7000		
W.P.	258-98-00	LOCATION	N 5019123.67 ; E 416622.45
DIST	HWY 417	BOREHOLE TYPE	Power Auger 108 mm I.D. Hollow Stem Auger
DATUM	Geodetic	DATE	February 28, 2005

ORIGINATED BY P.A.H.
COMPILED BY M.I.C.
CHECKED BY M.I.C.

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

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
65.0	Ground Surface										
0.0	ICE										
0.3	TOPSOIL Dark brown										
	Silty CLAY, with fine sand seams Very stiff to stiff Grey brown to red brown Wet	1	SS	3		64					
		2	SS	3							
63.2 1.8	Silty CLAY Firm to soft Grey Wet					63					
		3	TP	PH			16.2				
		4	SS	WH		62		0	1	(99)	
61.3 3.7	Silty CLAY Stiff Grey Wet					61					
60.8 4.2	End of Borehole Auger Refusal										

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

Golder Associates

RECORD OF BOREHOLE No 05-217

METRIC

PROJECT 04-1120-013-7000
W.P. 258-98-00
DIST ____ HWY 417
DATUM Geodetic

LOCATION N 5019276.92 ; E 417482.21
BOREHOLE TYPE Power Auger 108 mm I.D. Hollow Stem Auger
DATE February 24, 2005

ORIGINATED BY P.A.H.
COMPILED BY M.I.C.
CHECKED BY M.I.C.

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES
64.8	Ground Surface			
64.5 / 0.0	ICE			
	TOPSOIL Dark brown			
0.5	Silty CLAY Stiff Red brown and grey brown Wet	1	SS	3
63.4 / 1.4	Silty CLAY Soft to firm Grey Wet	2	SS	WH
		3	TP	PH
		4	SS	PM
		5	SS	PM
58.4 / 6.4	Sandy SILT Loose Grey Wet	6	SS	6
57.3 / 7.5	Sandy SILT, some gravel, trace clay with cobbles and boulders (TILL) Compact Grey Wet	7	SS	15
56.7 / 8.1	End of Borehole			

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (64, 63, 62, 61, 60, 59, 58, 57); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, X REMOULDED (20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (25, 50, 75); UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

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

TABLE 1
RECORD OF TEST PITS

Test Pit Number	Depth (metres)	Description
TP 05-218	0.00 – 0.20	TOPSOIL
(Elev. 68.81m)	0.20 – 1.00	Grey and grey brown layered fine and medium sand, trace silt (FILL)
	1.00 – 2.00	Red brown to grey brown silty clay (FILL)
	2.00	End of test pit
		Note: Test Pit Dry Upon Completion
TP 05-219	0.00 – 0.20	TOPSOIL
(Elev. 69.4 m)	0.20 – 0.80	Grey brown fine sand, trace to some silt (FILL)
	0.80 – 1.00	Red brown to grey brown silty clay (FILL)
	1.00 – 2.00	Red brown and grey layered fine sand, clayey silt, and silty clay (FILL)
	1.60	End of test pit
		Note: Test Pit Dry Upon Completion
TP 05-220	0.00 – 0.30	TOPSOIL
(Elev. 69.8 m)	0.30 – 2.00	Brown and yellow brown layered fine and medium sand (FILL)
	2.00	End of test pit
		Note: Test Pit Dry Upon Completion
TP 05-221	0.00 – 0.30	TOPSOIL
(Elev. 67.9 m)	0.30 – 0.70	Brown fine sand, trace silt (FILL)
	0.70 – 2.00	Grey and grey brown layered silty sand and fine to medium sand, trace gravel (FILL)
	2.00	End of test pit
		Note: Test Pit Dry Upon Completion

TABLE 2

COMPARISON OF EMBANKMENT ALTERNATIVES
EB CNR OVERPASS EMBANKMENTS

Embankment Option	*Feasibility*	*Advantages*	*Disadvantages*	*Relative Costs*	*Risks/Consequences*
Option 1 Allow embankments to settle	• Feasible, if can accept settlements	• No impact on construction schedule or costs	• Requires post-construction maintenance • Possible safety issue due to settlement	• Relatively low costs, but must consider short term post-construction maintenance costs	• Possible excessive roadway settlement of widened area and edge of adjacent lane.
Option 2a Pre-load	• Feasible	• Minimum post-construction maintenance required, depending on construction schedule	• Delays paving and use of ramp.	• Similar cost as Option 1	• Some uncertainty about schedule, since can not start construction until monitoring indicates sufficient settlement has occurred.
Option 2b Pre-load – delay surface course placement	• Feasible	• No post-construction maintenance required • Does not delay opening of ramp.	• Delays paving of surface course. • Requires possible interim maintenance if settlements present a safety concern.	• Similar cost as Option 1	• Some uncertainty about schedule, since can not place surface course until monitoring indicates sufficient settlement has occurred.
Option 3 Pre-load with surcharge	• Marginally feasible. Can not raise grade much above proposed level without impacting roadway drainage.	• No post-construction maintenance required • Reduces pre-load time	• Delays paving and use of ramp.	• Higher cost than Option 1 since need to also widen embankment. May need flatter slopes and locally acquire property.	• Some uncertainty about schedule, since can not start construction until monitoring indicates sufficient settlement has occurred. • If surcharge, caution required to not de-stabilize embankment.
Option 4 Light weight fill	• Feasible	• No post-construction maintenance. • Minimal impact on schedule • If used near east abutment, would reduce settlement in that area.	• Expensive	• Expensive	• Low risk option, but contractor may successfully propose one of other options as change order

Embankment Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/Consequences
Option 5 Pre-loading with wick-drains	• Not generally feasible since can not install into clay beneath future lanes, since beneath existing slope	• N/A	• N/A	• N/A	• N/A
Option 6 Excavate and replace silty clay	• Not generally feasible since can not excavate clay beneath lane widening without undermining existing roadway	• N/A	• N/A	• N/A	• N/A
Option 7 Lower profile grade	• Not feasible, since grade is fixed by current roadway, bridge, and railway vertical clearances.	• N/A	• N/A	• N/A	• N/A

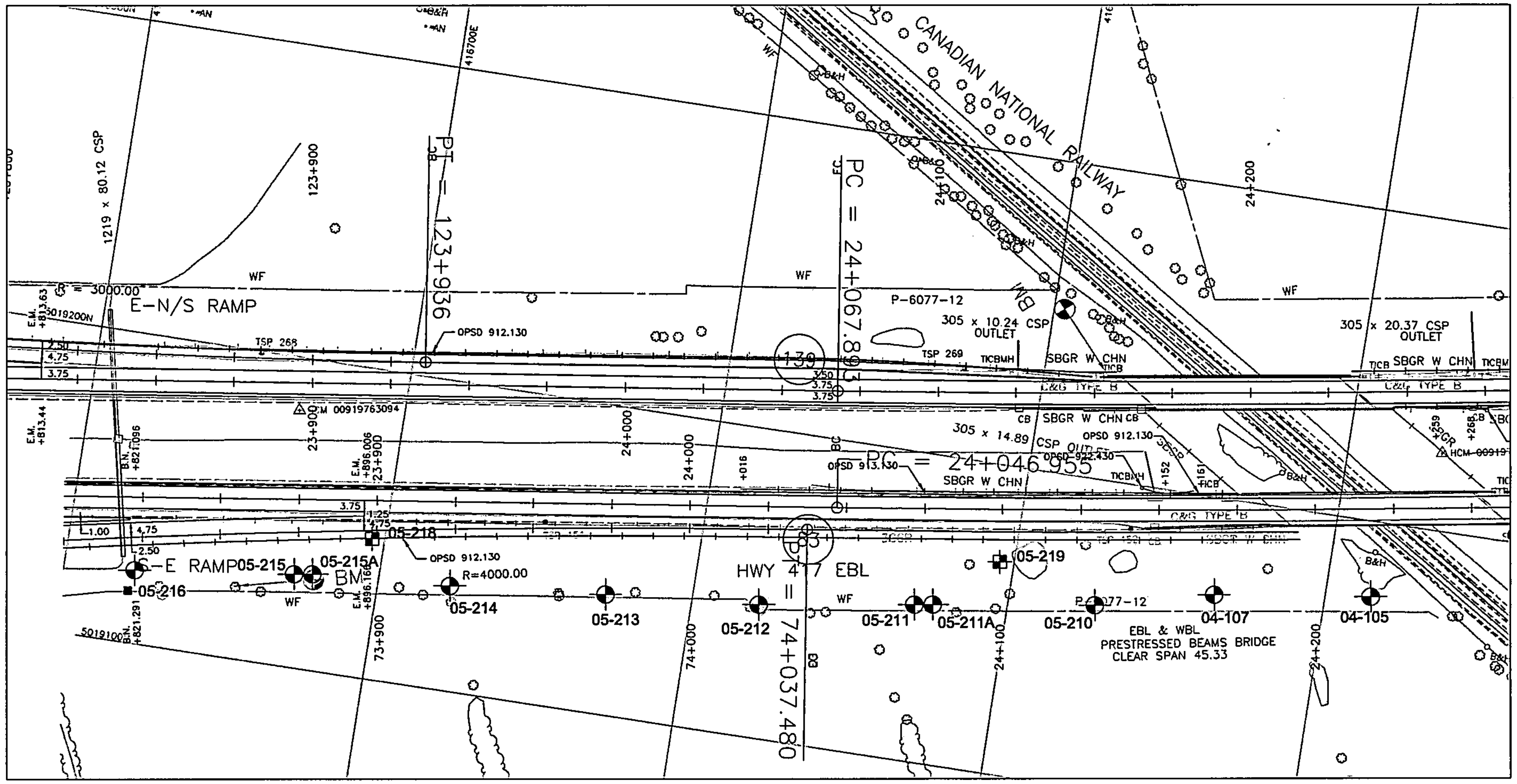
DIST. HWY. 417
CONT No.
WP No. WP 258-98-00

SHEET

BOREHOLE LOCATIONS AND SOIL STRATA

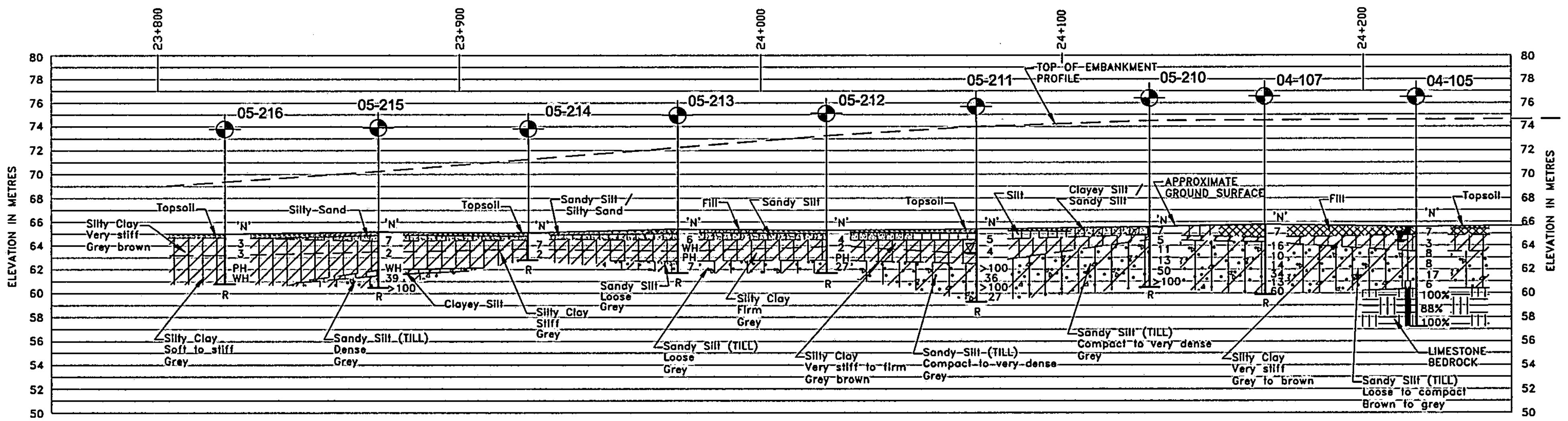
Golder Associates

Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



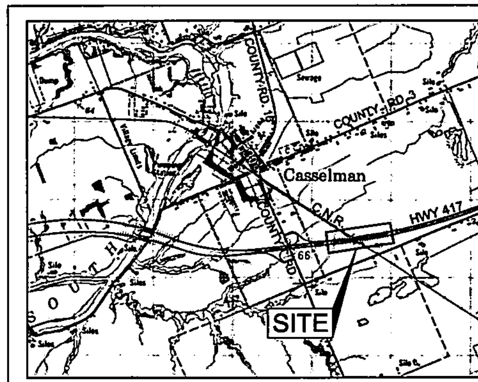
PLAN - WEST SECTION

SCALE 1:1000 METRES



PROFILE ALONG PROPOSED WIDENING OF HIGHWAY 417 EBL

SCALE 1:1000 HORIZONTAL METRES
SCALE 1:250 VERTICAL METRES



KEY PLAN

LEGEND

- Borehole – Current Golder Associates Ltd. Investigation
- Test Pit – Current Golder Associates Ltd. Investigation
- Borehole – Previous MTO Investigation Geocres No. 31GA48
- Seal
- Piezometer
- N Standard Penetration Test value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer
- WL in open borehole

No.	ELEVATION	LOCATION NORTHING	LOCATION EASTING
05-201	65.3	5019183.46	417080.81
05-202	65.3	5019182.29	417101.19
05-203	65.6	5019191.69	417144.93
05-204	65.4	5019201.03	417189.78
05-205	65.5	5019216.06	417238.11
05-206	65.3	5019226.89	417286.63
05-207	65.2	5019238.62	417335.66
05-208	65.6	5019251.26	417395.22
05-209	64.9	5019264.91	417434.19
05-210	65.6	5019159.59	416927.45
05-211	65.4	5019150.83	416870.26
05-211A	65.4	5019150.83	416870.26
05-212	65.3	5019143.23	416821.28
05-213	65.4	5019139.00	416772.35
05-214	65.1	5019134.00	416722.71
05-215	65.2	5019130.12	416673.05
05-215A	65.2	5019130.12	416673.05
05-216	65.0	5019123.67	416622.45
05-217	64.8	5019276.92	417482.21
05-218	68.8	5019145.22	416695.61
05-219	69.4	5019168.88	416895.31
05-220	69.8	5019224.43	417206.47
05-221	67.9	5019269.67	417407.26
04-105	65.6	5019175.50	417014.44
04-107	65.8	5019168.47	416964.75

NOTES

The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

Base plan provided in electronic format by Morrison Hershfield Limited

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

NO.	DATE	BY	REVISION

Geocres No.

HWY. 417	PROJECT NO. 04-1120-013-7000-01		DIST.
SUBM'D.	CHKD. M.I.C.	DATE: MAR. 2005	SITE:
DRAWN: S.L.	CHKD.	APPD.	DWG. 1

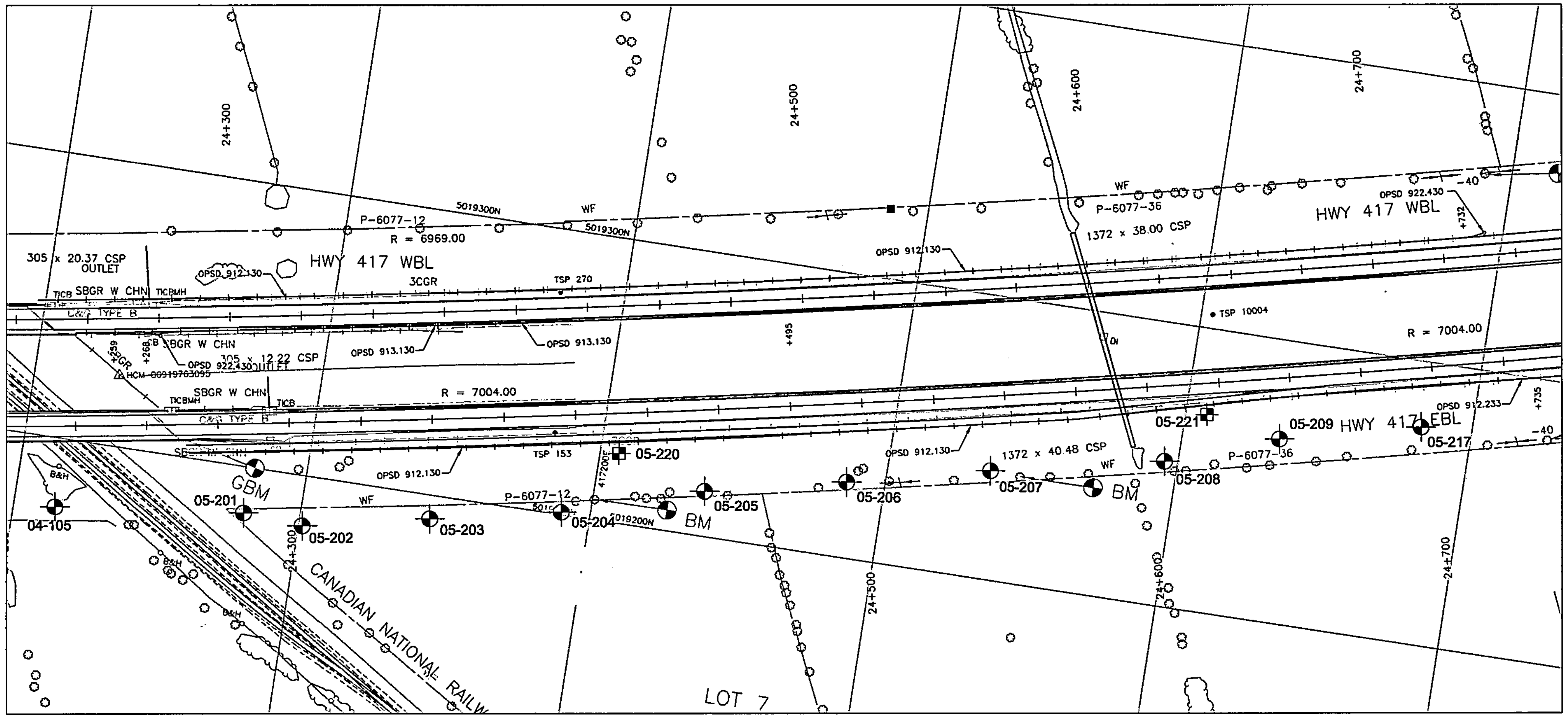
041120013-7000-01.dwg

DIST. HWY. 417
CONT No.
WP No. WP 258-98-00

SHEET

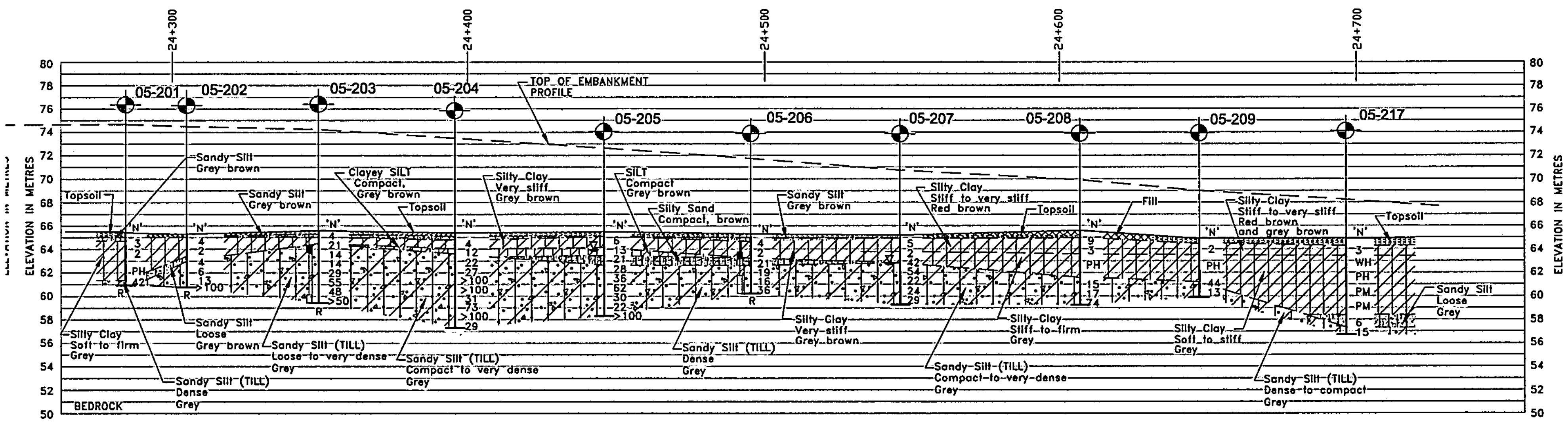
BOREHOLE LOCATIONS AND SOIL STRATA

Golder Associates
Golder Associates Ltd.
MISSISSAUGA, ONTARIO, CANADA



PLAN - EAST SECTION

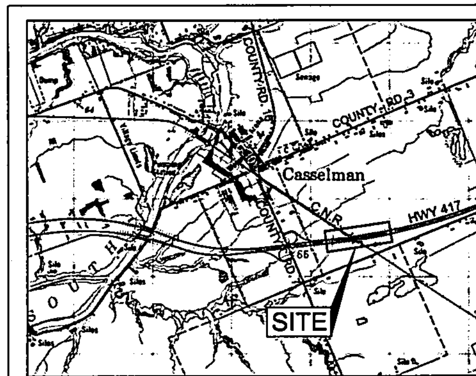
SCALE 1:1000 METRES



PROFILE ALONG PROPOSED WIDENING OF HIGHWAY 417 EBL

SCALE 1:1000 HORIZONTAL METRES

SCALE 1:250 VERTICAL METRES



KEY PLAN

LEGEND

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- Borehole – Previous MTO Investigation Geocres No. 31GA48
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- Piezometer
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- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Rock Quality Designation (RQD)
- WL in piezometer
- WL in open borehole

No.	ELEVATION	LOCATION	
		NORTHING	EASTING
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05-202	65.3	5019182.29	417101.19
05-203	65.6	5019191.69	417144.93
05-204	65.4	5019201.03	417189.78
05-205	65.5	5019216.06	417238.11
05-206	65.3	5019226.89	417286.63
05-207	65.2	5019238.62	417335.66
05-208	65.6	5019251.26	417395.22
05-209	64.9	5019264.91	417434.19
05-210	65.6	5019159.59	416927.45
05-211	65.4	5019150.83	416870.26
05-211A	65.4	5019150.83	416870.26
05-212	65.3	5019143.23	416821.28
05-213	65.4	5019139.00	416772.35
05-214	65.1	5019134.00	416722.71
05-215	65.2	5019130.12	416673.05
05-215A	65.2	5019130.12	416673.05
05-216	65.0	5019123.67	416622.45
05-217	64.8	5019276.92	417482.21
05-218	68.8	5019145.22	416695.61
05-219	69.4	5019168.88	416895.31
05-220	69.8	5019224.43	417206.47
05-221	67.9	5019269.67	417407.26
04-105	65.6	5019175.50	417014.44
04-107	65.8	5019168.47	416964.75

NOTES

The boundaries between soil strata have been established only at borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

Base plan provided in electronic format by Morrison Hershfield Limited

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN

NO.	DATE	BY	REVISION

Geocres No.

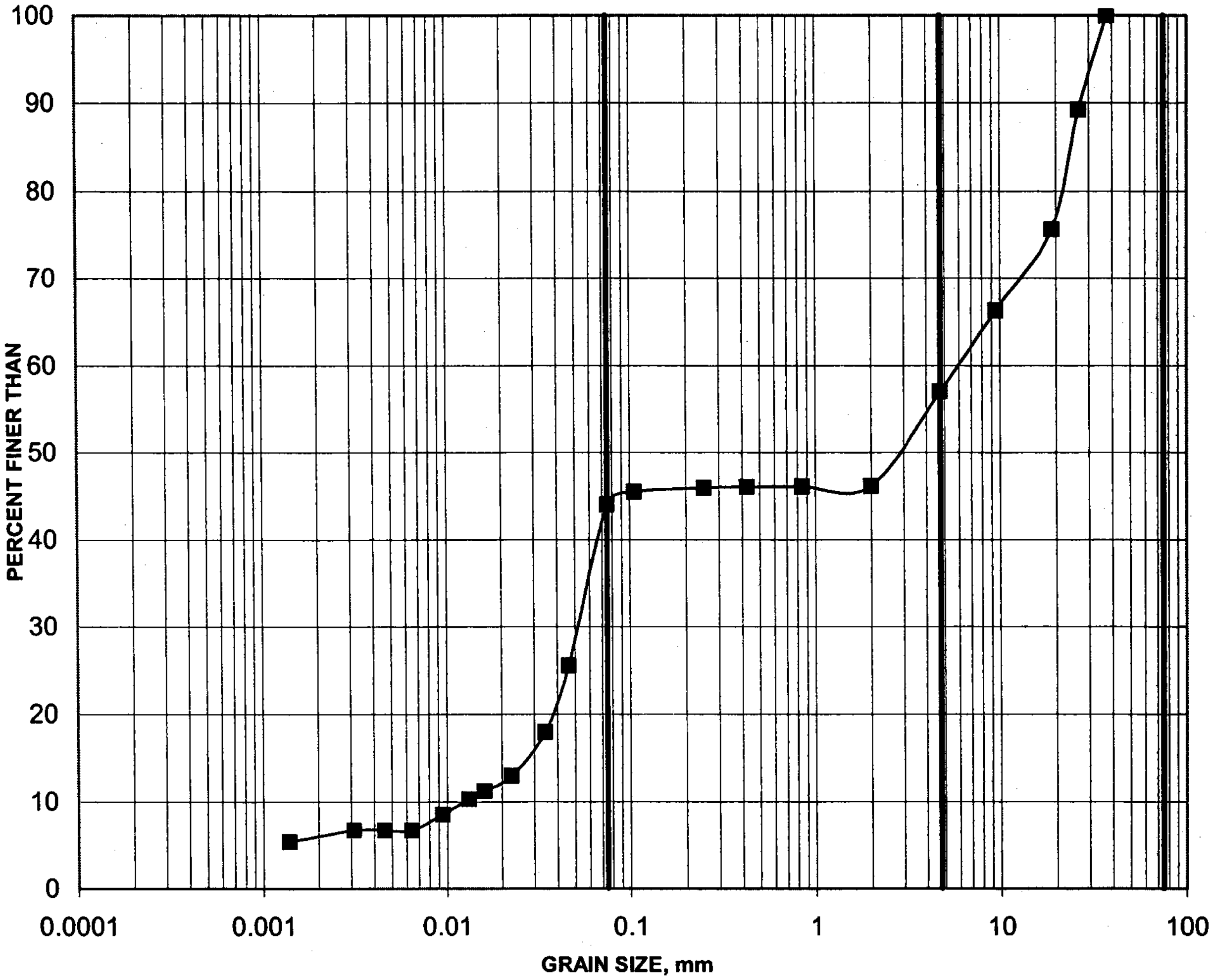
HWY. 417	PROJECT NO. 04-1120-013-7000-02		DIST.
SUBM'D.	CHKD. M.I.C.	DATE: MAR. 2005	SITE:
DRAWN: S.L.	CHKD.	APPD.	DWG. 2

041120013-7000-02.dwg

GRAIN SIZE DISTRIBUTION

FIGURE 1

Shallow Sandy Silt, Silt, and Silty Sand



<table>
<tr><td rowspan="2">SILT AND CLAY</td><td>FINE</td><td>MEDIUM</td><td>COARSE</td><td>FINE</td><td>COARSE</td><td rowspan="2">COBBLE SIZE</td></tr>
<tr><td colspan="3">SAND SIZE</td><td colspan="2">GRAVEL SIZE</td></tr>
</table>

Borehole	Sample	Depth (m)
05-213	1A	0.6-0.9

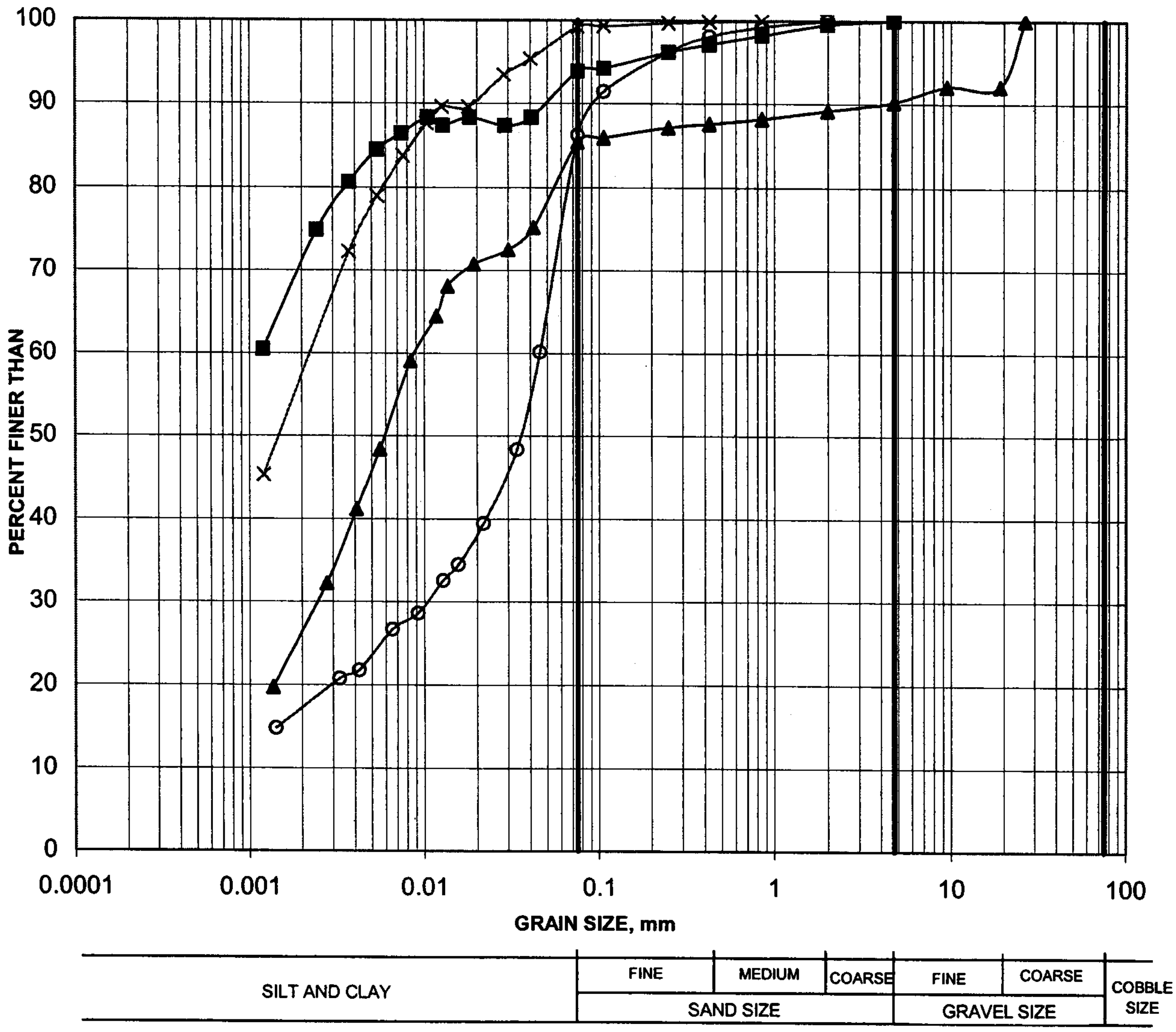
Project: 04-1120-013 **Golder Associates** 3-May-05

Created by: EWK
Checked by: TMS

GRAIN SIZE DISTRIBUTION

FIGURE 2

Silty Clay and Clayey Silt

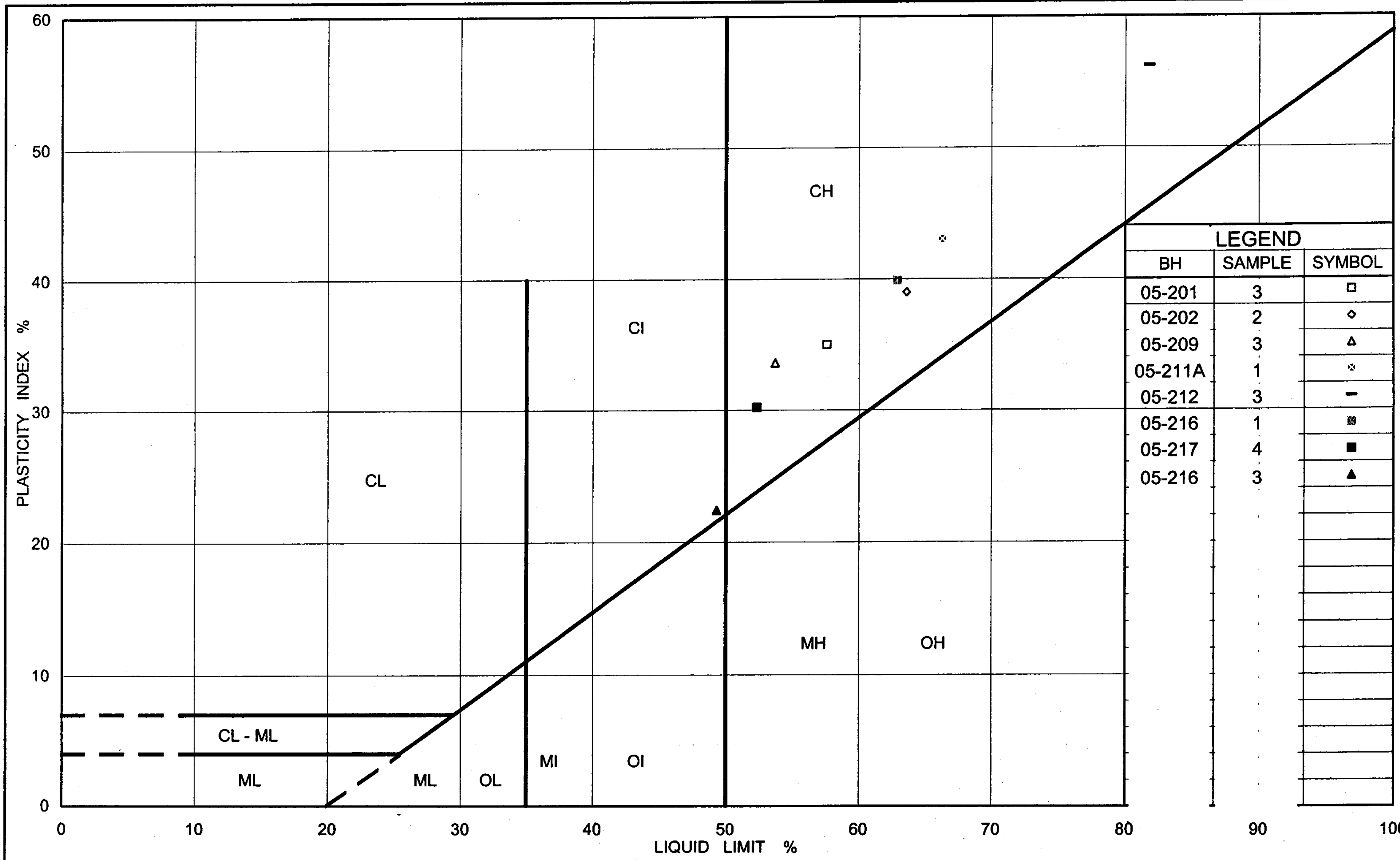


Borehole	Sample	Depth (m)
05-204 (■)	2	0.6-1.2
05-204 (▲)	3	1.4-1.7
05-215 (○)	2A	0.6-0.8
05-216 (×)	4	3.0-3.5

Created by: BAB
Checked by: TMS

Project: 04-1120-013 Golder Associates 3-May-05

Oct 75, FF-S-21



BH	SAMPLE	SYMBOL
05-201	3	□
05-202	2	◇
05-209	3	△
05-211A	1	⊗
05-212	3	━
05-216	1	▩
05-217	4	■
05-216	3	▲

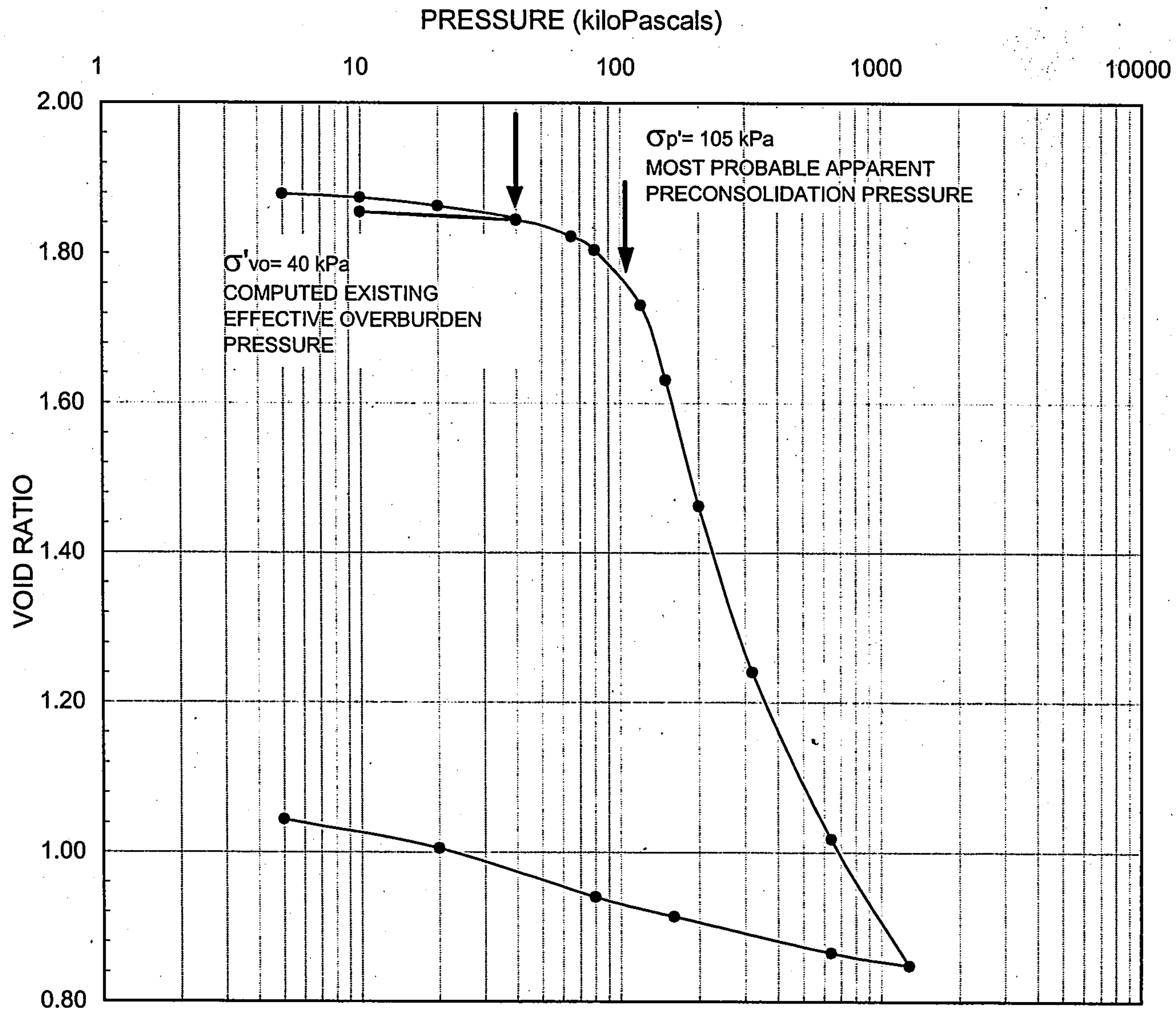
Ministry of Transportation

Ontario

PLASTICITY CHART
Silty Clay to Clay

FIG No. 3

Project No. 04-1120-013-7000

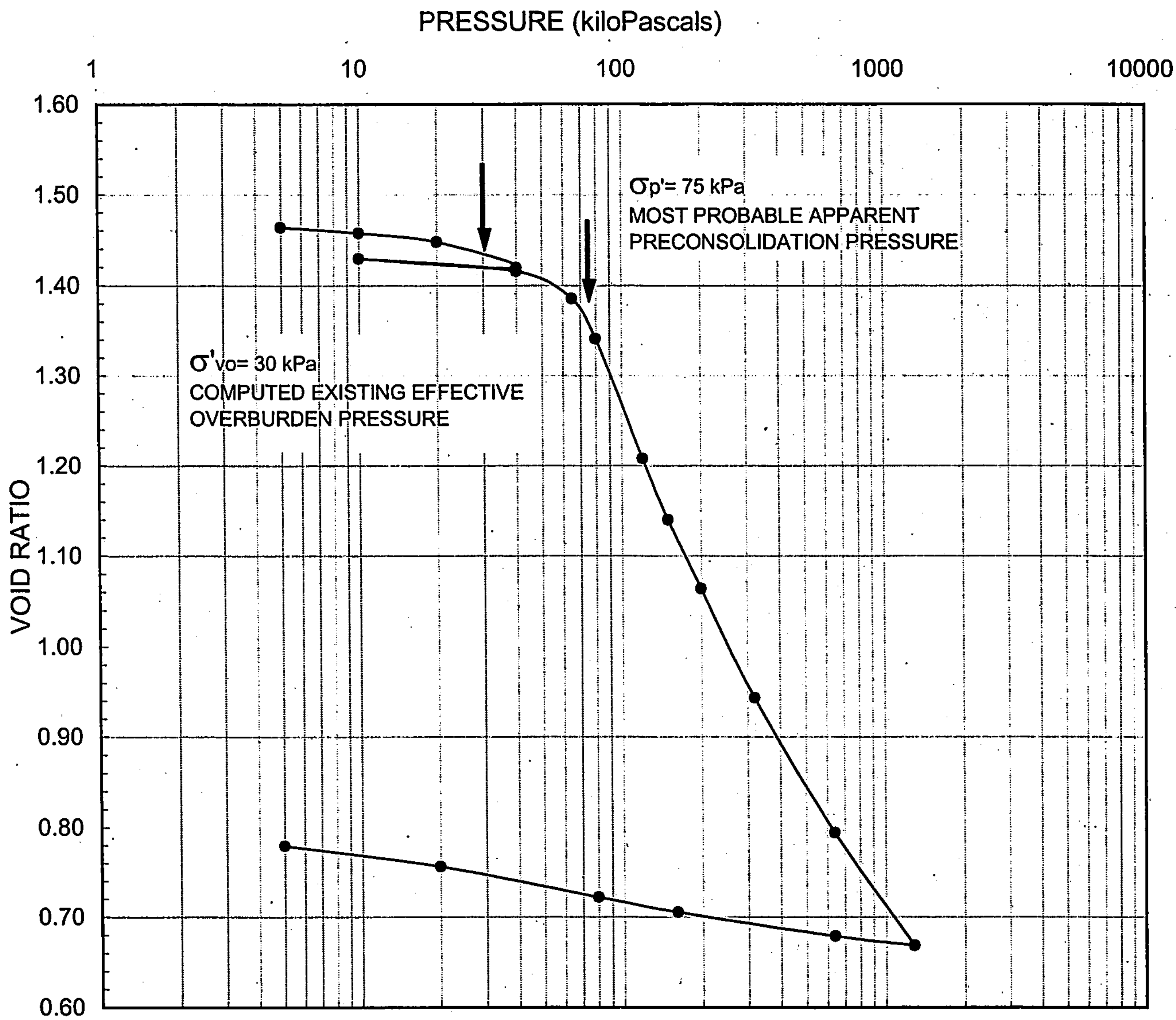


LEGEND

Borehole:	05-201	w_i = 67.5%	S_o = 98%
Sample:	3	w_f = 40.0%	C_c = 1.35
Depth (m):	3.40	w_l = 58	C_r = 0.018
		w_p = 23	

Golder Associates

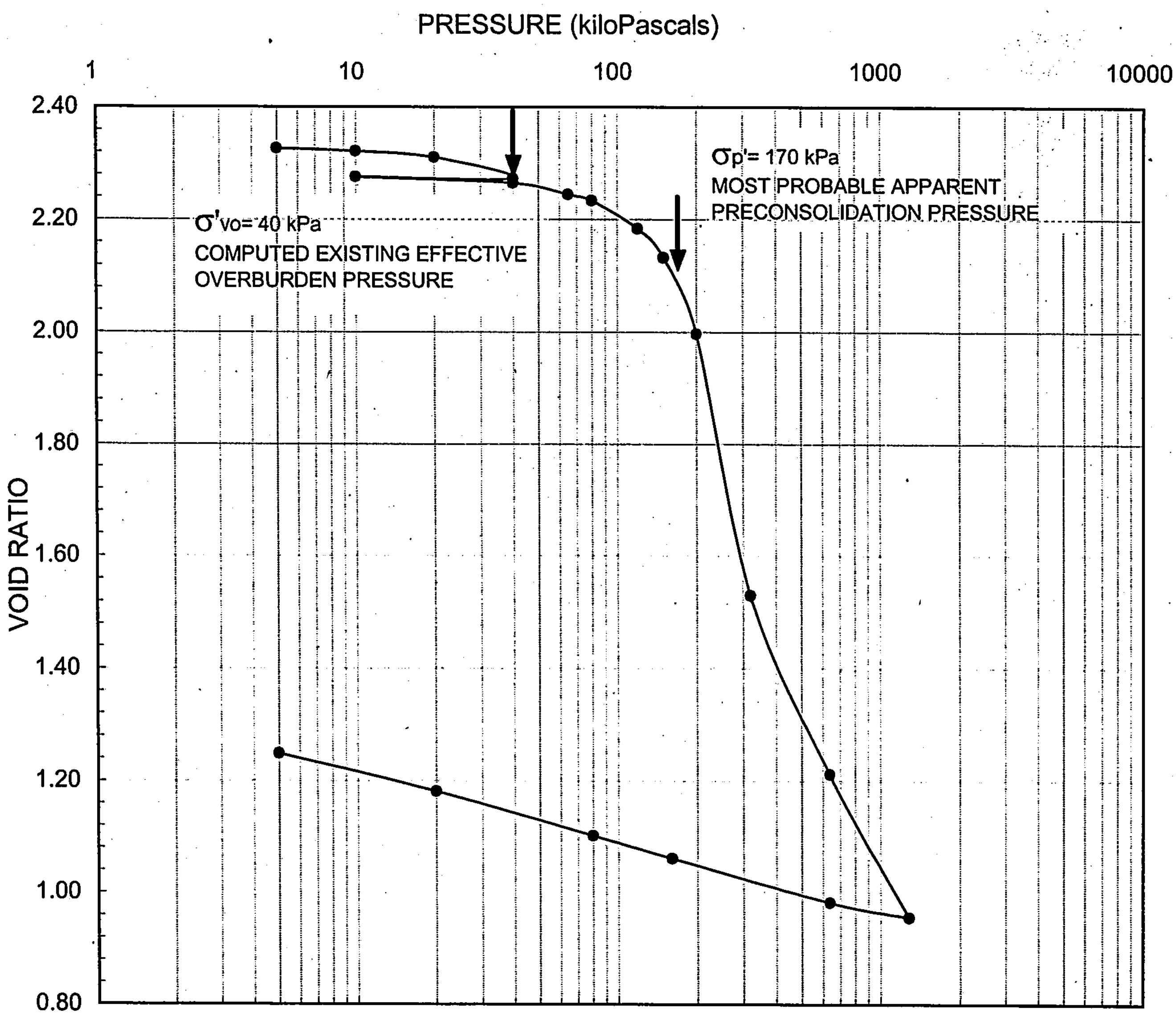
SCALE	AS SHOWN	TITLE	CONSOLIDATION TEST RESULTS
DATE	05/03/05		
DESIGN	NA		
CADD	NA		
FILE No. Consolidation summary	CHECK	EWK	
PROJECT No. 04-1120-013 REV. 0	REVIEW	MIC	FIGURE 4



LEGEND

Borehole: 05-209	w_i = 49.9%	S_o = 95%
Sample: 3	w_f = 26.8%	C_c = 0.70
Depth (m): 2.60	w_l = 54	C_r = 0.023
	w_p = 20	

Golder Associates	SCALE AS SHOWN	TITLE
	DATE 05/03/05	**CONSOLIDATION TEST RESULTS**
	DESIGN NA	
	CADD NA	
FILE No. Consolidation summary	CHECK EWK	
PROJECT No. 04-1120-013 REV. 0	REVIEW MIC	FIGURE 5



LEGEND

Borehole: 05-211A	w_i = 80.0%	S_o = 97%
Sample: 1	w_f = 45.5%	C_c = 2.49
Depth (m): 2.60	w_l = 66	C_r = 0.020
	w_p = 23	

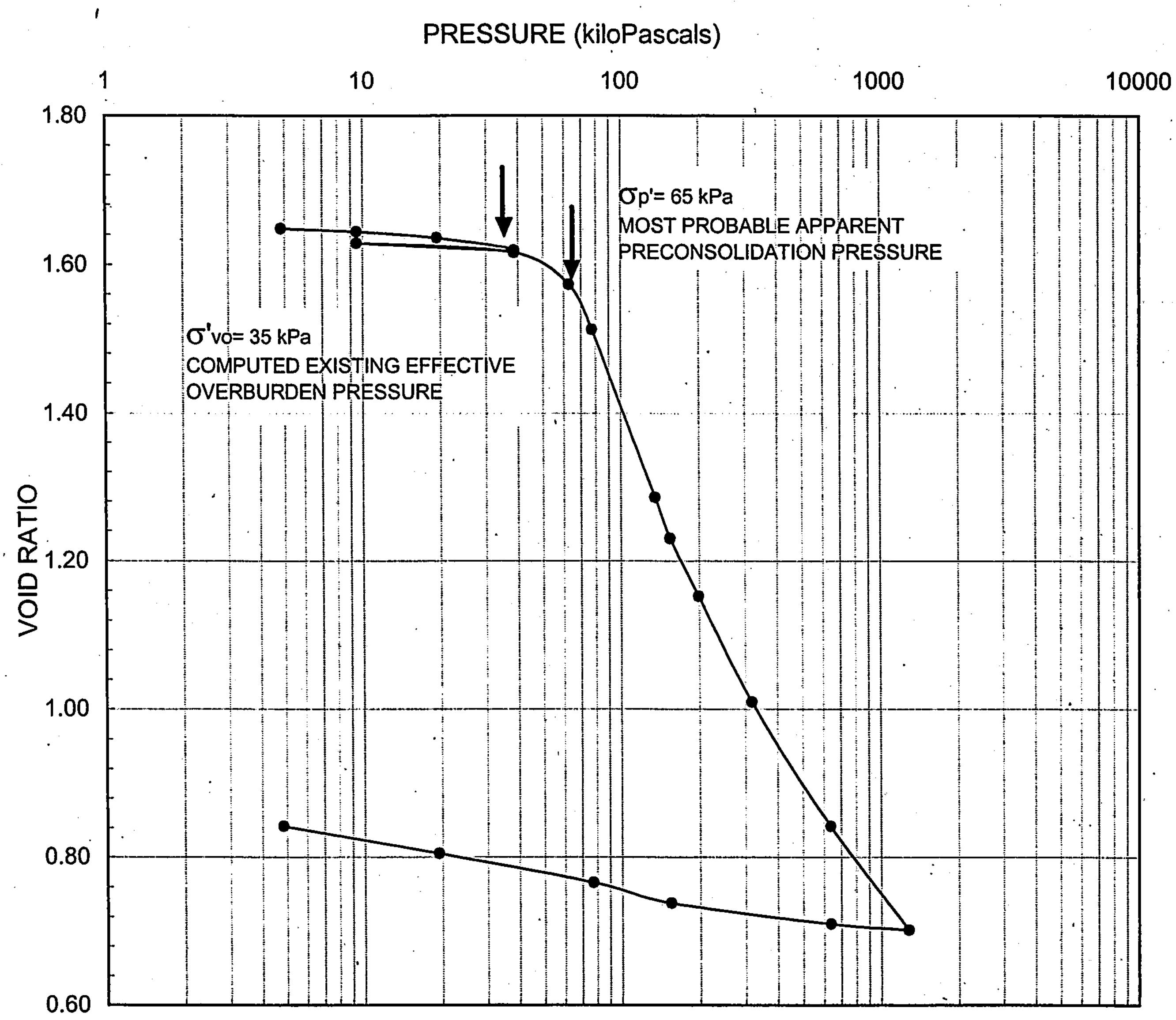
Golder Associates

SCALE	AS SHOWN
DATE	05/03/05
DESIGN	NA
CADD	NA
CHECK	EWK
REVIEW	MIC
FILE No.	Consolidation summary
PROJECT No.	04-1120-013
REV.	0

TITLE

CONSOLIDATION TEST RESULTS

FIGURE 6



LEGEND

Borehole: 05-216	w_i = 60.3%		S_o = 100%
Sample: 3	w_f = 32.9%		C_c = 0.93
Depth (m): 2.70	w_l = 49		C_r = 0.016
	w_p = 27		

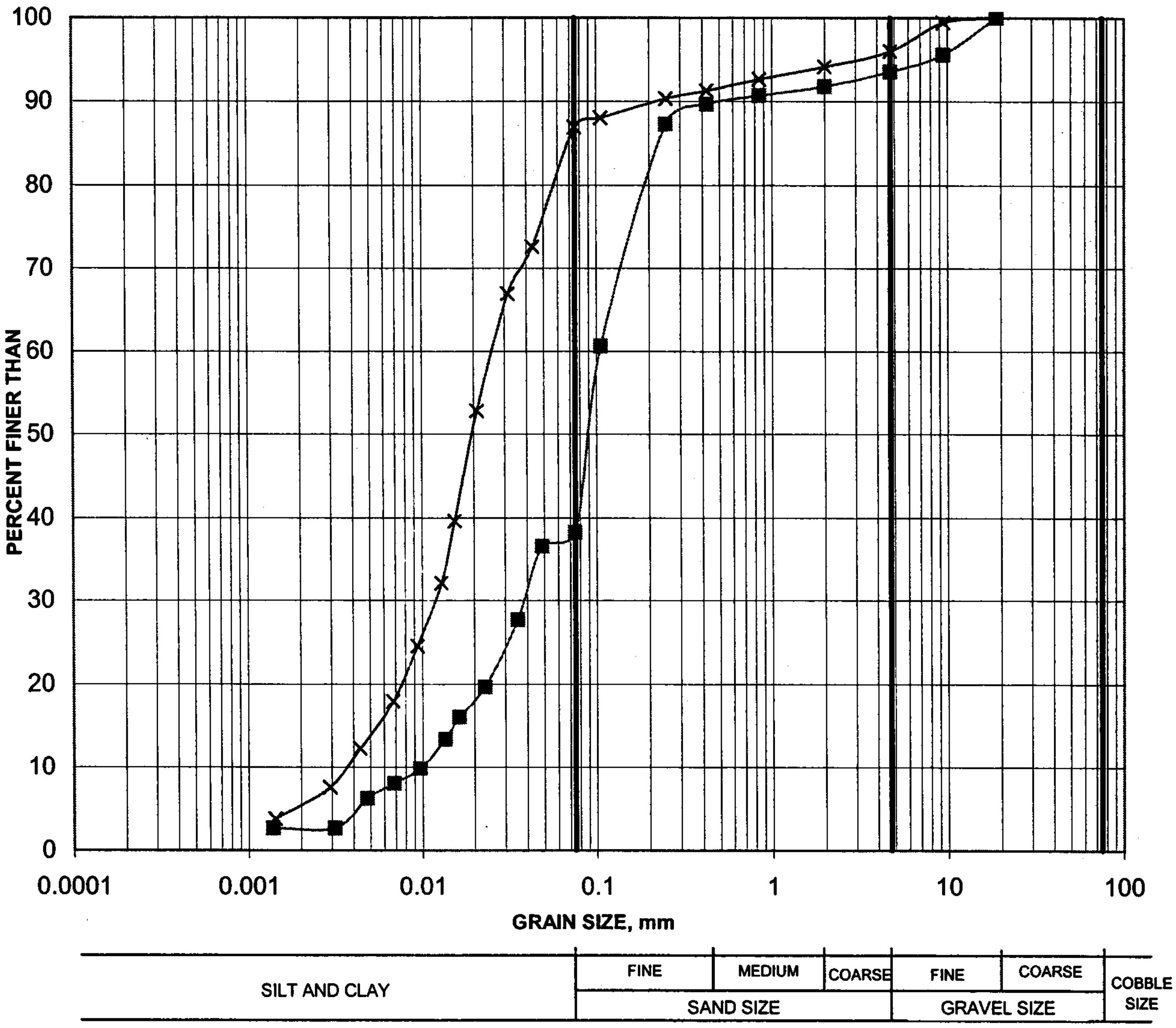
Golder Associates

SCALE: AS SHOWN	TITLE: **CONSOLIDATION TEST RESULTS**	
DATE: 05/03/05		
DESIGN: NA		
CADD: NA		
FILE No.: Consolidation summary	CHECK: EWK	
PROJECT No.: 04-1120-013 REV. 0	REVIEW: MIC	FIGURE 7

GRAIN SIZE DISTRIBUTION

FIGURE 8

Deep Sandy Silt, Silt, and Silty Sand



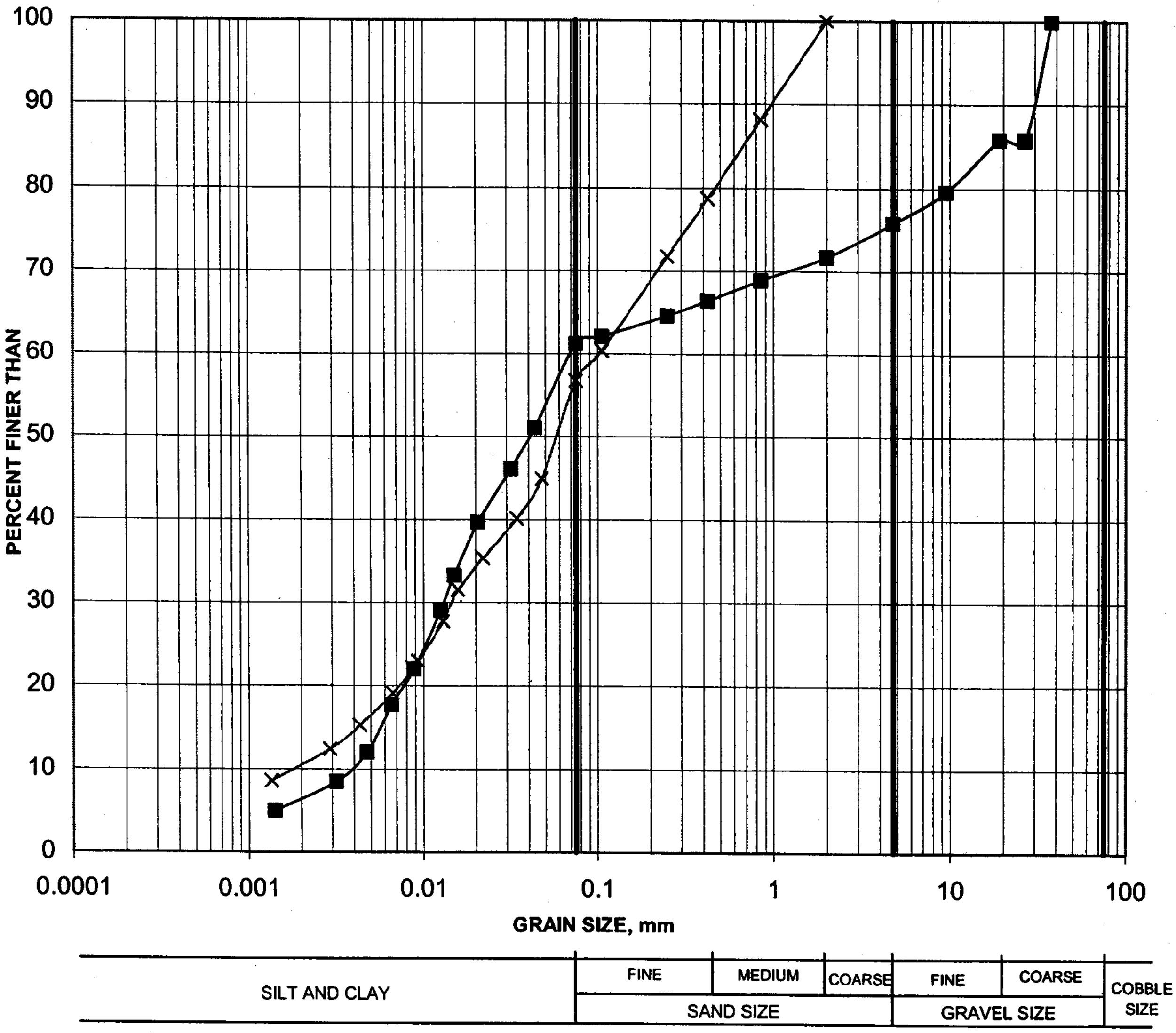
Borehole	Sample	Depth (m)
—×— 05-205	3	1.4-2.0
—■— 05-205	4	2.1-2.7

Project: 04-1120-013 **Golder Associates** 3-May-05

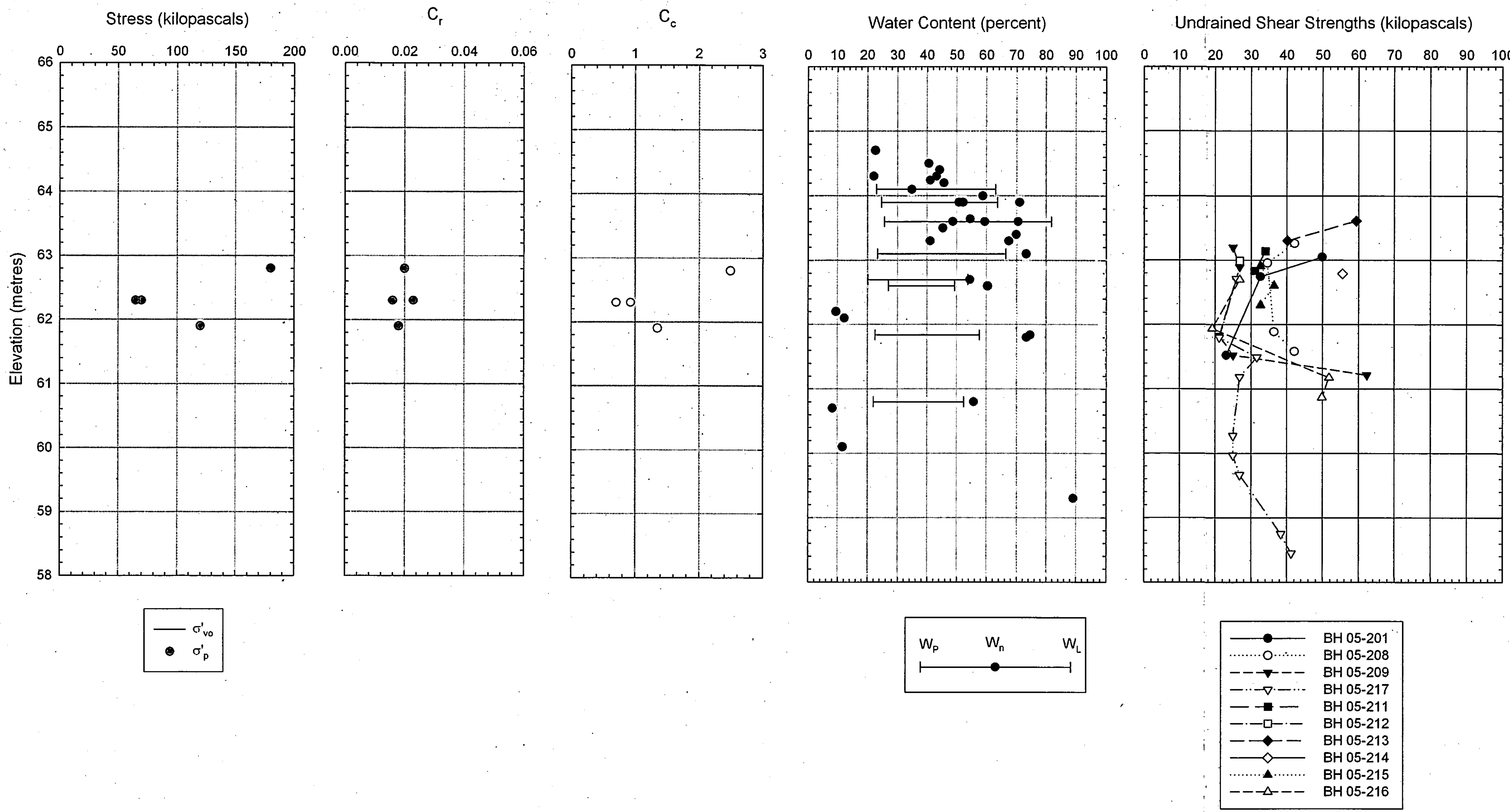
Created by: BAB
Checked by: TMS

GRAIN SIZE DISTRIBUTION	FIGURE 9

Sandy Silt Till



Borehole	Sample	Depth (m)
05-204	5	2.9-3.5
05-213	4	2.9-3.5



Golder Associates
Ottawa, Ontario

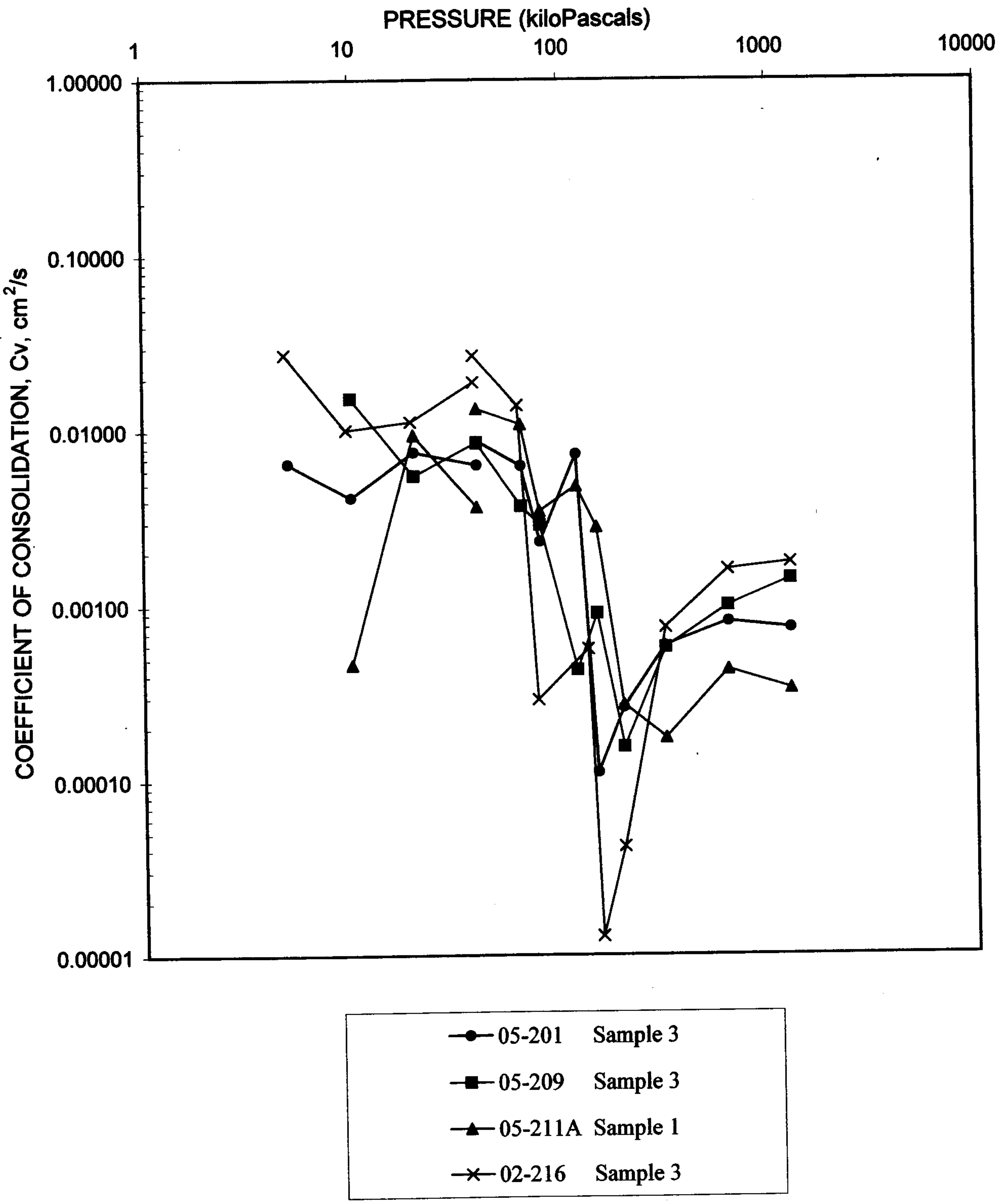
FILE No.	041120013–7000–10.dwg
PROJECT No.	04–1120–013
REV.	
SCALE	N.T.S.
DATE	APR. 2005
DESIGN	
CADD	S.L.
CHECK	
REVIEW	

TITLE

SUMMARY OF ENGINEERING PROPERTIES

FIGURE 10

Drawing file: 041120013–7000–10.dwg Jul 14, 2005 – 12:47pm



Golder Associates	SCALE AS SHOWN	TITLE	
	DATE 05/03/05	**Summary of Coefficient of Consolidation**	
	DESIGN NA		
	CADD NA		
FILE No. Consolidation summary	CHECK EWK		FIGURE
PROJECT No. 04-1120-013 REV. 0	REVIEW MIC		11

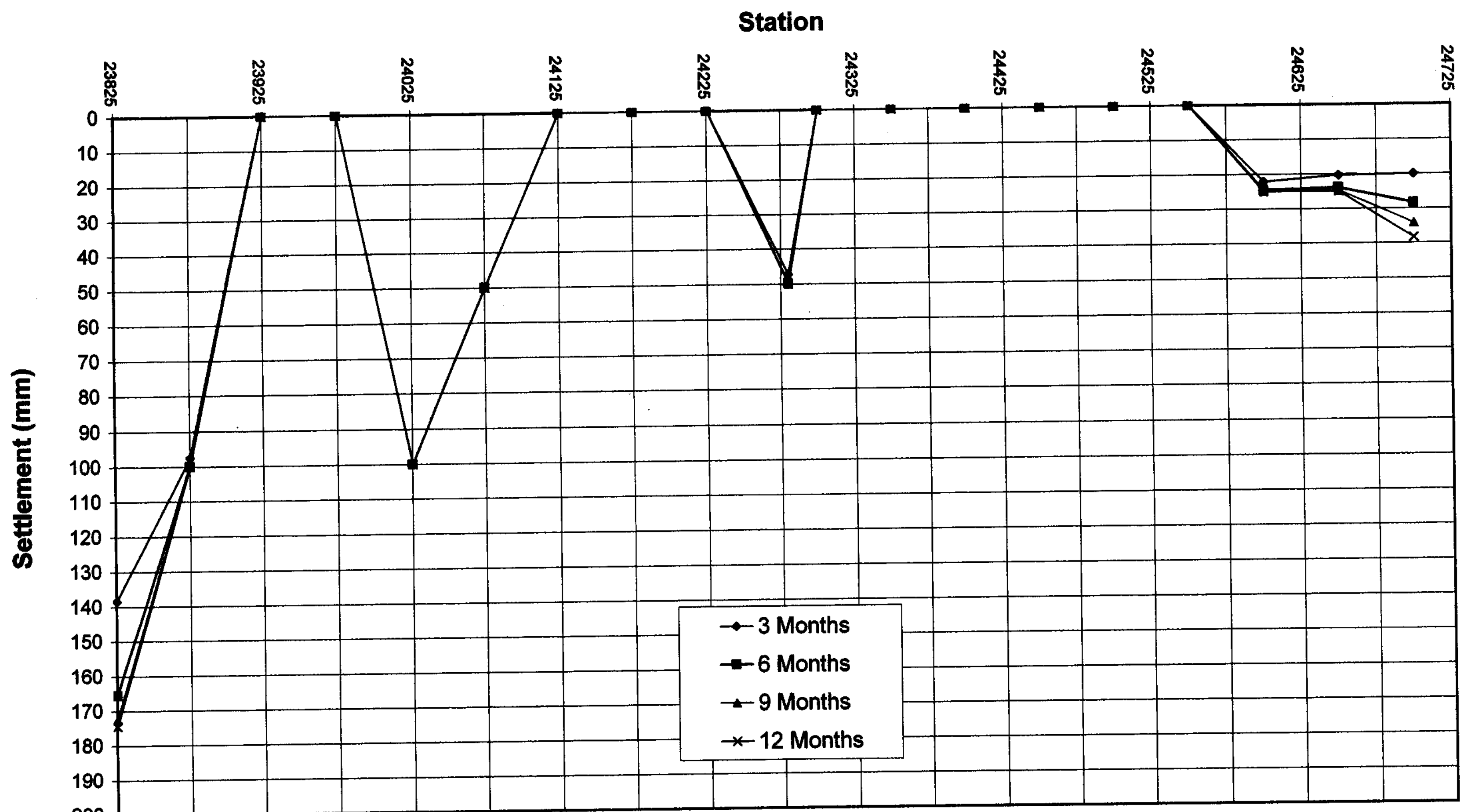


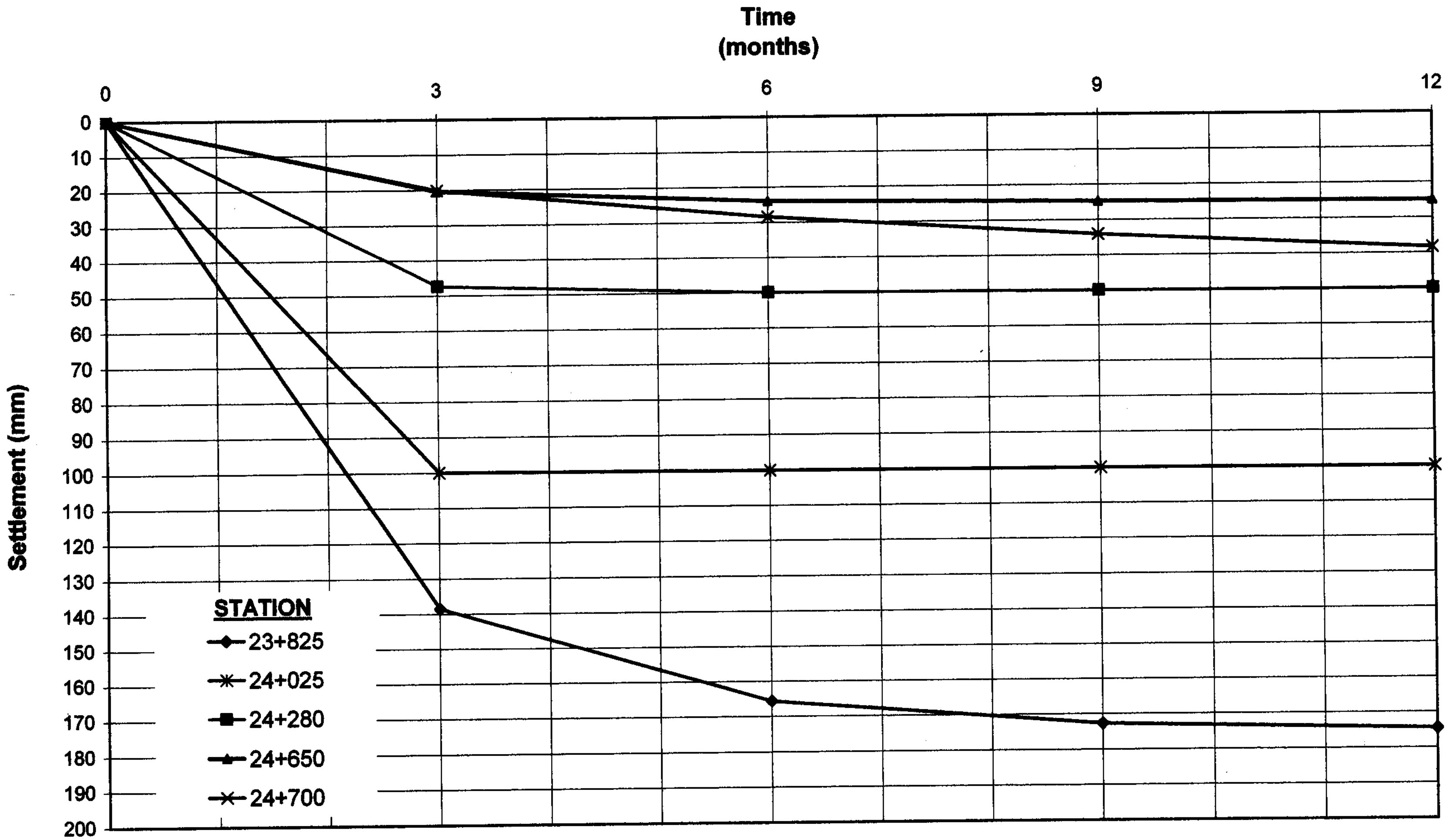
FIGURE 12

PROJECT No.	04-1120-013
FILE No.	04-1120-013
REV. 0	SCALE AS SHOWN
DESIGN	TMS
CADD	
CHECK	TMS
REVIEW	MIC

TITLE

Summary of Estimated Embankment Settlements

Golder Associates



PROJECT No.	04-1120-013
FILE No.	04-1120-013
REV. 0	SCALE AS SHOWN
DESIGN	TMS
CADD	
CHECK	TMS
REVIEW	MJC

TITLE

Summary of Estimated Embankment Settlements Versus Time

Golder Associates

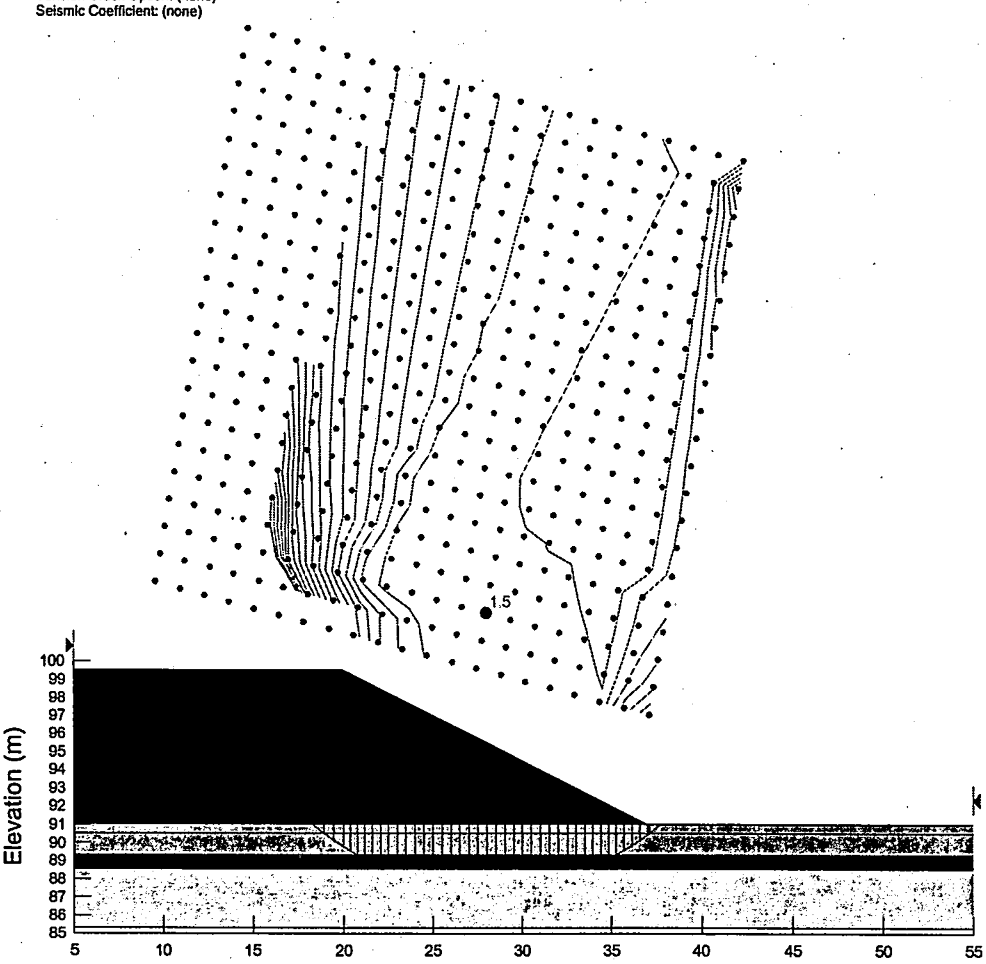
FIGURE 13

APPENDIX A

Figure A1

Station 24+075
Static Analysis

Description: 04-1120-013-7000 MH / HWY 417 / Limoges-Casselman
Comments: Station 24+075 West Approach Embankment
File Name: Borehole 05-211 West End.slz
Last Saved Date: 28/06/2005
Last Saved Time: 12:02:44 PM
Analysis Method: Morgenstern-Price
Direction of Slip Movement: Left to Right
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric lines with Ru
Tension Crack Option: (none)
Seismic Coefficient: (none)



Soil: 1
Description: Embankment - Silty Sand
Soil Model: Mohr-Coulomb
Unit Weight: 20.5
Cohesion: 0
Phi: 32
Piezometric Line #: 1
Ru: 0
Pore-Air Pressure: 0

Soil: 2
Description: Sandy Silt
Soil Model: Mohr-Coulomb
Unit Weight: 19.5
Cohesion: 0
Phi: 28
Piezometric Line #: 1
Ru: 0
Pore-Air Pressure: 0

Soil: 3
Description: Weathered Crust
Soil Model: Undrained (Phi=0)
Unit Weight: 17.5
Cohesion: 75
Piezometric Line #: 1
Ru: 0
Pore-Air Pressure: 0

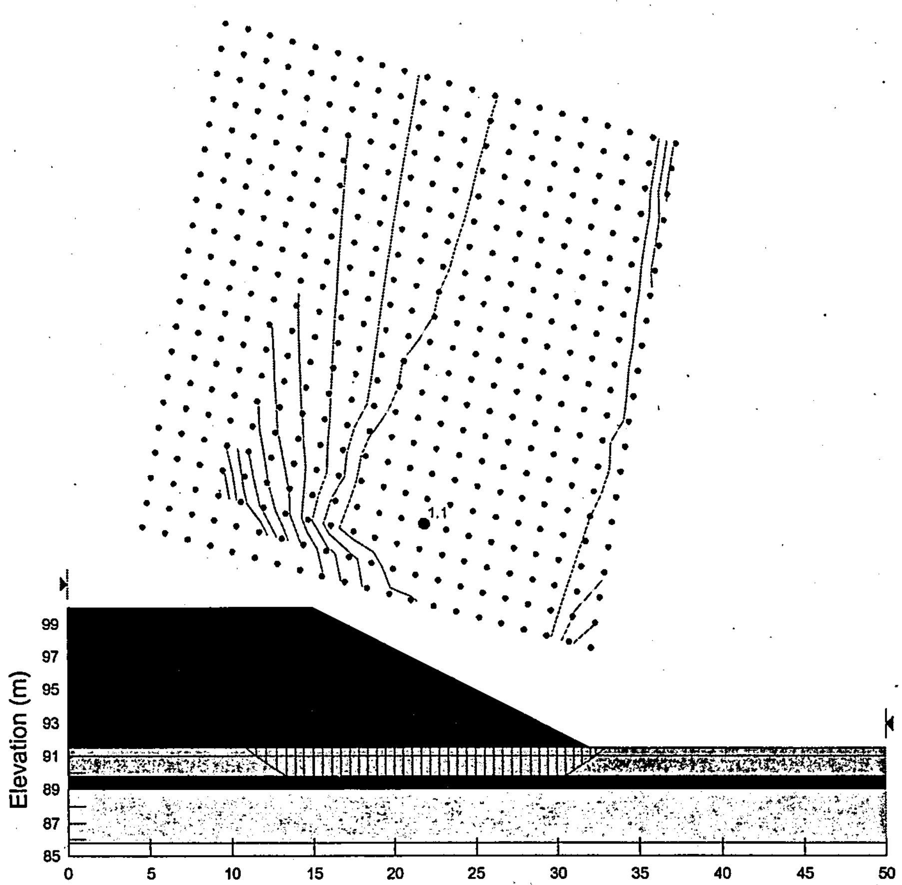
Soil: 4
Description: Grey Clay
Soil Model: Undrained (Phi=0)
Unit Weight: 15.7
Cohesion: 30
Piezometric Line #: 1
Ru: 0
Pore-Air Pressure: 0

Soil: 5
Description: Glacial Till
Soil Model: Bedrock
Piezometric Line #: 1
Ru: 0
Pore-Air Pressure: 0

Figure A2

Station 24+075
Seismic Analysis

Description: 04-1120-013-7000 MH / HWY 417 / Limoges-Casselman
Comments: Station 24+075 West Approach Embankment
File Name: Borehole 05-211 West End Seismic.slz
Last Saved Date: 28/06/2005
Last Saved Time: 12:01:25 PM
Analysis Method: Morgenstern-Price
Direction of Slip Movement: Left to Right
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric lines with Ru
Tension Crack Option: (none)
Seismic Coefficient: Horizontal



Soil: 1
Description: Embankment - Silty Sand
Soil Model: Mohr-Coulomb
Unit Weight: 20.5
Cohesion: 0
Phi: 32
Piezometric Line #: 1
Ru: 0
Pore-Air Pressure: 0

Soil: 2
Description: Sandy Silt
Soil Model: Mohr-Coulomb
Unit Weight: 19.5
Cohesion: 0
Phi: 28
Piezometric Line #: 1
Ru: 0
Pore-Air Pressure: 0

Soil: 3
Description: Weathered Crust
Soil Model: Undrained (Phi=0)
Unit Weight: 17.5
Cohesion: 75
Piezometric Line #: 1
Ru: 0
Pore-Air Pressure: 0

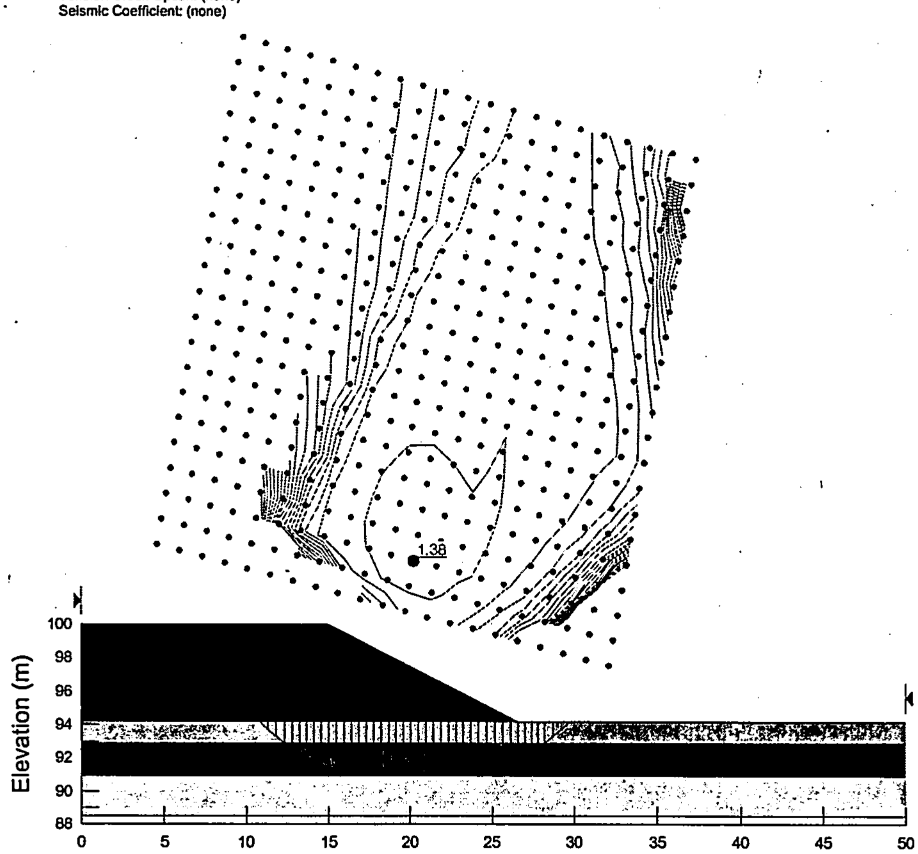
Soil: 4
Description: Grey Clay
Soil Model: Undrained (Phi=0)
Unit Weight: 15.7
Cohesion: 30
Piezometric Line #: 1
Ru: 0
Pore-Air Pressure: 0

Soil: 5
Description: Glacial Till
Soil Model: Bedrock
Piezometric Line #: 1
Ru: 0
Pore-Air Pressure: 0

Figure A3

Station 24+600
Static Analysis

Description: 04-1120-013-7000 MH / HWY 417 / Limoges-Casselman
Comments: Station 24+600 East Approach Embankment
File Name: Borehole 05-208 East End.slz
Last Saved Date: 28/06/2005
Last Saved Time: 12:00:35 PM
Analysis Method: Morgenstern-Price
Direction of Slip Movement: Left to Right
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric lines with Ru
Tension Crack Option: (none)
Seismic Coefficient: (none)



Soil: 1
Description: Embankment - Silty Sand
Soil Model: Mohr-Coulomb
Unit Weight: 20.5
Cohesion: 0
Phi: 32
Piezometric Line #: 1
Ru: 0
Pore-Air Pressure: 0

Soil: 2
Description: Weathered Crust
Soil Model: Undrained (Phi=0)
Unit Weight: 17.5
Cohesion: 75
Piezometric Line #: 1
Ru: 0
Pore-Air Pressure: 0

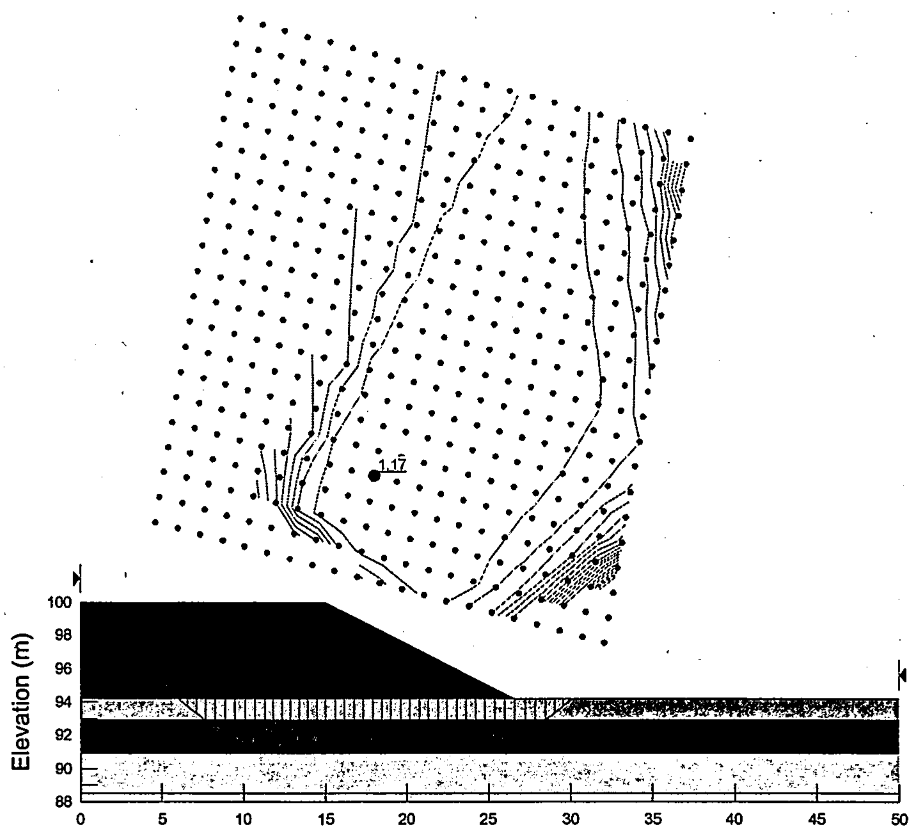
Soil: 3
Description: Grey Clay
Soil Model: Undrained (Phi=0)
Unit Weight: 15.7
Cohesion: 15
Piezometric Line #: 1
Ru: 0
Pore-Air Pressure: 0

Soil: 4
Description: Glacial Till
Soil Model: Bedrock
Piezometric Line #: 1
Ru: 0
Pore-Air Pressure: 0

Figure A4

Station 24+600
Seismic Analysis

Description: 04-1120-013-7000 MH / HWY 417 / Limoges-Casselman
Comments: Station 24+600 East Approach Embankment
File Name: Borehole 05-208 East End Seismic.slz
Last Saved Date: 28/06/2005
Last Saved Time: 11:59:33 AM
Analysis Method: Morgenstern-Price
Direction of Slip Movement: Left to Right
Slip Surface Option: Grid and Radius
P.W.P. Option: Piezometric lines with Ru
Tension Crack Option: (none)
Seismic Coefficient: Horizontal



Soil: 1
Description: Embankment - Silty Sand
Soil Model: Mohr-Coulomb
Unit Weight: 20.5
Cohesion: 0
Phi: 32
Piezometric Line #: 1
Ru: 0
Pore-Air Pressure: 0

Soil: 2
Description: Weathered Crust
Soil Model: Undrained (Phi=0)
Unit Weight: 17.5
Cohesion: 75
Piezometric Line #: 1
Ru: 0
Pore-Air Pressure: 0

Soil: 3
Description: Grey Clay
Soil Model: Undrained (Phi=0)
Unit Weight: 15.7
Cohesion: 20.2
Piezometric Line #: 1
Ru: 0
Pore-Air Pressure: 0

Soil: 4
Description: Glacial Till
Soil Model: Bedrock
Piezometric Line #: 1
Ru: 0
Pore-Air Pressure: 0